ROLE OF FLY ASH ON GEOTECHNICAL PROPERTIES OF CLAYEY SOILS

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A thesis submitted to Bahria University, Islamabad in partial fulfillment of the requirement for the degree of BS in Geology

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ABSTRACT

Geotechnical investigation plays a fundamental role in building stable infrastructure. Soil stabilization is a primary element in focus for the development of durable infrastructure. Clayey soils are more problematic as they show considerable volume change. They become hard in the dry state while their strength is decreased in the wet state. This behavior results in a decrease in the stability and durability of any infrastructure. This research is conducted to overcome the problems and to observe the change in behavior of clayey soils of Hassan Abdal area by the addition of fly ash. An attempt was made by adding various fly ash proportions i.e., 5%, 10%, 15%, 20%, 25% and 30%. A variety of laboratory tests are performed. To generate the PI trend with varying fly ash percentages, liquid limit and plastic limit tests are conducted. Maximum dry density (MDD) and optimum moisture content (OMC) are obtained from the proctor compaction test. OMC is added to the clayey soil to perform CBR test to find the load bearing capacity of soil as subgrades. The results showed that PI decreases by adding fly ash up to 10%. Under soaked conditions, the CBR values increase up to 10% of fly ash and decline afterward. Thus, the addition of fly ash in clayey soils can enhance its geotechnical properties.

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LIST OF ABBREVIATION

USCS	Unified Soil Classification System
CCPs	Coal Combustion Products
UCS	Unconfined Compressive Strength
CBR	California Bearing Ratio
MKT	Main Karakoram Thrust
KIA	Kohistan Island Arc
MMT	Main Mantle Thrust
MBT	Main Boundary Thrust
OFZ	Owen Fracture Zone
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
SL	Shrinkage Limit
ASTM	American Society for Testing and Materials
OMC	Optimum Moisture Content
MDD	Maximum Dry Density

CHAPTER 1
INTRODUCTION

1.1 Background

Stabilization of soil is the process of transforming the physical properties of the soil, resulting in a long-term, permanent increase in strength. Their geotechnical properties can be enhanced by adding various materials which in return can increase their properties such as bearing capacity, dry unit weight. These materials can also improve in situ soil performance. The methods whereby the soil performance can be increased come under the umbrella of soil stabilization.

The engineering properties of clayey soils are troublesome. Any change in their physical conditions can lead to problems such as a decrease in shear strength and expansion up to ten percent (10%). This drastic increase in volume damages buildings, roadways, and other infrastructure. Brooks and Sciences (2009).

In stabilization methods, weak soils can be transformed into permanent pozzolanic reactions which reduces the potential for shrinkage and swelling and increases the resistance to freezing and thawing. In addition, stabilized soils have also been slightly modified. Lighter compaction makes it easier to reach the MDD.

1.2 Clayey soil

Clay soils are earthy material that is loose and very soft. It comprises less than 4-micrometer grain size particles and constitutes more than 25% clay particles. These soils are the product of weathered and eroded parent rock. These soils are nutrient-rich therefore considered heavy. Clay belongs to the feldspar group of minerals. The common minerals found in clays are smectite, kaolinite, micas, and chlorite (Kumari and Mohan, 2021). The kaolinite group dos not expand. The mica group can expand to some degree (Nelson and Miller, 1997).

Some types of clayey soils undergo expansion under the influence of moisture and subsequently contracts when it gets dry (Figure 1.1). Due to this swell-shrink behavior, clayey soils are also termed active soils or swelling soils. Considerable volume change is observed on varying the moisture content of the soil.

Figure 1.1 Cracks in clayey soils due to shrink-swell behavior.

Expansive soil is another type of clayey soil whose principal mineral is montmorillonite. The negatively charged clay mineral surfaces attract the cations that are dissolved in water. The water molecules, along with cations, penetrate the layers of clay minerals resulting in the swelling of clay. This undesirable characteristic of clayey soils is responsible for settlement and heaving problems as shown in Figure 1.2 (Rashid, 2015).

Figure 1.2 Road failure due to low strength subgrade soil.

The smectite group clay minerals are formed in the arid zones of the world. In these regions, repeated drought and rainfall conditions prevail. This is the reason that clayey soils are majorly distributed in arid areas (Shahzada et al., 2017). Swelling in clays is influenced by the engineering properties(compaction, density etc.), environmental conditions i.e., in situ moisture content, and the geology of the area (Rashid, 2015).

In Pakistan, most of the areas of Punjab and Sindh are suffering from clayey soil problems. The clayey soils of the districts of Attock, Bahawalpur, Faisalabad, Jhelum, and Rawalpindi undergo very low to low degree of expansiveness. Whereas, Chakwal, Khairpur, Gujranwala, and Sialkot have a low to high degree of expansiveness based on PI (Rashid, 2015).

1.3 Fly ash

The waste residue produced in the electric power plants due to the combustion of coal is termed fly ash (Figure 1.3). During the combustion process, this unburned residue is moved with the gases to the boiler where it accumulates on the walls. Electrostatic or mechanical separators collect fly ash (Ghazali et al., 2019).

Figure 1.3 Heap of Fly Ash near brick furnace in Gujar Khan.

Fly ash can be broadly classified into two main types. Combustion of bituminous or Anthracite rank coal produces Class F fly ash. Similarly burning of coal of lignite or sub-bituminous rank coal generates fly ash of Class C. Both the classes are comprised of either siliceous or siliceous + aluminous materials therefore can be coined as pozzolans. The USCS classifies fly ash as non-plastic fine silt (Kumar et al., 2007).

A wide range of compositions of various elements is produced by the emission of fly ash from coal combustion plants. Silica, ferric oxide, alumina, and other oxides are some of the hazardous materials associated with fly ash. They can cause a variety of health and environmental issues by polluting the air, water, and soil (Ghazali et al.).

Although, fly ash is a residual cheap product but still it can be used in mega construction projects because it causes positive effects on engineering properties such as infilling material, roads and dam’s construction, soil stabilization, bricks and cement manufacturing, ceramic industry, etc. (Malik et al., 2009)

The reaction mechanism behind the soil stabilization using fly ash is because of the minerals present in it, i.e., silica and alumina. A variety of ionized divalent and trivalent cations such as Ca^{2+}, Fe^{+3} $Al^{3+,}$ etc. are found in fly ash under ionized conditions. These ions assist the flocculation of scattered particles of clay minerals which enhances the strength and compressibility properties of soil. Fly ash possess pozzolanic property due to which a gel is formed by its reaction with the water present in the soil. Hence, fly ash cation exchange effectively improves the stabilization of clayey soils (Cokca and Engineering, 2001).

1.4 Literature review

Foundation studies before the infrastructure building are essential because an entire load of any structure is exerted on the underlying geological material. This in turn determines the durability of the structure. A significant volume change is observed in clayey soils when water is introduced. This can damage the structures that are built above clayey soils (Katti, 1978). Chemical stabilization can help to improve their properties. To stabilize clayey soils, cement, fly ash, and lime are some of the common additives used (Singh, 1996).

Fly ash can be utilized in a number of manners (Raymon, 1961) (Toth et al., 1988). It is used in geotechnical engineering as backfill and soil stabilizing material, embankments, and water retaining structures. If soil is treated with fly ash, the shrinkage is decreased more as compared to other additives such as lime or cement (Natt and Joshi, 1984). Fly ash is used for the improvement of the ground since it is an economically viable material (Indraratna et al., 1991).

Pisa tower is a well-known example of geotechnical failure Due to the insufficient knowledge about soil profile and subsurface geology, differential settlement occurred resulting in 5^{o} leaning of the tower (Figure 1.4).

Figure 1.4 Leaning Pisa Tower.

Construction over clayey soils may cause damage to the engineering structures due to settlement problems. The bearing capacity is reduced if the soil gets saturated, also the arid climate can lead to subsequent volume decrease. These shrinking and swelling conditions can cause uneven settlement of foundations. Shallow foundations are more likely to be affected by settling problems as compared to deep foundations. This is because an entire load of shallow foundations is exerted on the soil.

1.4.1 Soil stabilization techniques

a) Stabilization using cement

Soil-cement is commonly used as a base material in stabilizing soils. This product can be obtained by mixing a measured amount of wat and Portland cement with the soil to achieve the required density. It is used effectively to protect the slope of embankments and dams, rail and truck terminals, highway pavement reservoir and channel lining, parking lots, and many other areas. It is an older technique and has been in use for around a hundred years. Soil cement increases the engineering and mechanical properties like permeability, strength, durability, and volume stability. This can be achieved by the addition of certain additives.

b) Stabilization using lime

Three types of lime namely quick lime, hydrated lime, and hydrated lime slurry are formed by the breakdown of limestone at elevated temperatures. Soils can be treated by all three types. Chemical transformation reaction of calcium carbonate into calcium oxide produces quick lime. When reacted with water, the quick lime is transformed into hydrated lime. Mixing of hydrated lime and clay creates strong cementitious bonds. The strength, optimum moisture content, shrinkage limit of the soils can be increased by lime addition which in return decreases its maximum dry density, plasticity index, swelling potential, and liquid limit.

c) Stabilization using fibers

Stabilizing the soil with the help of fibers is the cheapest method. Polypropylene fibers are hair-sized and they avoid leaching and show greater biological and chemical resistance against degradation. Studies show that no prominent change was observed in the Atterberg limits by adding hay fiber but the MDD and OMC decreased with the introduction of hay fiber.

d) Stabilization using fly ash

Combustion of coal leads to the production of certain by-products known as Coal Combustion Products (CCPs). Fly ash is one of these residual products. (Zulkifley et al., 2014) experimented with the tropical soils with fly ash to observe their engineering properties and found that the PI and the LL of the tropical soils were decreased, while CBR (ASTM) was observed. Class F fly ash is mixed with either cement or lime to create its pozzolanic mixtures (Firoozi et al., 2017).

1.5 Problem statement

Clayey soils have the general characteristic to absorb the moisture content and change their volume considerably. This shrink-swell potential is problematic for infrastructures and causes settlement issues. A variety of additives are used to control the properties of clayey soils. An effort is made to stabilize clayey soils by using fly ash.

1.6 Objectives

The objectives of this study are:

i) To observe the geotechnical properties of clayey soils.

ii) To determine the effect of fly ash on the geotechnical properties of clayey soils.

1.7 Methodology

The initial study is based upon the literature review. In the desk study, the problem was pointed out and the relevant research was discussed. The area which is selected for the study is Hassan Abdal, Distt. Attock. On-field visits, clayey soils were identified and samples were collected carefully. As per the demand of our study, fly ash was obtained from a brick factory situated in Gujar khan. Samples were taken to the geotechnical laboratory for testing. Different tests were conducted to classify soil type and to determine its properties like strength, bearing capacity, and the response of the soil to the moisture content. The complete methodological workflow that was adopted to conduct the thesis is shown in Figure 1.5.

