CALIFORNIA BEARING RATIO (CBR) AND STRENGTH BEHAVIOUR OF COMPACTED DHAKA CLAY AS ROAD SUBGRADE

A Project

By

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MY PARENTS

DECLARATION

It is hereby declared that this project or any part of it has not been submitted elsewhere for the award of any degree or diploma.

(Md. Mahabubuzzaman)

The project titled 'California Bearing Ratio (CBR) and Strength Behaviour of Compacted Dhaka Clay as Road Subgrade' submitted by Md. Mahabubuzzaman, Roll: 9304219(P) Session 1992-93-94 has been accepted as satisfactory in partial fulfillment of the requirement for the degree of Master of Engineering in Civil Engineering on 30/09/2001.

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ABSTRACT

In this study California Bearing Ratio (CBR) and other strength characteristics are investigated for compacted Dhaka clay. The clay samples were collected from Pallabi Phase II, land project area of Eastern Housing Ltd. on the east side of National Botanical Garden. After air drying, the soil samples were broken down for use in different laboratory tests. The samples were compacted at modified compaction energy at different percentage of water content. Unconfined Compression Strength test and California Bearing Ratio (CBR) test for both soaked and unsoaked conditions were conducted. Swelling was measured during soaking and Vane Shear test was performed for soaked samples using a torvane.

The results indicate that unconfined compressive strength, dry density and CBR value increase with the increase of water content on the dry side upto Optimum Moisture Content (OMC), but with the addition of more water beyond the OMC these values decrease. During preparation of unconfined compression strength test samples and also from the failure pattern, it is observed that sample prepared with water content less than OMC becomes more stiff and rate of strength gain is high. Deformation at failure in unconfined compression test is lower for samples with lower percentage of moulding water. With the increase of water content, deformation at failure increases. At twenty percent or more water content there is no peak value and the stress-strain curves run parallel to the abscissa.

Unsoaked CBR value is quite high as compared to soaked CBR value. At 9.5, 12.0, 15.3, 18.5, 19.9, 22.4 and 27.5 percentages of water content, soaked CBR values are 5.65, 16.67, 21.37, 17.42, 13.05, 6.31 and 3.60 percentages, whereas unsoaked CBR values are 37.41, 45.57, 53.29, 32.13, 17.66, 8.10 and 5.50 percentages respectively.

During soaking swelling occurs. With the increase of moulding water content, percentage of swell decreases and at twenty percentage or more moulding water content swelling becomes zero due to better saturation, but percentage of swell increases with the decrease of moulding water content. At 9.5% moulding water content percentage of swell is 2.8 and it is observed to be the maximum value.

Based on the above experimental tests results correlation are developed among Unconfined Compressive Strength, Water Content, California Bearing Ratio, Dry Density, Vane Shear Strength and Swelling index. Some relationships are found linear and some are found nonlinear.

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NOTATION

CBR_s	-	Soaked California Bearing Ratio
CBR_u	-	Unsoaked California Bearing Ratio
Cc	-	Compression index
Cc (iso)	-	Compression index from isotropic consolidation test
$Cc_{(Ko)}$	-	Compression index from Ko consolidation test
C_s	-	Percent of swell
E_{50}	-	Stress-strain modulus or secant modulus at 50% of ultimate shear strength
I_p	-	Plasticity Index
K_o	-	Co-efficient of earth pressure at rest
MDD	-	Maximum dry density
OCR	-	Over consolidation ratio
OMC	-	Optimum moisture content
q_u	-	Unconfined compressive strength
S	-	Undrained shear strength ratio for normally consolidated clay (Su / σ_v')
SM	-	Silty sand
S_r	-	Degree of saturation
S_u	-	Undrained shear strength
S_{uv}	.	Vane Shear Strength
w	-	Water content
W_L	-	Liquid limit
W_P	-	Plastic limit
γ_{dry}	-	Dry density
σ_v'	-	Vertical effective stress
ε	-	Axial strain

CHAPTER 1
INTRODUCTION

1.1 General

The load from the vehicle moving on the road surface is ultimately transferred to the road subgrade, which may be natural soil deposit or compacted fill materials. The ability of subgrade soil to support the imposed load is governed by the shear strength of the soil and ultimately subgrade strength determines the pavement thickness. As such shearing strength of soil becomes of primary importance for highway design as well as design of foundation of a structure.