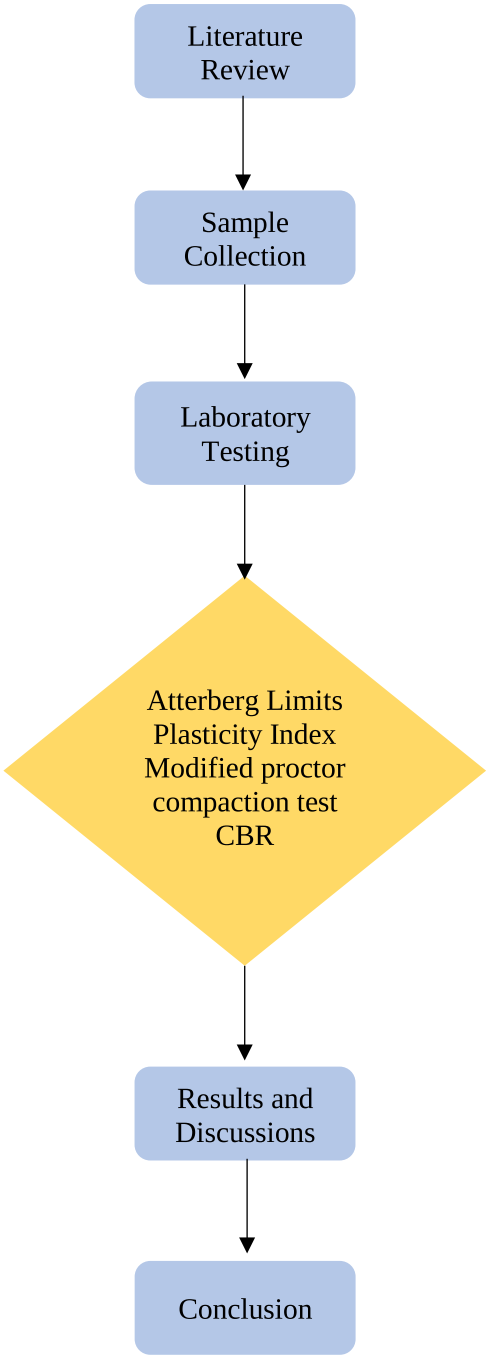

Figure 1.5 Workflow of methodology adopted for research work.

CHAPTER 2
GEOLOGY AND TECTONICS

2.1 General geology of Pakistan

Pakistan is located in South Asian country having 796,096 sq miles of area. India is surrounding it from the East, to the West lies Iran and Afghanistan while China is situated in the North. Arabian Sea bounds Pakistan from the south (Kaplan, 2010). This country is placed at the intersection between Indo-Pakistan tectonic plate, the Eurasian plates, and the Arabian plate. The world's highest mountain ranges, the Karakoram and the Hindukush range are formed due to the collision of these plates during the Quaternary time. This complex process of collision also resulted in the formation and presence of economically important mineral and ore deposits.

The geology of Pakistan has emerged through Gondwanaland which was the Southern fragment of supercontinent Pangea in Permian after its separation which engendered the emergence of Paleo-Tethys Sea thus bisected Pangea into Laurasia (North) and Gondwanaland (South) around 180 million years ago (Figure 2.1). Laurasia comprised of North America and Eurasia whereas Gondwanaland consisted of India, Australia, South America, etc. Africa and Antarctica are the newly formed landmasses that are formed over some time by the accumulation of the relics of the supercontinent (Chatterjee et al., 2013).

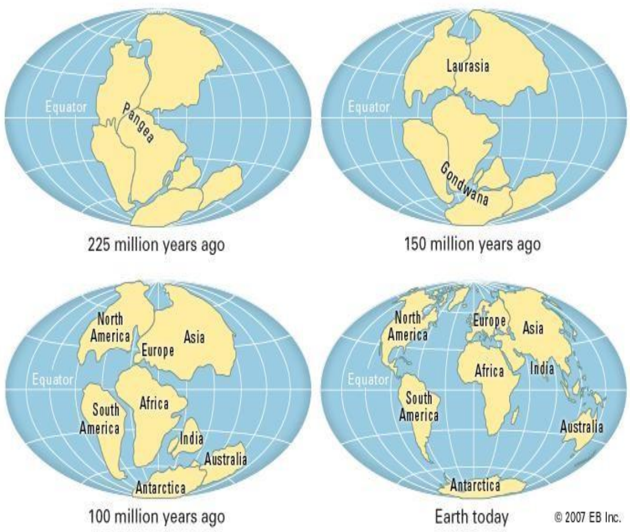

Figure 2.1 Splitting of Pangea and continental drift.

Pakistan possesses a complicated structure due to the intersection of three types of domains; Gondwanian, Tethyan, and Eurasian (South to North). The Eastern part of

Pakistan contains Indian Shield associated with Gondwana land. The southern part of the Eurasian domain is located to the North of Pakistan. The Tethyan domain is wedged between the other two domains. There is a vast diversity of scenes in Pakistan. In the Northwest, the Pamir Mountain hills and Karakoram Ranges are situated. They are convoluted by a network of elevated ranges and lowlands, hostile plateaus followed by the Indus Plain which is a very productive land and ends in the Arabian Sea situated at the South.

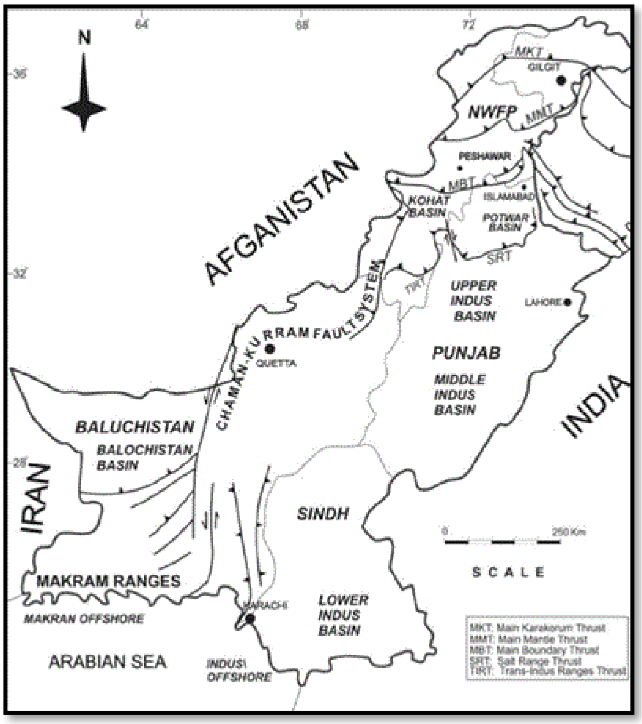

Figure 2.2 Tectonic map of Pakistan.

2.2 Tectonic setting

The intense mountainous topography makes up about three-fifths of the country’s territory while the remaining two-fifths comprises flat plains. The Indian tectonic plate is surrounded by four major tectonic plates, the African plate to the South-West, in the South lies the Australian plate, the Eurasian plate to the North and the Arabian plate is situated at the West as shown in Figure 2.2.

The Indian plate lies in the northern hemisphere. About 140 million years ago, this landmass was initially a part of a supercontinent namely the Gondwana land (Searle et al., 1999 Gough, & Jan, 1999 Gough, & Jan, 1999). The rifting of the supercontinent is thought to occur due to the mantle plume that rises to the surface. This rifting caused the Indian plate to drift at the speed of 18-20 cm/year towards the north and caused the widening of the Indian Ocean.

This plate then collided with the Eurasian plate. As there are multiple domains and fragments involved, there must exist some sort of sutures to bound different domains. Starting from the north, MKT (also known as Shyok Suture Zone) is a suture that separates the Eurasian domain from the Higher or Greater Himalayas which mainly contain Kohistan Ladakh Arcs. Higher Himalayas were formed right after the collision of Kohistan Island Arc (KIA) with Eurasian domain Karakoram Block to be precise around 50 million years ago (Dietrich et al., 1983 & Research, 1983 & Research, 1983). MKT draws a fence between Late Paleozoic meta-sediments in the north and Cretaceous-Tertiary rocks towards the south.

MKT is followed by Main Mantle Thrust (MMT) to its south. It is also known as Indus Suture Zone. It was formed when collision and subduction between the Indian Plate and KIA took place in Eocene. Hence, it is the boundary between the northern part of the Indian plate and the southern part of KIA. This suture contains Nanga Parbat Haramosh Massif which separates KIA (west) and Ladakh Arc (east). Here, mineral deposits of chromite, peridot, asbestos, etc. are in abundance. MMT is followed by Main Boundary Thrust (MBT) which has a hairpin structure containing a series of thrust faults. It marks the foothills of the Himalayas as shown in Figure 2.3. Margalla Hills is located on its hanging wall and contains many folds (Searle et al., 1999).

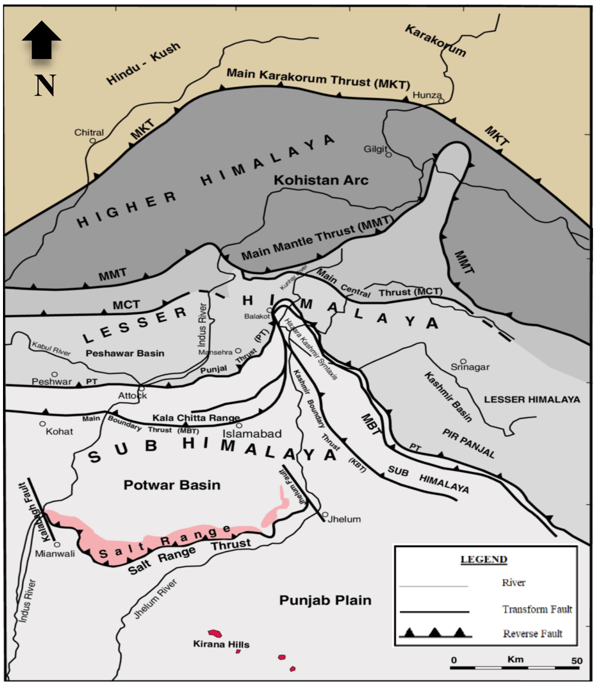

Figure 2.3 Major thrust faults of Pakistan.

2.3 General stratigraphy of Pakistan

Stratigraphy is used to study the origin of the strata, its composition, characteristic age, and its process of evolution. It is predominantly used to study sedimentary rocks but it can also be applied for studying and classification of metamorphic and igneous rocks.

Table 2.1 Major basins of Pakistan.

Indus Basin	Upper Indus Basin (Kohat & Potwar Areas)
	Middle Indus Basin (Punjab Plains & Suleiman Range)
	Lower Indus Basin (Sindh Plains & Kirthar Range)
Balochistan Basin	Balochistan Sub Basin
	Pishin Sub Basin
Offshore Basin	Indus Offshore
	Makran Offshore

When it comes to stratigraphy, Pakistan has three basins; Indus Basin, Balochistan Basin, and Offshore Basin (Table 2.1). Indus is a super basin thus divided into two parts Lower and Upper Indus Basin. Lower Indus Basin is subdivided into Central and Southern Indus Basin. Balochistan Basin Contains Balochistan Sub Basin and Pishin Sub Basin. Lastly, Offshore Basin stretches from the Rann of Katch to the Iranian border. It is bisected into Indus Offshore (east) and Makran Offshore Basin (west) by Murray Ridge (extension of Owen Fracture Zone) (Shah, 1977).

2.4 Geology and tectonic setting of study area

Our study area is Hassan Abdal which is located in district Attock, Punjab (Figure 2.4). Its average elevation is 308m and its area is around 79,284 sq. miles.

Figure 2.4 Route Map of the study area (Google satellite imagery).

It is situated in the Attock basin. It lies to the northeast of Kherimar Hills and south of Gandhar Range and is terminated by MBT. Nathiagali thrust abridges Kherimar Hills to form Hassartang Fault. The formations that are mostly exposed in the study area are Hazara Formation being the oldest of the Precambrian age, Samana Suk Formation of Jurassic, Lockhart Limestone, and Patala Formation of Paleocene. Patala Formation strata aren't exposed in many areas due to erosion. Hazara Formation contains phyllite and slate, Samana Suk contains thick beds of limestone which is very brittle in nature and also shows nodular behavior. However, Kherimer/Hassan Abdal Hills comprised rocks of Jurassic to Paleocene. Overall, Hassan Abdal is located on a block called Hassan Abdal Block. It encompasses an area of around 1838 sq. miles. The stratigraphy of this block is from Jurassic to Miocene (Figure 2.5) (Hylland et al., 1999). The quaternary deposits comprising of alluvial and fluvial basin fill overlay the Miocene rocks.

About 1.8 million years ago (Ma), the uplifting of MBT and the Kala Chitta ranges resulted in sedimentation which was resumed till 0.6 Ma. This long sedimentation period created hundreds of meters thick deposits. Fluvial, lacustrine, stream channel deposits, loess, and the deltaic systems are the source of sedimentation in the flood plains of river Indus and Kabul (Qadri et al., 2017).

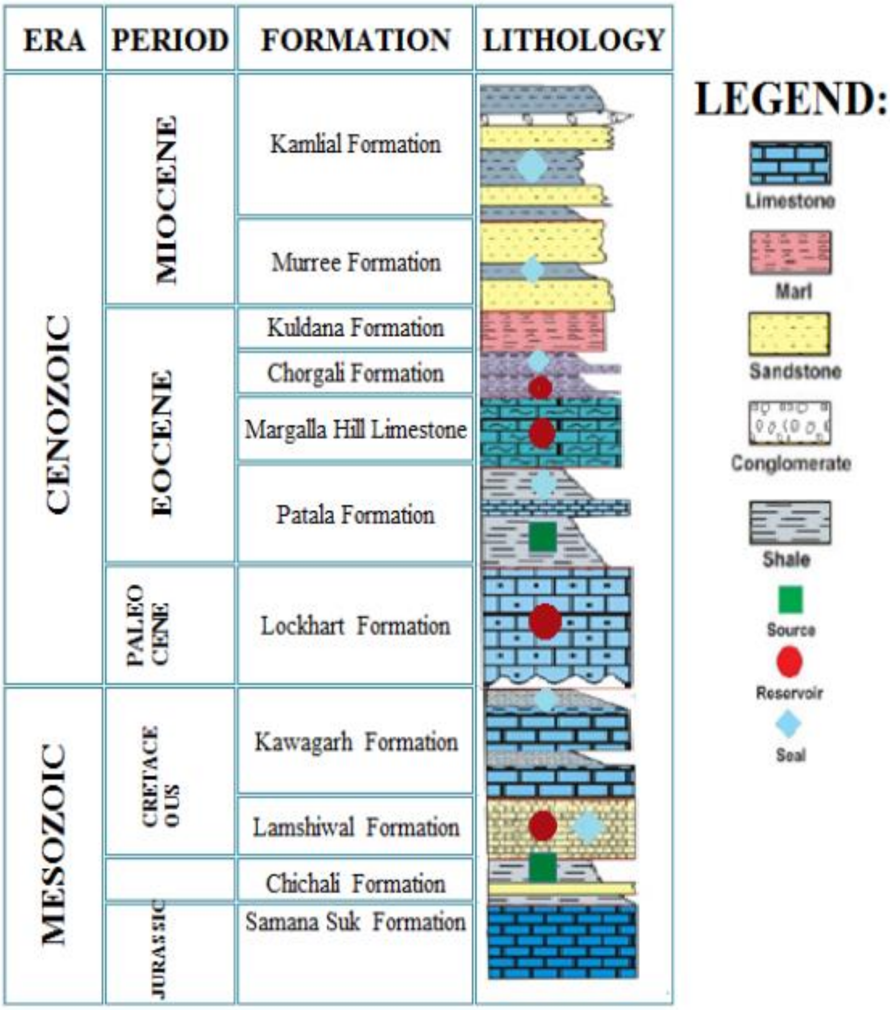

Figure 2.5 Generalized stratigraphy of Hassan Abdal Block (Hylland et al., 1999).

CHAPTER 3
MATERIALS AND METHODS

To investigate the geotechnical properties of the soil like OMC, MDD, permeability, compressibility, and shear strength, a variety of laboratory tests have been performed. These tests include:

1) Atterberg limits
2) PI
3) Proctor compaction
4) CBR

3.1 Atterberg limits

Albert Atterberg defined the fined grained soils based on their limit of consistency. Initially, six limits of consistency were introduced namely, upper viscous flow limit, LL, cohesion limit, SL, sticky limit, and PL. However current engineering usage only includes LL and the PL and sometimes SL as Atterberg limits.

3.2 Liquid limit

Generally, it can be defined as "The moisture content of the soil, expressed in percent, due to which it changes its state from liquid to plastic". The value of the Liquid limit can be determined using the ASTM standard D-4318.

3.2.1 Apparatus

1) Casagrande Liquid limit device (Figure 3.1).
2) A flat grooving tools
3) A metal gage block
4) Water content containers
5) Mixing and storage containers
6) Balance
7) Oven (Figure 3.1)
8) 425μm Sieve (No. 40)

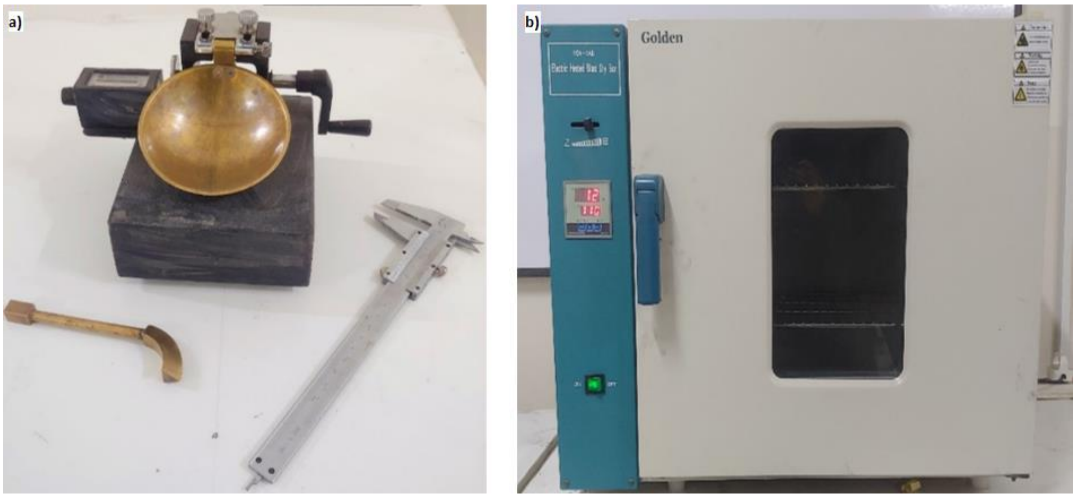

Figure 3.1 a) Casagrande apparatus for the liquid limit test, b) oven for drying samples.

3.2.2 Sample preparation

Pass the material from sieve No. 40. If some of the material is retained on the sieve, remove these soil particles. 150 to 200 g of sample needs to be prepared. For this purpose, take the mixing container add the soil sample and small ratio of distilled water, and thoroughly mix the materials with the help of a spatula. Check the consistency of the sample and cure it for 16 hours. Place the lid to prevent moisture loss.

3.2.3 Procedure

1) Remix the prepared sample thoroughly before starting the test.
2) Calibrate the Apparatus. Now, place the prepared sample in the cup squeeze, and spread it to form a horizontal surface at the place where the cup of Casagrande liquid limit device rests on the base. The deepest point should be 10mm thick.
3) Draw the grooving tool into the soil by holding it against the cup's surface to make an arc by cutting the soil paste perpendicular to the cup surface.
4) Turn on the crank at the rate of 1.9 to 2.1 revolutions in one second. This will lift and drop the cup.
5) Stop the device and check whether the groove has been closed to half an inch. Record the number of blows.
6) If this is the case, take the sample at the bottom of the groove to analyze its moisture content.
7) Repeat the process several times by varying the amount of distilled water and record the number of blows and its water content respectively.

8) To find water content, weigh the soil sample in its original state. Place the sample in the oven overnight at 110°C so that the whole moisture is removed. Now weigh the dried soil sample. By putting these values in the formula, moisture content can be calculated.

3.2.4 Calculations

Moisture content of a soil sample is calculated in weight percentage (wt. %) by using the following formula:

$$w = [(M_{cms} - M_{cds}) / (M_{cds} - M_c)] \times 100$$

$$w = (M_w / M_s) \times 100$$

Where:

M_{cms} = Mass of the container and moist sample.

M_{cds} = Mass of the container and dry sample.

M_c = Mass of the container.

w = Water content.

M_w = Mass of water.

M_s = Mass of dry sample.

Plot a graph between the number of blows and the respective water content. The water content should be plotted on the arithmetic scale and the number of blows on a logarithmic scale. Calculate the water content required to close the groove by 25 blows by drawing the line on the graph. This is the LL of the soil sample

3.3 Plastic limit

It is the percent of moisture content due to which it changes its state from plastic to a semi-solid. ASTM D-4318 standard is used to calculate the value of the PL.

3.3.1 Apparatus

1) Sieve No.40 (425µm)
2) Ground glass plate
3) Washing pan
4) Spatula
5) Distilled water
6) Drying oven
7) Wash bottle
8) Balance (Figure 3.2).
9) Storage containers (Figure 3.2)

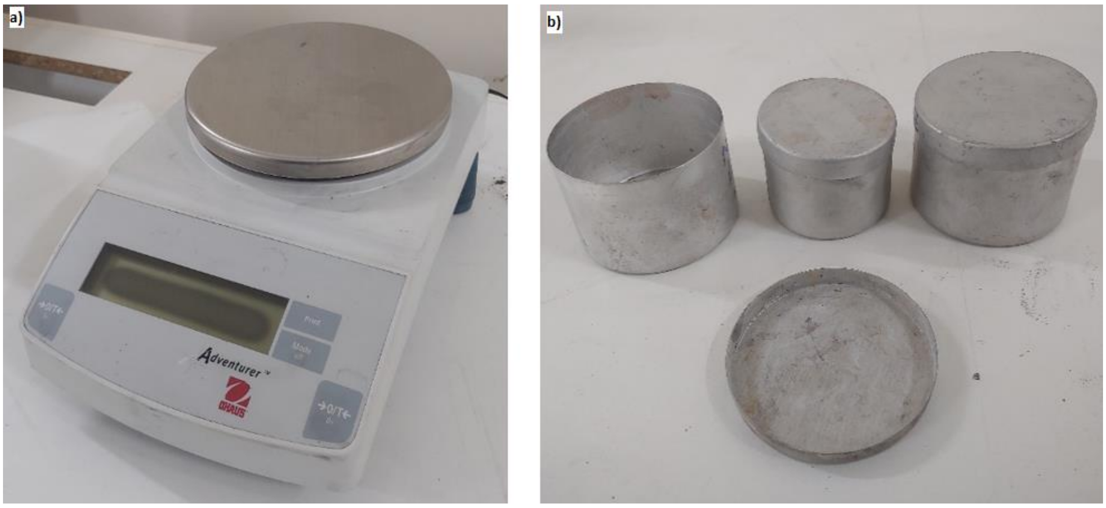

Figure 3.2 a) Balance, b) Storage containers.

3.3.2 Preparation of test specimen

Take around 20g of a soil sample that was prepared to perform the Liquid limit test and dry it such that it can be rolled easily and does not stick to the hands. You can prepare a fresh sample paste by sieving the sample from sieve No. 40 and adding distilled water.

3.3.3 Procedure

1) Take 1.5 to 2g of the sample and roll the soil mass into a uniform diameter thread either with the help of fingers and palm or glass plate.
2) When the roll reaches the diameter of 3/8 inches, it should start to crumble (Figure 3.3).
3) Repeat the process a few times and find the average value of the moisture content of soil (Soil and Rock, 2010).

Figure 3.3 Crumbling of soil at 3mm diameter.