The shear strength of a soil deposit may be related to the type of clay minerals, water content, density, and also to the consolidation pressure to which the soil had been subjected to in the past, i.e. the soil stress history. Any change in shear strength of a clay may be affected by the above factors.

Bangladesh is a developing country; its development is going on through construction of various infrastructures like roads & highways, airfields, houses, institutions, markets, drainage structures etc. The Modhupur clay (Dhaka clay), with varying characteristics forms a substantial part of land area in Bangladesh and is used as subgrade for road construction in Dhaka, Gazipur, Tangail, Mymensingh, Manikganj, Noagoan, Nawabganj and part of Rajshahi and Bogra district.

1.2 Geology of Dhaka Clay

Sediments from the adjacent highlands like the Himalayas form the Bengal Basin. The greater part of this land building process has been due to the sediments carried by the Ganges and the Brahmaputra rivers.

From the studies of Morgan et al (1959), Hunt (1976) the geological formation of the land of Bangladesh can be broadly classified into three group i.e.

Pleistocene sediment, Uplifted Alluvium Terraces and Recent Floodplain. The Pleistocene sediments are flood plain deposits of earlier Ganges and Brahmaputra. They occur in several extensive areas above the level of present flood plains. There are also indication of differential movement of these Pleistocene deposits.

The city of Dhaka stands on the southern part of Modhupurgar, which is formed by older Pleistocene sediments. The Dhaka city is at an elevation of 20 feet to 27 feet above the Mean Sea Level. In general, top layer of which extends upto a depth of 20 feet to 25 feet and is a mixture of silt and clay. Deposits of sand and gravel occur at relatively deeper horizons with a sequence of finer material at top and coarser material downward. The consistency of the top layer is medium to stiff and the soil is overconsolidated. A description of the soil profile over Dhaka is provided by Eusufzai (1967).

It is also covered by highland and lowland alluvium in some places. Dupi Tila formation underlies the Modhupur clay residuum, which is locally called Dhaka clay. Dupi Tila formation consists of clay, fine sands, medium sands, clayey lenses, sub-ordinate shale and poorly consolidated sand stone. It is massive thick bedded, yellow to brownish colour.

1.3 The Research Area

In spite of a good number of study on Dhaka clay no comprehensive study is made on Dhaka clay regarding CBR and other strength behaviour as road subgrade. It is important for practical purposes to undergo a detailed study regarding CBR and strength characteristics of Dhaka clay as road subgrade. For determining the strength of subgrade soil CBR test under both soaked and unsoaked condition as specified by Corps of Engineers Method is performed (Yoder and Witczak, 1975). There are several methods for determining shear strength of soil and Unconfined Compression Strength test one of the methods which is widely used for determining shear strength of soil taking shorter time in comparison with other method and used normally for the design of foundation.

The present research is performed with a view to investigate the relationship among CBR, unconfined compression strength, water content and dry density both for dry and wet side of optimum water content for Dhaka clay.

In the research program compacted samples have been used. This is decided primarily because the aim of the program is to study strength behavior at different water content in terms of CBR and unconfined compression strength at different water content, which is not possible from undisturbed sample. Moreover, for road construction compacted fills are used as road subgrade. It is also quite hard to prepare undisturbed soil sample at k_0 condition in large quantities.

1.4 Objectives

The research program is directed in assessing the geotechnical properties of compacted Dhaka clay and to find out a relation between various strength parameters (CBR, Unconfined Compression Strength, Vane Shear Strength etc.). Specific objectives of the present study are to attain the followings for Dhaka clay:

I) To determine strength parameters and CBR value of Dhaka clay as road subgrade.

II) To establish correlation among index properties, strength parameters and CBR.

III) To examine the variation in correlation among index properties, strength parameters and CBR based on wet and dry side of maximum dry density.

1.5 Overview of the Project

Dhaka Clay forms a substantial portion of the land area of Bangladesh. Roads and Highways are being constructed all over the country as a sign of development to facilitate the communication. Dhaka the capital of Bangladesh, is the most important city in the country. It is expanding rapidly with the construction of different types of structures everywhere. Similarly, its importance is also expanding and need to be well communicated with other places of the country by constructing new roads and highway as well as rehabilitating the existing roads. Therefore, it is expected that a large number of roads and highways will be constructed / rehabilitated to facilitate well communication between the different cities and the capital, Dhaka as well.