3.4 Plasticity Index

PI is described as the moisture content range at which soil behaves like a plastic material (Soil and Rock, 2010). The results of LL and PL are used for the calculation of the PI with the help of following formula.

$$PI = LL - PL$$

3.5 Modified proctor compaction test

Soils that are used as engineering fill require compaction into a dense state so that the engineering properties including compressibility, shear strength, and permeability can be improved. These properties are determined in the laboratory by using the compaction test according to the standards of ASTM D-1557. Proctor compaction test is useful in determining the compaction percentage and the optimum moisture content significant for geoengineering purposes.

3.5.1 Apparatus

1) Proctor mold comprises of removable base plate and collar (Figure 3.4).
2) A 4.5kg manual rammer can be raised to a height of 18 inches (Figure 3.4).
3) Balance
4) Sample extruder
5) Moisture cans
6) Oven
7) Spray bottle
8) Straight edge
9) A tool for mixing

Figure 3.4 Proctor mold and rammer for compaction of soil.

3.5.2 Sample preparation

Acquire air-dried soil samples. In the mixing pan, take 4.5kg of the sample and break any lumps that are present in it. Sieve the sample with the help of the No.4 sieve. To increase the moisture content to 5%, add distilled water using a spray bottle in the soil. Divide the sample into five equal portions.

3.5.3 Procedure

1) Weight the mold by removing the collar and base plate. Attach them again.
2) Fill the mold with one portion of the sample and compact it by giving 25 blows, falling the hammer from the height of 18 inches. Repeat the process for all five layers.
3) After compacting the final layer, make sure that the compacted soil is above the rim.
4) Carefully take out the collar and trim any excess soil by using a straight edge.
5) Note the combined weight of the mold and the soil.
6) Acquire a representative sample, ideally from the center of compacted soil, for the moisture content test.
7) Break the soil sample, sieve it, and add 5% more moisture to it.
8) Repeat the compaction process and determine the respective water content.
9) The dry density of the soil will increase with the addition of more water and it will start decreasing after a certain amount of moisture is added.
10) Draw a graph to show the relation between the moisture content and dry unit weight. Plotting at least four values will show the trend of the graph (Connelly et al., 2008).

3.5.4 Calculations

W_1 (lb) = Weight of the mold without the base and collar.

W_2 (lb) = Weight of the mold + moist soil.

Weight of the moist soil (lb) = W_2-W_1

Moist unit weight (lb/ft^3) = γ = [(W_2- W_1) / (1/30)]

W_3 (g) = Weight of moisture can.

W_4 (g) = Mass of can + moist soil.

W_5 (g) = Mass of can + dry soil.

Moisture content wt (%) = [(W_4-W_5) / (W_5-W_3)] x 100.

Dry unit weight of compaction (lb/ft^3) = γd = γt / [1+ (w/100)]

3.6 California Bearing Ratio

In 1930, the California state highway department of the USA developed the CBR penetration test to evaluate the load bearing capacity of coarse grade aggregate used in highways and pavements. The purpose of the test is to determine the load-bearing capacity, components used in the layers, and thickness of individual pavements of the road.

CBR can be defined as the ratio of the required force that is applied per unit area to penetrate 0.1 in. (2.5mm) and 0.2 in. (5mm) of the mass of soil at the specific rate with the help of a standard circular piston and the force required to penetrate the corresponding standard material (ASTM, 1883).

3.6.1 Apparatus

1) Loading machine (Figure 3.5).
2) Spacer disc
3) Mold
4) 2.5 kg metal rammer
5) Mixing tools
6) Sieves No. 4
7) Filter paper
8) Swell measurement device
9) Soaking pan
10) Drying oven
11) Balance
12) Straightedge
13) Weights
14) Penetration piston
15) Expansion-Measuring apparatus (Figure 3.5).

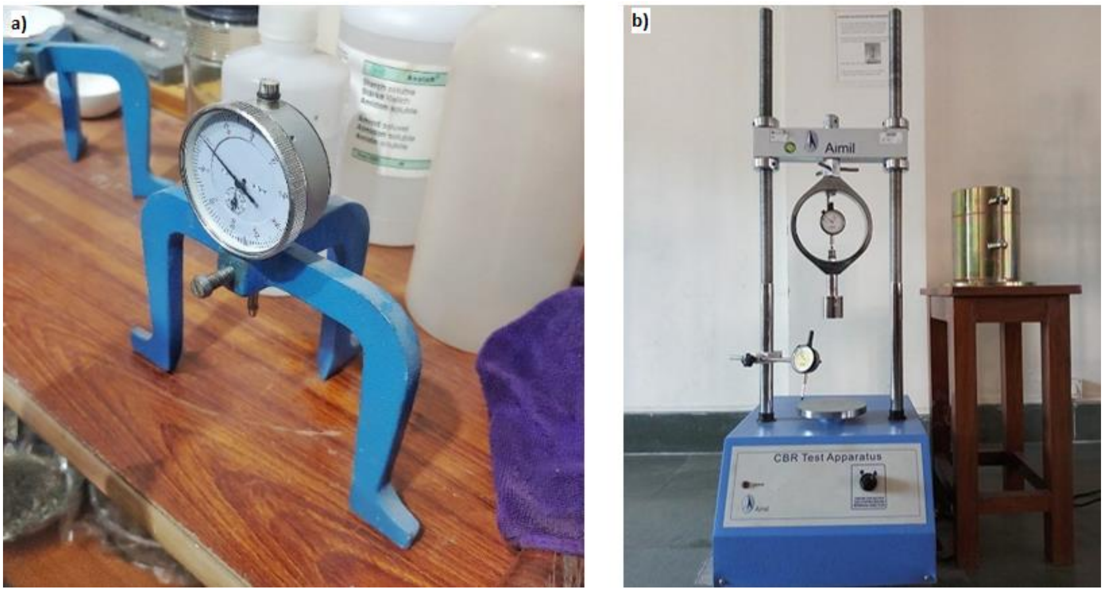

Figure 3.5 a) Expansion measuring apparatus, b) CBR testing machine.

3.6.2 Sample preparation

Sieve the soil sample using No. 4 sieve. Material passed from No. 4 sieve is to be used for CBR. Add enough moisture content (optimum moisture content) in the sample so that soil achieves its maximum dry density.

Arrange the mold by fixing the base plate and extension collar. Place the spacer disc inside the base of the mold and filter paper above the disc. Lubricate the filter paper.

Divide the sample into five parts. Compact the soil sample by applying 56 blows to each layer. Remove the collar and by using straight edge level the top of the mold. Determine the water content before and after compacting the layer.

Remove base plate, filter paper, and spacer disc. Weight the compacted soil along with the mold. Turn the mold upside down and attach the base plate this time with coarse filter paper.

Place 4.54 kg annular weight upon the base plate and soak for 4 days in a soaking pan (Figure 3.6). To determine the swell percentage, measure the height of the sample by using expansion-measuring apparatus before and after soaking. Weight the sample plus mold after 4 days.

Figure 3.6 CBR sample in mold ready to soak.

3.6.3 Procedure

1) Introduce the sample loaded with annular weights (4.54kg) into the compressing machine under the piston.
2) Penetrate the sample with a constant rate of 0.05 in. per minute by starting the compressing machine.
3) There are two indicators in the machine, the dial gauge, and the proving ring. The amount of penetration is indicated by dial gauge whereas applied load is indicated by proving ring.
4) Note the readings of the load after every minute. Also, measure the depth of indentation caused by the piston using a ruler.
5) Calculate the amount of penetration (pounds per square inch) and plot its graph.

3.6.4 Calculations

The dry density of the compacted sample is calculated by the following formula:

$$\rho_d = M_{sas}/V_m$$

Where:

$M_{sac} = (M_{m + ws} – M_m)/(1+w_{ac})$

M_{sac} = Dry mass of soil as compacted (Mg/m^3)

$M_{m + ws}$ = Weight of molded soil plus the weight of the mold. (mg)

M_m = Weight of the mold. (mg)

W_{ac} = Water content of the representative scraps.

V_m = Mold volume. (m_3)

Conversion of the units of dry density

$\gamma_d = 9.8066 \times \rho_d$ (kN/m^3)

Swell percentage is calculated as:

$S = (S/h_i) \times 100$

Where:

s = swelling percentage (%).

S = Vertical swelling (mm).

h_i = Initial height of the sample (mm).

CHAPTER 4
RESULTS AND DISCUSSIONS

4.1 Atterberg limits

To calculate the LL of our soil sample, three trials by varying the moisture content at each percentage of fly ash were performed according to the ASTM standards. Tests were also conducted to calculate the PL by changing the percentages of fly ash (Table 4.8). The results are recorded and the plasticity Index values were calculated at every percentage of fly ash that are presented in Table 4.9 and a trend of PI is generated (Figure 4.11). Shrinkage limit values are calculated from PI and LL and the results can be seen in Table 4.9.

Table 4.1 LL of soil sample without addition of fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
34.86	52.8	48.1	14	35.49848
17	33	29	35	33.33333
32	41	39	40	28.57142

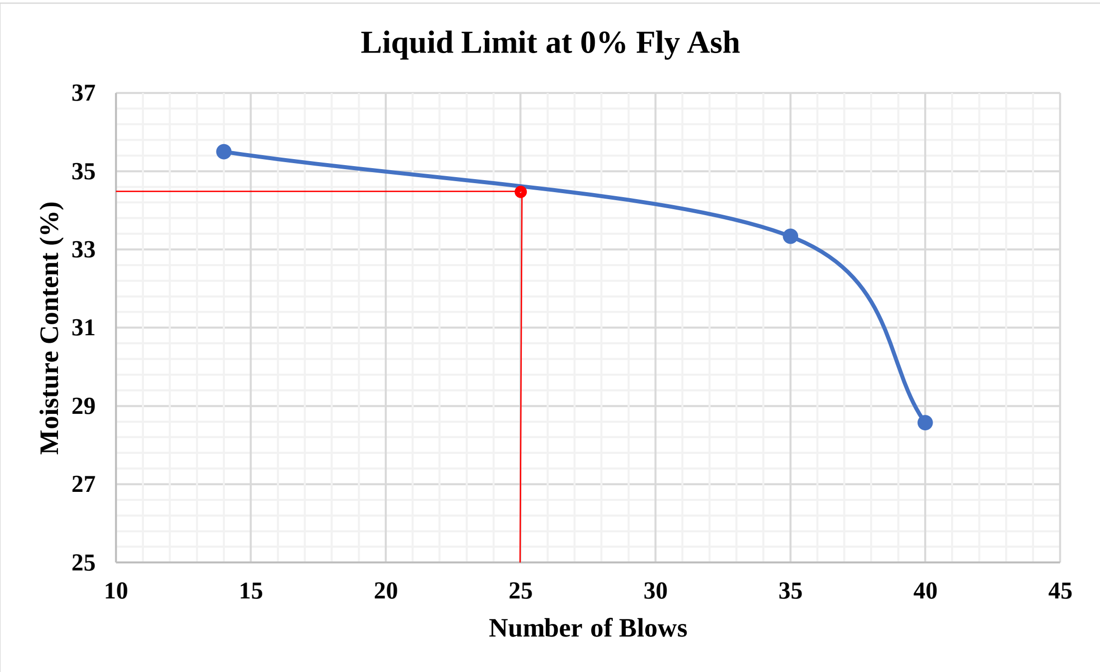

Figure 4.1 LL of soil sample without addition of fly ash.