A good number of studies have been made for the investigation of geotechnical characteristics of Dhaka clay (Ameen, 1985; Uddin, 1990; Siddique and Safiullah; 1995). These studies give permeability, consolidation and settlement characteristics of Dhaka clay. No comprehensive study is made on Dhaka clay regarding CBR and strength behavior as road subgrade. Serajuddin and Azmal (1991) established correlation of strength parameter (Unconfined Compression Strength) with CBR value of alluvium soil as road subgrade. Detail geotechnical investigations of Dhaka Clay in light of road construction will help road Engineers for road design and prepare project proposal quickly.

This study for Dhaka clay is carried out according to the following phases:

(i) Firstly, Grain size analysis, Atterberg's limit and Modified Proctor tests are performed for the collected soil samples.

(ii) Unconfined Compression Strength and California Bearing Ratio (both soaked and unsoaked condition) are determined for compacted soil samples at different percentage of moulding water content. During soaking swelling index is determined. Vane shear strength using torvane is also determined.

(iii) Finally, the correlation among index properties, strength parameters and CBR are established for compacted Dhaka clay.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

One of the most important aspects of the study of the soil mechanics is the prediction of load carrying capacity. Different types of infrastructures like roads, building, bridges, stadium, ports, harbor etc are being constructed on the ground.

Road structures are constructed for the movement of various transports like buses, trucks, car, auto rickshaws, rickshaw, bullock carts etc. Load from these traffics is ultimately distributed on the ground.

The load carrying capacity of soil is measured by various methods like Unconfined Compression test, California Bearing Ratio (CBR) test, Direct Shear test, Triaxial test, Vane Shear test, Plate Load test etc.

The engineering properties of soil depend on the composite effects of several interacting and/or inter-related factors. These factors may be divided into two groups i.e. compositional and environmental. Compositional factors consist of type and amount of mineral, shape and size distribution of particles, type of adsorbed cations, pore water composition etc. Environmental factors include water content, density, confining pressure, temperature etc.

Load carrying capacity of different type of soil is different. The principal factors affecting the load carrying capacity are soil texture, moisture content and density. Consolidation characteristics also affect load carrying capacity of a particular soil.

From the geological information of Bangladesh there are six regions each with different engineering properties and load carrying capacity as well. Dhaka clay, which is in the uplifted alluvium terraces, differs from other five groups.

2.2 Geology of Bangladesh

Bangladesh is criss-crossed by many rivers and is itself a flat delta. It consists of a large alluvial basin with Quaternary sediments deposited by the Ganges-Padma, the Brahmaputra-Jamuna and Meghna river systems and their numerous tributaries and distributaries. The north eastern and eastern boundaries of Bangladesh follow the mountainous areas in India and Mayanmar (Burma). The uplifted alluvium traces found in Modhupur area and some north western part of Bangladesh.

The geology of Bangladesh is divided into six regions and out of these three are distinct (T. Hunt, 1976) based on formation, which are as follows:

Recent Floodplain: This occupies roughly 70% of the total land area. The Quaternary floodplain sediments were deposited mainly by the Ganges, Brahmaputra, Tista and Meghna river and their distributaries. Sands frequently occupy large areas in the northeast and southwest. Generally elsewhere, however, silts and silty clay predominate. Piedmont deposits are usually found close to the existing hill areas and usually overlie older floodplain alluvium.

Uplifted alluvium terraces: These terraces, commonly known as Modhupur and Barind tracts under Dhaka, Gazipur, Narshingdi, Manikganj, Tangail, Naogaon, Nawabganj, Joypurhat, Rajshahi, Bogra and partly in some other districts. Both Madhupur and Barind area are underlain by a relatively homogenous clay known as Modhupur or Dhaka clay. The clay is underlain by fine sand and the land systems are fault blocks, which have been uplifted and locally tilted. This type of clay is found to be pre consolidated.

Tertiary and Pleistocene hill formations: The tertiary and Pleistocene hill formations consist almost entirely of unconsolidated or poorly consolidated sand-stones, silt stones and shales. These hill formations run roughly north-south in the Chittagong Hill Tract and south of Sylhet district. The higher ranges, which is folded and faulted, generally contain sandstone and shale of the Shurma and

Tipam formations. The lower rounded hills comprise unconsolidated sand stones of the Dupi Tila formation of mid-Miocene to Pliocene age. In the hill areas north of Sylhet and Mymensingh there are some small localized area which are underlain by unconsolidated sands and boulder beds of the Dihing formation which is Pliocene to Pleistocene in age.

2.3 Hydrology of Bangladesh

The most significant feature of the hydrology of Bangladesh is the annual cycle of flooding and drainage. About 50 percent of the land area is lying between zero and twenty five feet above mean sea level with high concentration of flood discharge from the 600,000 square mile catchment area of the Ganges, Bhramaputra and Meghna rivers basin. Between 20 to 35 percent of the country area is flooded in each year. In 1998 flood 80% of the land went under flood for about 3 months. In the year 2000 the higher elevated district i.e Jessore, Satkhira, Jhenaidah, Chuadanga, Meherpur, Kustia, Nawabganj etc. also experienced flooding.

The country has a tropical monsoon climate which has three main seasons; pre monsoon, monsoon and dry. The annual rainfall varies from 50 inches to 200 inches. Flooding are generally caused by rainwater falling locally which is unable to drain off the land because of the simultaneously high river levels. Only a very small proportion of the country side is subject to active soil deposition from the silt laden river water. In dry season tidal flows are encountered in the rivers almost as far north as the Ganges-Jamuna confluence in the west, and Sylhet in the east. Another significant feature of the hydrology of Bangladesh, which can particularly affect land transportation, is the lateral movement of the main river courses and channel.

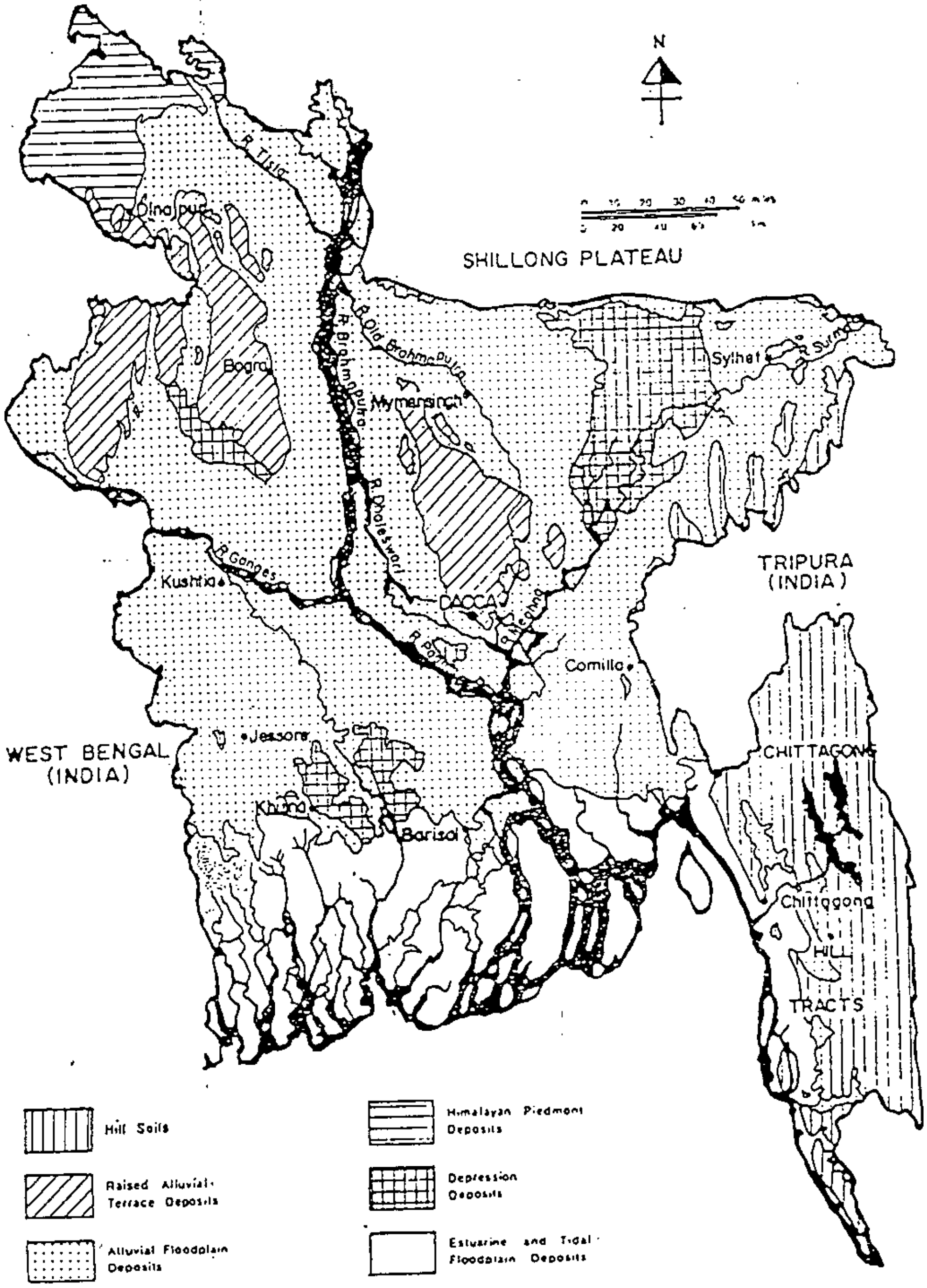


Figure 2.1 Map of Bangladesh showing Physio-graphical Divisions according to Geological Formation (T. Hunt, 1976)

2.4 The Properties of Dhaka Clay

Serajuddin and Ahmed (1967) investigated and furnished data on physical and engineering properties of soils of Bangladesh up to depth of 20 feet. From those data, it is found that Dhaka soil consists of mainly clay or a mixture of clay and silt. Soil classification by unified system was mainly CL and CH type. Most of their samples of Dhaka soil fall above A-line of Cassagrande plasticity chart. Similar sort of information on Dhaka clay can be found from the soil profile data reported by Eusufzai (1967).

Bore charts of Dhaka city show that the top clay layer exists with thickness of 25 to 30 feet above sand layer. Index and other properties of the clay after Uddin (1990) are summarized in Table 2.1, which were found from different soil reports such as sub-soil investigation of proposed Bishal Bhaban in Maghbazar, soil testing reports of a number of proposed residential buildings in Ghulsan, and sub-soil investigation reports of housing and building research institute for proposed buildings in Mirpur (Uddin 1990).