Table 4.2 LL test for soil by adding 5% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
40	50	48	44	25
65	77	74	29	33.33333
52.36	64.6	61.4	19	35.39823

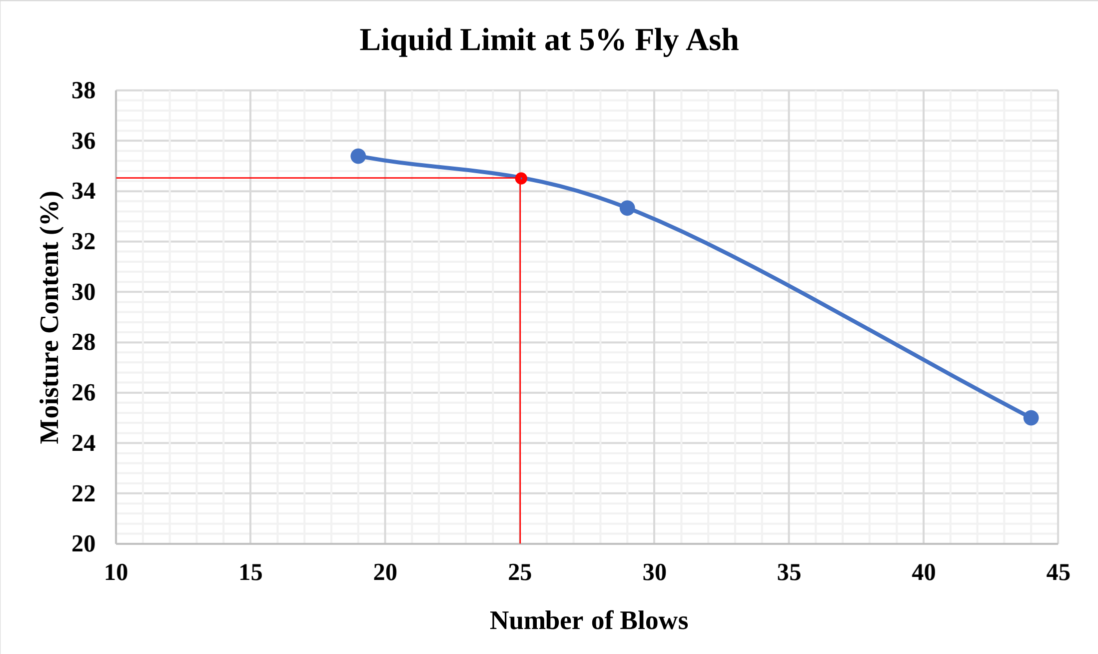

Figure 4.2 LL of soil by adding 5% Fly ash.

Table 4.3 LL test for soil by adding 10% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
40.4	50.2	47.6	10	36.11111111
48.1	62.5	59	28	32.11009174
70.1	78.2	76.4	31	28.57142857

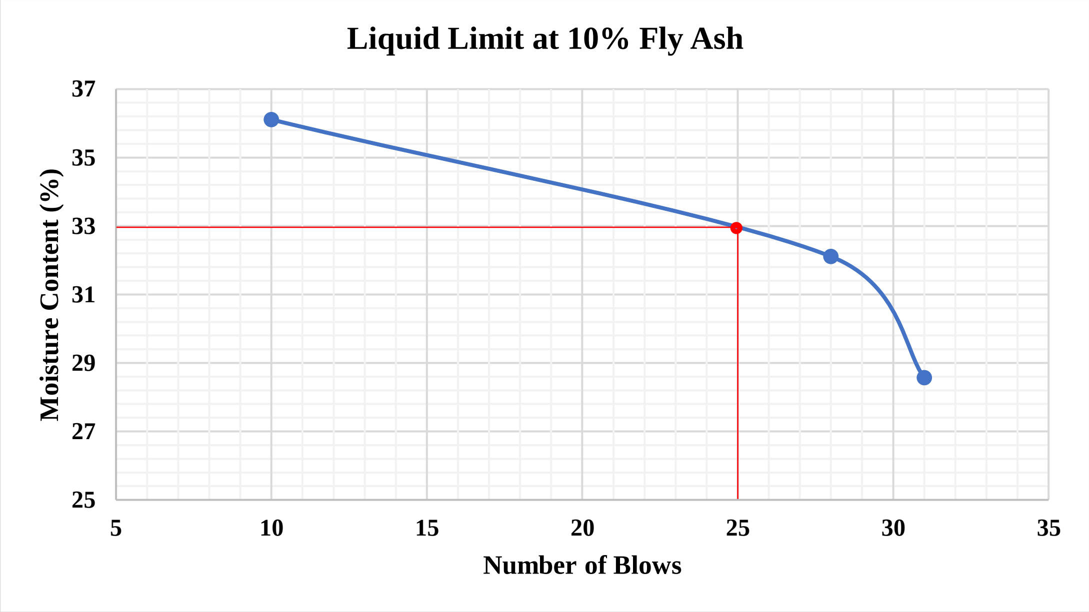

Figure 4.3 LL of soil by adding 10% Fly ash.

Table 4.4 LL test for soil by adding 15% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
47.5	62.3	58.9	41	29.8245614
49.9	64.7	61.2	33	30.97345133
39.7	51	48.3	20	31.39534884

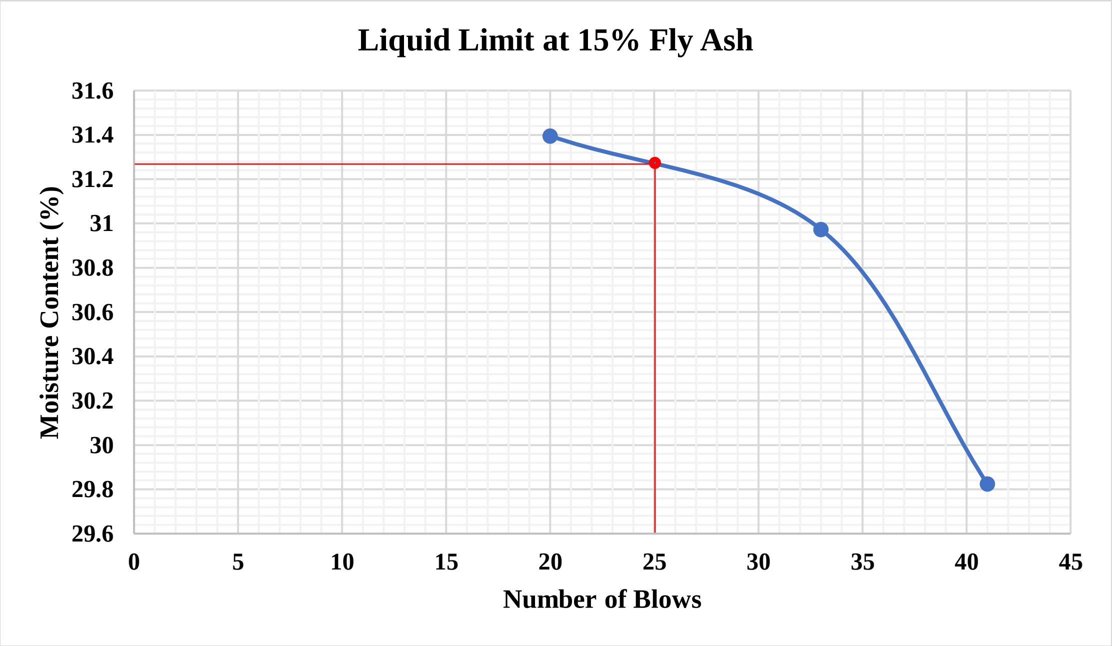

Figure 4.4 LL of soil by adding 15% Fly ash.

Table 4.5 LL test for soil by adding 20% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
15.2	27	23.7	13	38.82352
49.88	66.12	61.9	23	35.10815
16.9	24.9	23	26	31.14754

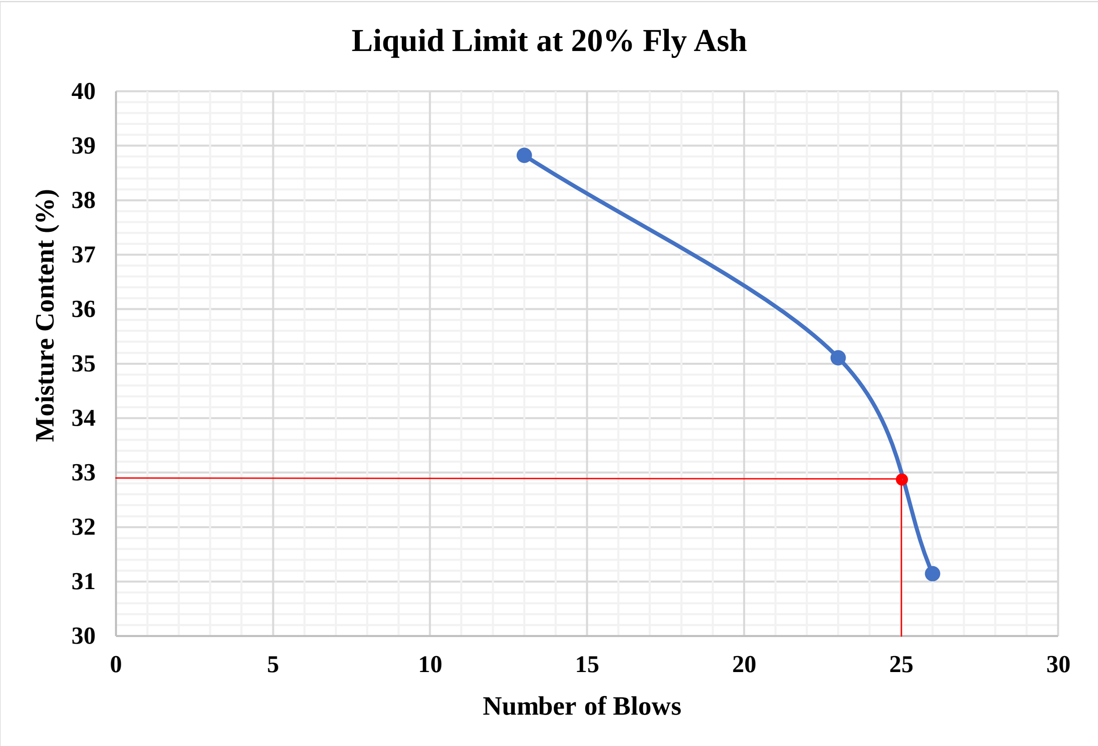

Figure 4.5 LL of soil by adding 20% Fly ash.

Table 4.6 LL test for soil by adding 25% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
43.5	51.4	49.5	49	31.66666
48.2	56.5	54.2	28	38.33333
26.57	42.23	37.7	13	40.70080

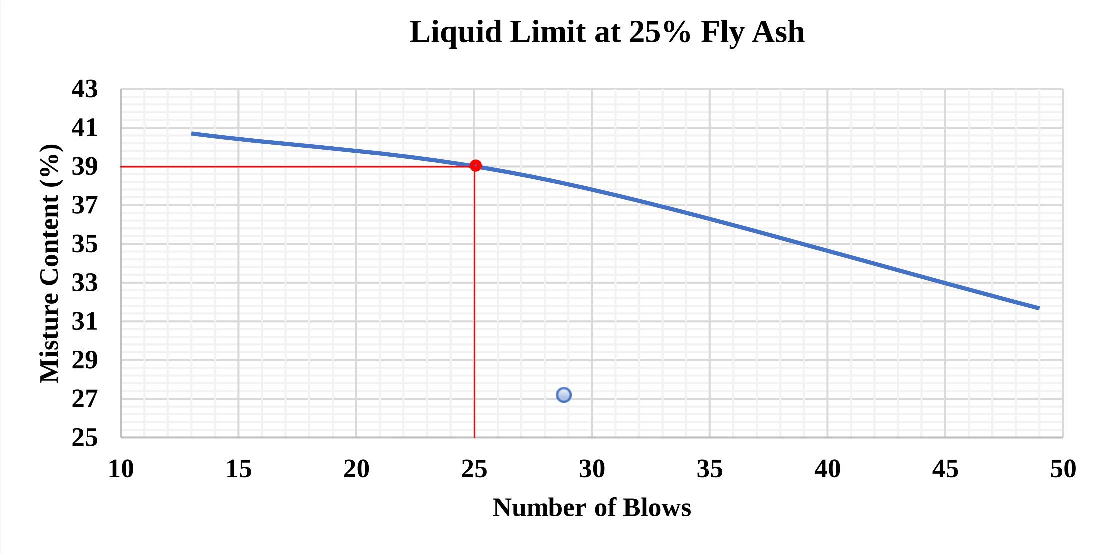

Figure 4.6 LL of soil by adding 25% Fly ash.