According to the reports furnished by Safiullah (1977), Islam (1980), Ameen (1985) and Uddin (1990) Dhaka soil deposits are slightly to heavily overconsolidated or at least have an overconsolidated crustal stratum. Their data yielded straight lines with the plot of undrained shear strength ratio versus OCR. Ameen (1985) made a study on Dhaka soil collecting sample from the campus of Bangladesh University of Engineering and Technology, Dhaka from a depth of five feet from ground level. He reported the values of different geotechnical parameters of the Dhaka clay which are presented in Table 2.2. Dhaka clay possesses a considerable amount of swelling index, which is a measure of volume increase of soil due to immersion under water. Undrained shear strength versus water content had been plotted by Ameen (1985) with data from Safiullah (1977), Kabir (1978), and Islam (1980) for naturally occuring Dhaka clay, which is shown in Figure 2.2. The plots show that no definite relationship existed between the water content and shear strength. It is known that, at the same water content, undrained shear strength may be different at normally loaded and preloaded state. Thus it shows that Dhaka soil deposits are over consolidated to various degree.

Table 2.1 Index and other properties of Dhaka Clay (Uddin, 1990)

Sl. No	Properties	Generally Vary from
1	Liquid limit	39% to 50%
2	Plastic limit	18% to 25%
3	Plasticity index	18 to 29
4	Clay Content (less than 2 micron)	15% to 35%
5	Sand content	0% to 11%
6	Silt content	65% to 85%
7	Water content	17% to 37%
8	Co-efficient of consolidation	0.14 to 0.34
9	Soil classification under unified classification system	CL and CH

Table 2.2 Geotechnical Parameter of Dhaka Clay (Ameen, 1985)

Sl. No	Parameter	Value
1	Isotropic compression index [Cc (iso)]	0.27
2	Isotropic swelling index	0.04 ±0.01
3	Ko compression index [Cc (Ko)]	0.27
4	Ko swelling index	0.02 ±0.005
5	Ko value at normally loaded state	0.46
6	Ko value at preloaded state (OCR=1.2 to 12)	0.512 to 3.23
7	S (S_u/σ_v') for Ko consolidated soil	0.19
8	S (S_u/σ_v') for isotropically consolidated soil	0.30

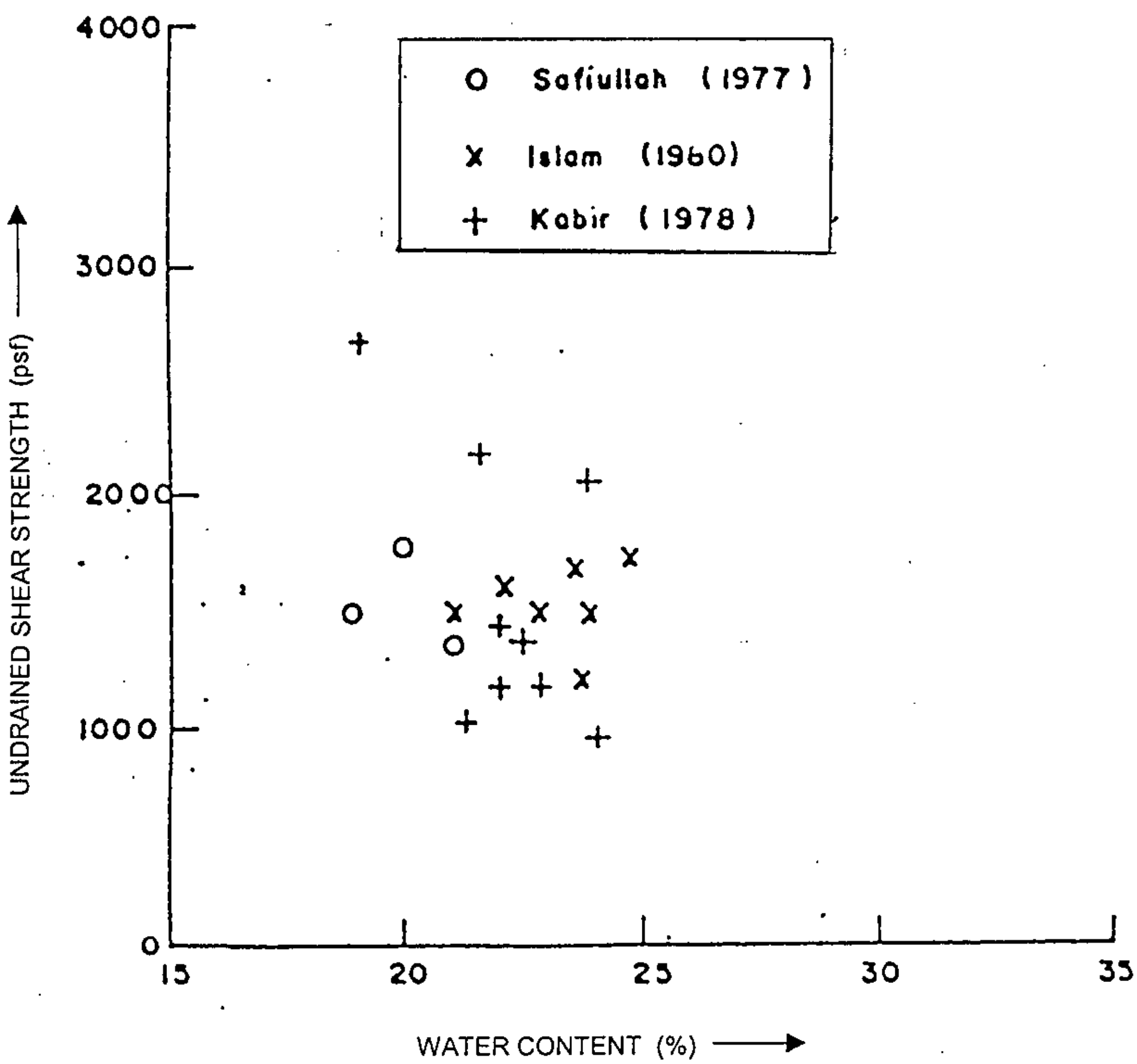


Figure 2.2 Relationship between Undrained Shear Strength and Water Content for Natural Occurring Dhaka Clay (Ameen, 1985)

2.5 Road Pavement

The word 'Pavement' refers to a hard surface of flat stones or a mixture of aggregate, sand and soil or without soil to support the load of traffic plying on it as well as to facilitate the movement of traffic. The pavement consists of a few layers of pavement materials over a prepared soil subgrade to serve as a carriage way. The pavement carries the traffic loads and transfers the load through a wider area on the soil subgrade below. The surface of the pavement should be stable and non-yielding under heaviest road traffic. This property of the pavement makes the road traffic to move with least possible rolling resistance. The pavement also keeps its temporary elastic deformation within the permissible limits so that it can sustain a large number of repeated load applications during the design life.