Table 4.7 LL test for soil by adding 30% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	No. of blows	Moisture content (%)
26.9	34.4	32.1	32	44.23076
35.9	43.5	41	28	49.01960
33.7	40.4	38.1	19	52.27272

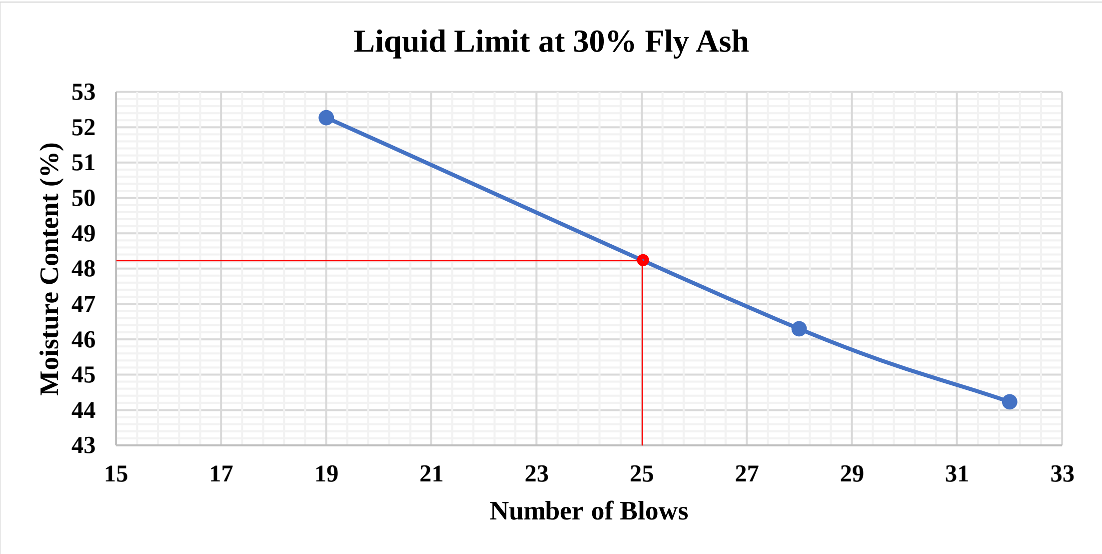

Figure 4.7 LL of soil by adding 30% Fly ash.

Table 4.8 Variation of the plastic limit with fly ash content.

Fly Ash Percentage (%)	Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	Moisture content (%)
0%	36.8	44.6	43.36	18.90243902
5%	11.47	14.12	13.7	18.83408072
10%	71.3	78.7	77.6	17.46031746
15%	16.2	18	17.7	20
20%	19.93	27.89	26.34	24.18096724
25%	20	21.7	21.3	30.76923077
30%	19.31	27.33	25.68	25.90266876

Table 4.9 Variation of plasticity index and SL with fly ash content.

Fly Ash Percentage	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)
0%	34.8	18.902	15.898	16.308
5%	34.5	18.834	15.666	16.313
10%	33	17.46	15.54	15.3
15%	31	20	11	18.33
20%	33	24.18	8.82	22.46
25%	39	30.769	8.231	28.27
30%	48.2	25.9	22.3	19.78

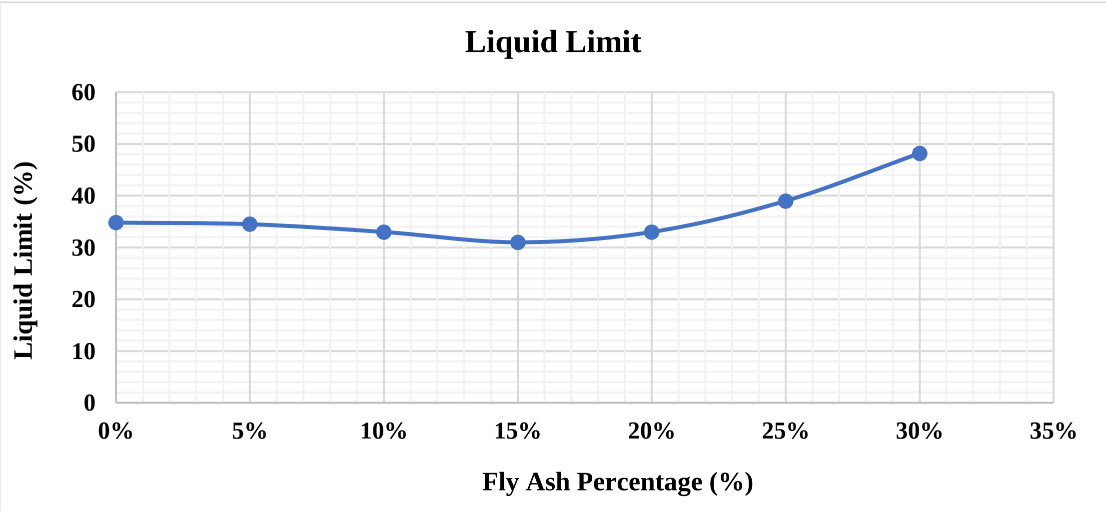

Figure 4.8 LL trend at various fly ash percentages.

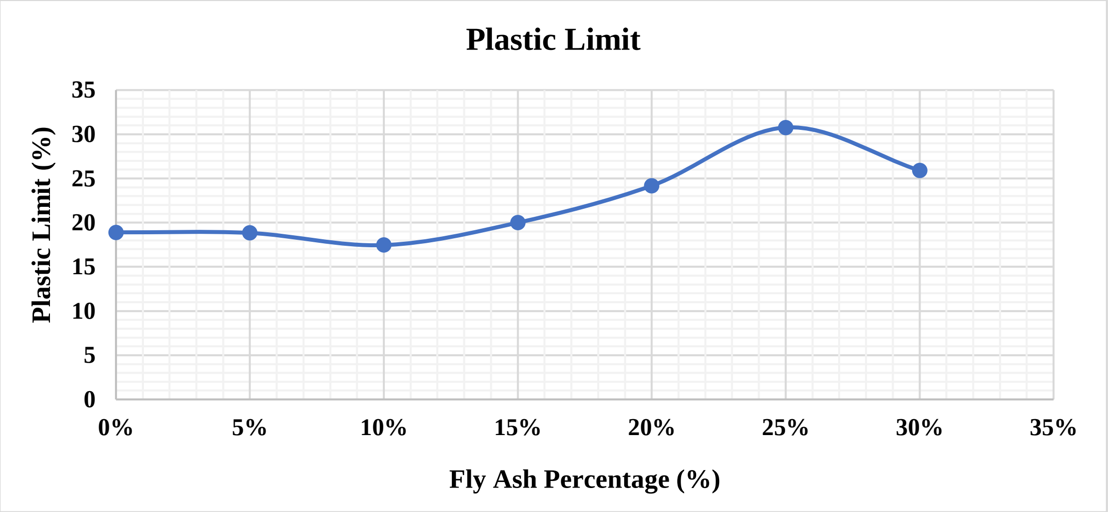

Figure 4.9 PL trend at various fly ash percentages.

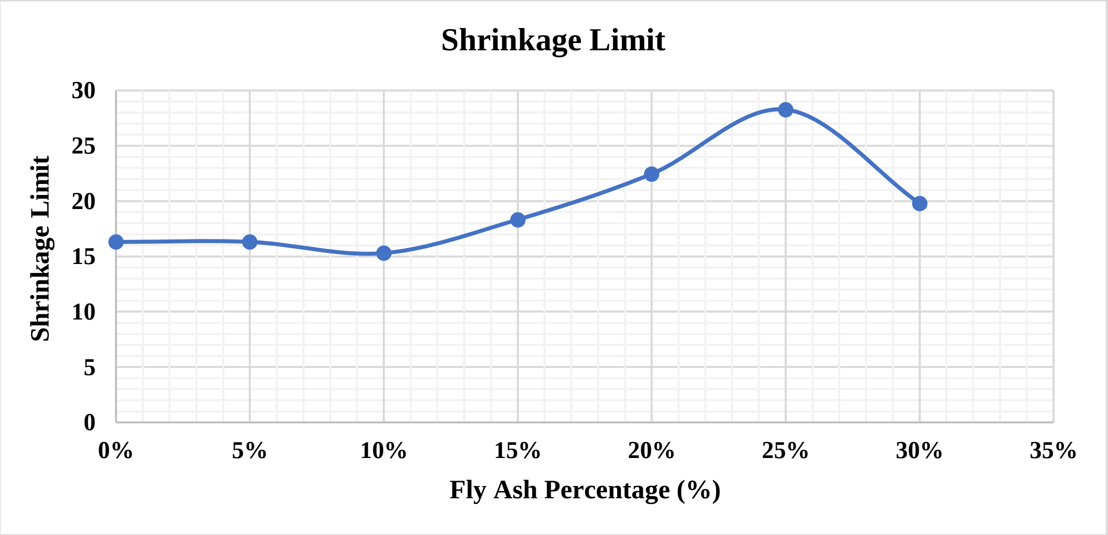

Figure 4.10 SL at various fly ash percentages.

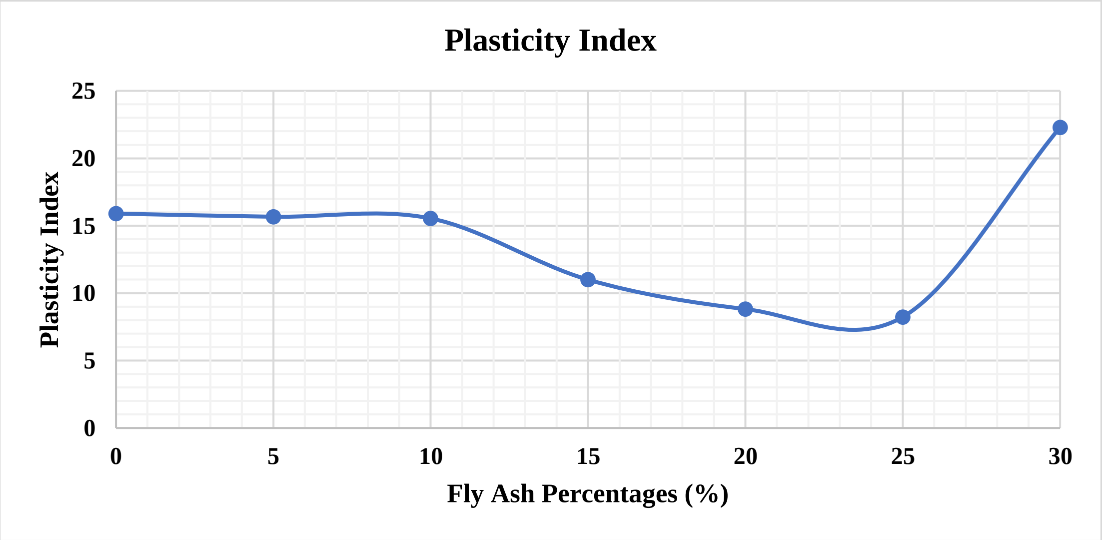

Figure 4.11 Trend of PI at different fly ash percentages.