2.5.1 Types of pavement

For design purpose, pavements fall under two categories, namely Rigid and Flexible.

Rigid pavement as the name implies is a cement concrete slab acting as wearing surface of the road. Rigid pavement contains high rigidity and modulus of elasticity and is able to bridge over any localized failures. It provides a good riding surface and lasts long with very little maintenance. All other types of pavement other than rigid can traditionally be classified as **Flexible pavement**. The widely accepted definition of a flexible pavement is that, ' a flexible pavement is a uniform or composite structure that maintains an intimate contact with and distributes loads to the subgrade by mechanical interlocking of aggregates, particle friction and cohesion for developing stability'. Thus, the classical flexible pavement includes primarily those pavements which are composed of a series of granular layers (bituminous or non bituminous) with a relatively thin layer of wearing surface made of high quality materials. A flexible pavement may be composed of a single layer or a series of layers depending mainly on traffic volume. Sometimes, multiple bituminous layers are provided for heavy duty pavements.

2.5.2 Pavement structure

Followings are the main component layers of Flexible pavement:

Sub-grade

Sub-base

Base course

Wearing course or surface course.

The general structure of a flexible pavement is given below:

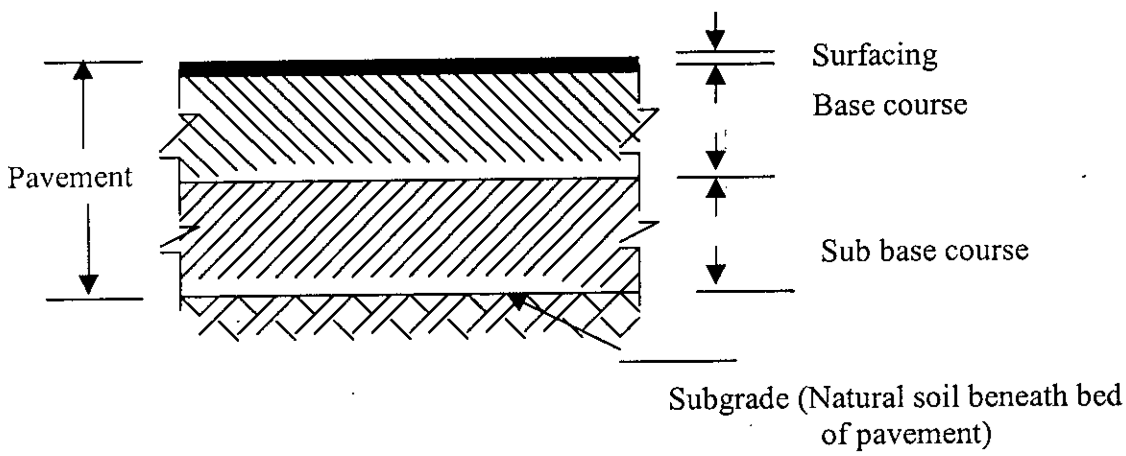


Figure 2.3. Structure of Flexible pavement

Pavement structure may be having all these elements or some of them may have been eliminated and some new have been added depending upon the actual site and traffic conditions. Improved subgrade is provided underneath Sub base course where subgrade soil strength is poor.

2.6 Road Subgrade

The upper layer of the Embankment or natural ground whether in cut or fill is termed as sub-grade. Load from the traffic is ultimately distributed on subgrade through other component layers. Therefore, the strength of the subgrade is a basic factor in determining the thickness of pavement. Sometimes, selected or imported materials is used for subgrade preparation either mixing with the available natural or embankment materials.

The strength of road subgrade is commonly assessed in terms of the California Bearing Ratio (CBR) of the subgrade soil and this is dependent on the type of soil, its density and its moisture content. Due to the heterogeneous nature of the floodplain soils, changes in subgrade California Bearing Ratio (CBR) values are frequent and large. The soil type is largely determined by the location of the road, but where the soils within the possible corridor for the road vary significantly in strength from place to place, it is clearly desirable to locate the pavement on the stronger soils, if this does not conflict with other constraints. The density of the subgrade soil can be controlled by compaction at suitable moisture content at the time of construction. The moisture content of the subgrade soil is governed by the local climate and the depth of the ground water table below the road surface.