4.2 Modified proctor compaction test

This test was performed following the standard set by ASTM. The soil sample with varying fly ash percentages was prepared. Five layers of sample were compacted by applying 25 blows to each layer. Multiple trials were performed by adding a constant amount of moisture content until a prominent decrease in the weight of compacted wet soil was observed. The values were recorded in the tables below at each fly ash percentage. The weight of the mold was 5013.8g and its volume was calculated as 2137.05cm^3.

Table 4.10 Modified proctor test for soil without fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	Weight of mold + soil (g)	Moist unit weight (g/cm^3)	Moisture content (%)	Dry unit weight (g/cm^3)
53.59	114.1	109.7	8868.9	1.803935	7.8417	1.6727
51.8	104.7	99.3	9194.1	1.956108	11.3684	1.7564
51.3	111.2	101.8	9460	2.080532	18.6138	1.7540
50.1	123.1	109.7	9289.2	2.000608	22.4832	1.6333

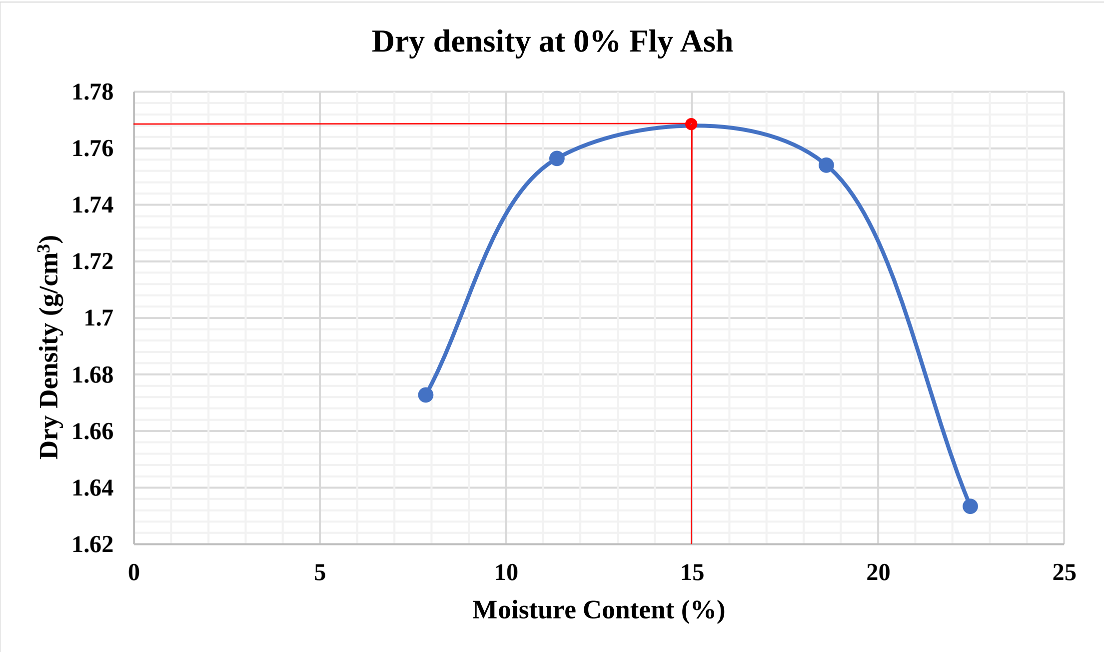

Figure 4.12 Variation of dry density with moisture content without adding fly ash.

Table 4.11 Modified proctor test for soil with 15% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	Weight of mold + soil (g)	Moist unit weight (g/cm^3)	Moisture content (%)	Dry unit weight (g/cm^3)
48.4	76.6	73.21	9055	1.891018	13.66385	1.663693
50.2	101.1	93.21	9338	2.023444	18.34457	1.70979
43.7	101	90.17	9256	1.985073	23.30536	1.609884

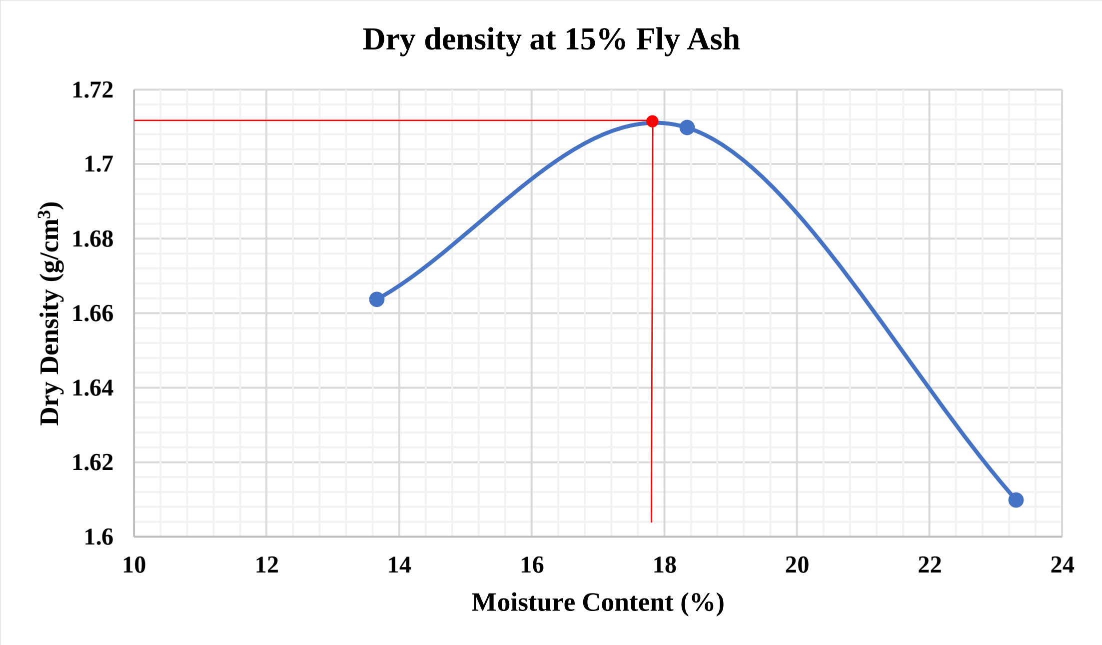

Figure 4.13 Variation of dry density with moisture content by adding 15% fly ash.

Table 4.12 Modified proctor test for soil with 20% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	Weight of mold + soil (g)	Moist unit weight (g/cm^3)	Moisture content (%)	Dry unit weight (g/cm^3)
48.28	114	106.19	8811	1.776842	13.48644	1.565686
48.32	114.77	105.8	8963.9	1.848389	15.60543	1.598874
33.53	82.14	75.03	9172.8	1.946141	17.13253	1.661401
43.6	120.41	108	9370.3	2.038558	19.27019	1.709125
35.52	91.47	81.02	9316.8	2.013523	22.96703	1.637970
53.5	137.1	120.69	9281.7	1.997099	24.42328	1.605553

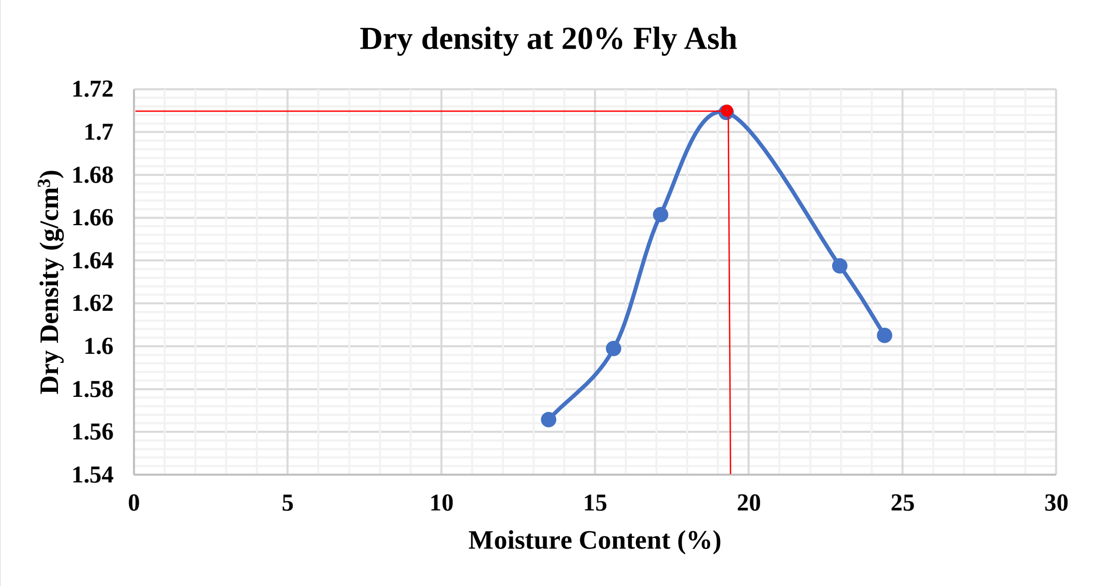

Figure 4.14 Variation of dry density with moisture content by adding 20% fly ash.

Table 4.13 Modified proctor test for soil with 25% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	Weight of mold + soil (g)	Moist unit weight (g/cm^3)	Moisture content (%)	Dry unit weight (g/cm^3)
53.4	95.4	91.9	8546	1.652839	9.090909	1.515102
48.1	80	76.8	8860	1.799771	11.14982	1.619229
50	81	76	9245	1.979926	19.23076	1.66058
49.8	128.7	113.8	9311	2.010809	23.28125	1.631074
51.1	113.4	100.2	9082	1.903652	26.88391	1.500310

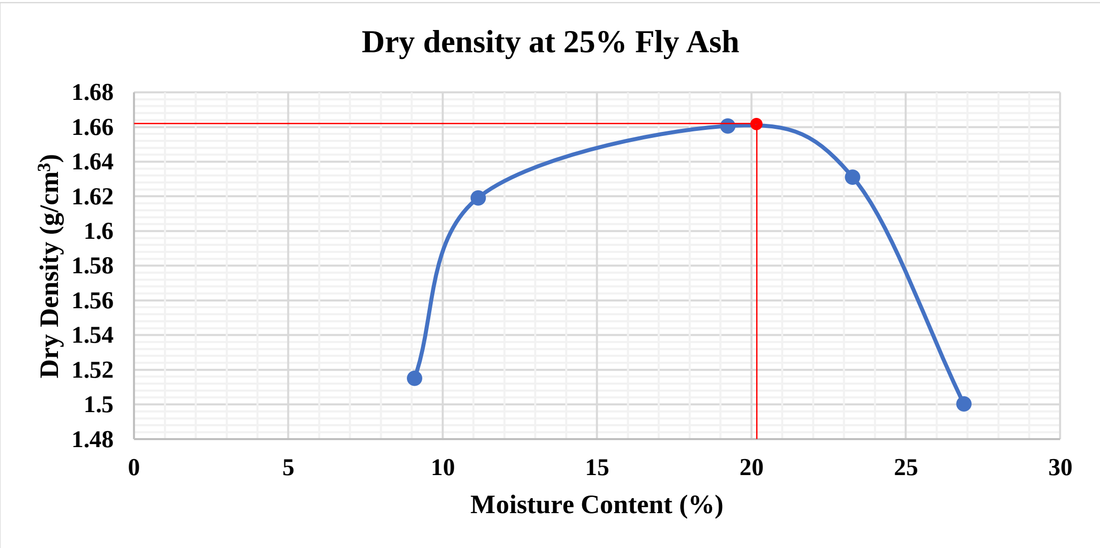

Figure 4.15 Variation of dry density with moisture content by adding 25% fly ash.

Table 4.14 Modified proctor test for soil with 30% fly ash.

Weight of can (g)	Weight of can + soil (wet) (g)	Weight of can + soil (dry) (g)	Weight of mold + soil (g)	Moist unit weight (g/cm^3)	Moisture content (%)	Dry unit weight (g/cm^3)
35.4	83.7	79.96	8617	1.686063	8.393178	1.555506
33.2	59.9	57.08	8885.9	1.81189	11.80905	1.620522
49.7	104.2	95.59	9219	1.967759	18.76226	1.656889
42.9	118.4	102.77	9006	1.868089	26.10656	1.481358

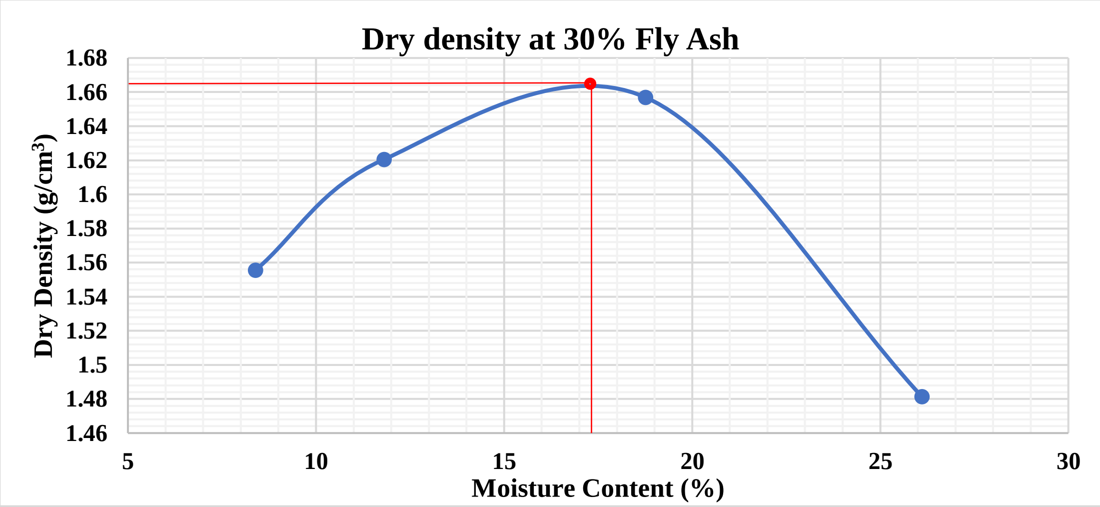

Figure 4.16 Variation of dry density with moisture content by adding 30% fly ash.

The values of OMC and MDD were calculated at different fly ash percentages as shown in Table 4.15 and a general curve was generated to show their trend with fly ash in Figure 4.17 and Figure 4.18 respectively.

Table 4.15 Values of MDD and OMC with varying Fly ash percentages.

Fly ash Percentage	Optimum Moisture Content (%)	Maximum Dry Density (g/cm^3)
0%	15	1.77
15%	18	1.71
20%	19.5	1.7
25%	20	1.66
30%	17.5	1.67

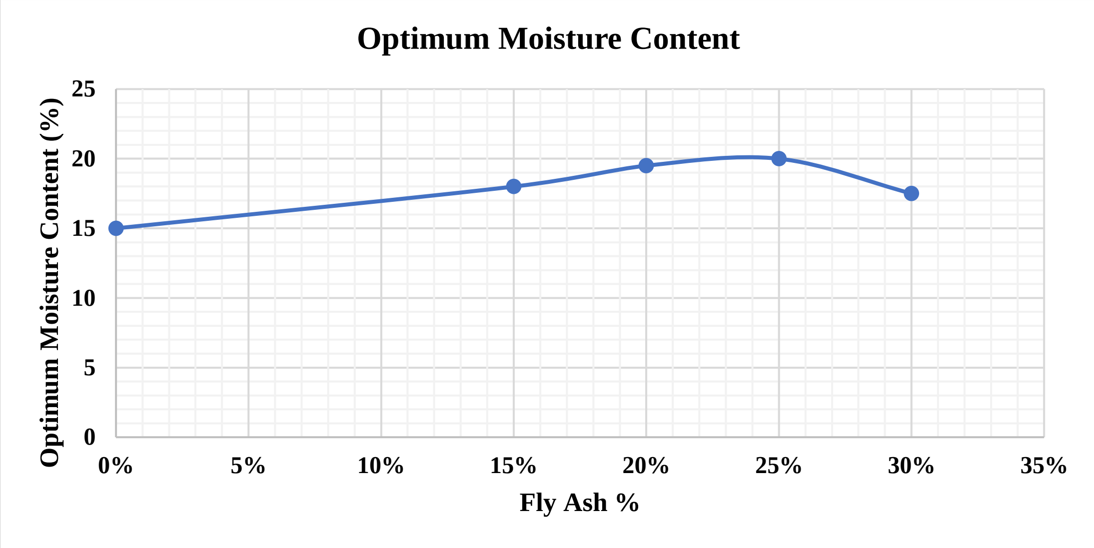

Figure 4.17 Variation of OMC with different fly ash quantity.

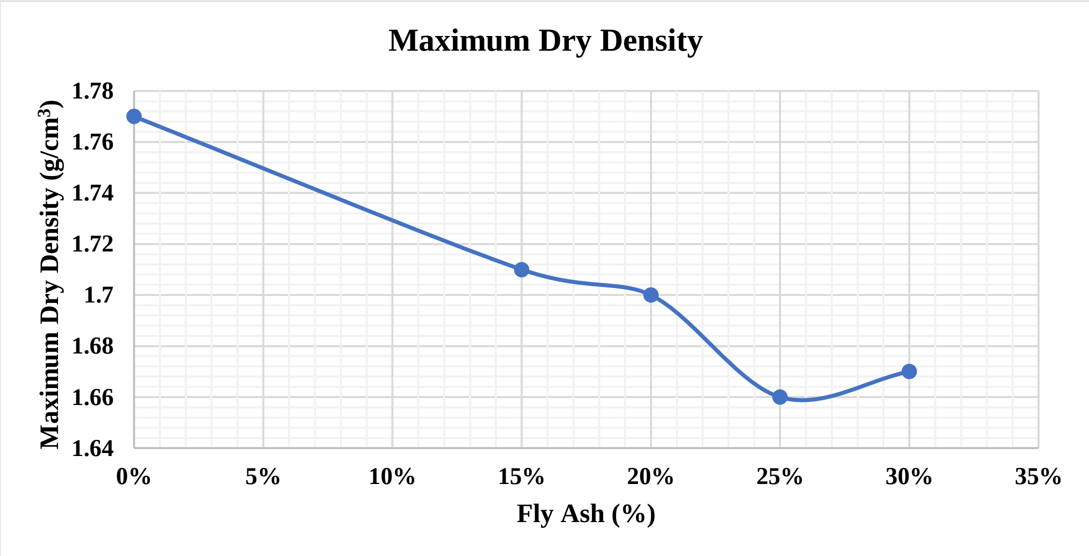

Figure 4.18 Variation of MDD at different amount of fly ash.

4.3 California Bearing Ratio

At different fly ash percentages, CBR was also performed. For this, the sample was prepared by adding optimum moisture content for each percentage calculated from the proctor compaction test. After compacting the samples in the mold, gauges were fixed on all molds and they were soaked for 96 hours in water. Gauge readings were recorded after 24, 48, 74, and 96 hours. Soaked samples were then mounted on a CBR machine and penetrated one by one by the plunger. Calculations were performed using all the readings from CBR and the following trend of CBR percentage was generated.

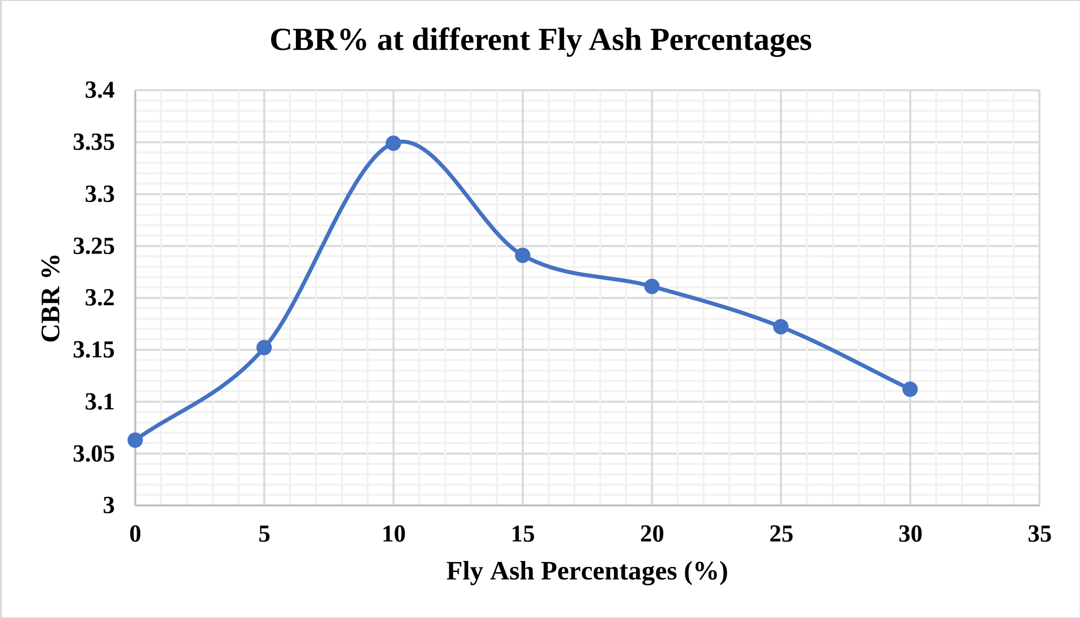

Figure 4.19 Variation in CBR by varying fly ash quantity.

4.4 Discussions

The altering fly ash percentages from 0% to 30% is added to clayey soil to observe the behavior of soil.

Atterberg limits (LL and PL) at different fly ash percentages were calculated. The results were analyzed to generate a Plasticity Index curve. A continuous decrease in the PI values with increasing percentages of fly ash was seen at 10% fly ash. Those soils which have decreasing PI have low shrinkage as well. Fly ash cements the grains of soil together and restricts the individual movement of soil particles. Hence, by increasing fly ash, the shrink-swell behavior of clayey soils is reduced.

By increasing fly ash percentage, MDD decreases while OMC increases. This is because fly ash particles are coarser than clay but their density is lesser. Reduction in MDD also decreases the capability of clayey soils to shrink and swell in a compacted state. Increasing fly ash percentage increases the water holding capacity between particles of soil which results in flocculation. That is why an increase in OMC is observed.

A soaked CBR test was performed with different fly ash percentages. CBR values were calculated and a graph was generated between the fly ash percentages and values of CBR. It was noted that the highest value of CBR was achieved by mixing the soil with 10% fly ash.

CONCLUSIONS

The conclusions which can be drawn from the tests results are:

Atterberg limits were calculated for clayey soil and results obtained showed decreasing LL, PL, and SL by adding fly ash up to 10%. Due to this the PI trend also decreases.

CBR trend was generated which showed maximum value at 10% and decreases after that. Hence, PI and CBR trends suggest that 10% fly ash can be used with clayey soils for landfilling and subgrade material.

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