Current design methods for flexible pavements are usually based on CBR values obtained from soil samples compacted at optimum moisture content to 95% modified AASHTO and soaked for a period of 4 days. In some cases this soaking procedure may be an unrealistic since road elevations can usually be at least 3 ft above normal high flood level, inundation may rarely occur and adequate cross falls should restrict the surface penetration of rain water.

In the tropics, subgrade moisture conditions under impermeable road pavements can be classified into the following three main categories (Road Note 31):

I) *Subgrade where the water table is sufficiently close to the ground surface to control subgrade moisture content.* This type of subgrade soil governs the depth below the road surface at which a water table becomes the dominant influence on the subgrade moisture content. In non plastic soils the water table dominates the subgrade moisture content when it rises to within 0.9m of the road surface, in sandy clay ($I_p \leq 20$ percent) the water table dominates when it rises to within 3 m of the road surface, and in heavy clays ($I_p \geq 40$ percent) the water table dominates when it tries to within 7m of the road surface. In addition to areas where the water table is maintained by rainfall, this category includes coastal strips and flood plains where the water table is maintained by the sea, by lake or by a river.

II) *Subgrade with deep water tables and where rainfall is sufficient to produce significant seasonal changes in moisture conditions under the road.* These conditions occur where rainfall exceeds evapo transpiration for at least two months of the year. The rainfall in such areas is usually greater than 250 mm per year is often seasonal.

III) *Subgrade in areas with no permanent water table near the ground surface and where the climate is arid throughout the year.* Such areas have an annual rainfall of 250mm or less.

Considering road pavement virtually impermeable moisture content of subgrade changes only with the change of ground water level. If permeable base and sub base materials are used rain water shed from the road surface can also penetrate to the subgrade and may saturate it. In these cases the strength of subgrade with moisture condition in category (I) and category (II) areas should be assessed on the basis of saturated CBR i.e. Soaked CBR. Subgrade with moisture conditions in category (III) are unlikely to become saturated when covered by a permeable base and sub base and the subgrade moisture content; in such situations unsoaked CBR value can be taken for estimating the subgrade strength. Table 2.3 shows the estimated minimum design subgrade CBR values at different depth of water table from road formation level (Road Note 31).

2.7 Determining the Subgrade Strength

Having estimated the ultimate subgrade moisture content it is then possible to determine the appropriate design CBR value at the specified density for the subgrade (Road Note 31). During construction it is recommended that all subgrade should be compacted to a relative density of at least 95% of the MDD achieved in Standard / Modified Proctor Density test. Compaction not only improves the subgrade bearing strength but also reduces permeability and subsequent compaction by traffic. As a first step it is necessary to determine the compaction properties of the subgrade soil by carrying out a standard laboratory compaction test. Samples of the subgrade soil at the estimated subgrade moisture content can then be compacted in CBR moulds to the specified density and penetrated to determine the design CBR value. This value is then used to determine the required pavement thickness from the design chart given in Figure 2. 4. If the samples of cohesive soil are compacted at moisture content equal to or greater than the optimum moisture content, they should be left sealed for 24 hours before being penetrated so that excess pore water pressures induced during compaction are dissipated.

Alternatively, a more complete picture of the relationship between density, moisture content and CBR for the subgrade soil can be obtained by compacting the soil at several moisture contents and at least two levels of compaction, measuring the CBR of each sample. The design CBR is then obtained by interpolation. This method is preferable since it enables an estimate to be made of the subgrade CBR at different densities and thus indicates the value of achieving the specified density in the sub grade. Figure 2.5 shows a typical dry density / moisture content / CBR relationship for sandy clay soil that was obtained by compacting samples at five moisture contents to three levels of compaction i.e. 4.5 kg, 2.5 kg and an intermediate level of compaction. By interpolation a design subgrade CBR of about 12 percent would be obtained in this case if a relative density of 95% of British standard maximum dry density was specified and the ultimate subgrade moisture content was estimated at 20 percent.

If saturated subgrade conditions are anticipated because of the use of permeable base and sub base materials in areas with more than 250mm annual rainfall, the compacted samples for the CBR test should be saturated by immersion in water for 4 days before being tested. In all other cases when CBR is determined by direct measurement, the CBR samples should not be immersed since this result in over design.

In situ CBR measurements of subgrade soils are not recommended because of the difficulty of the ensuring that moisture and the density conditions at the time of the test are representative of those expected under the completed pavement.

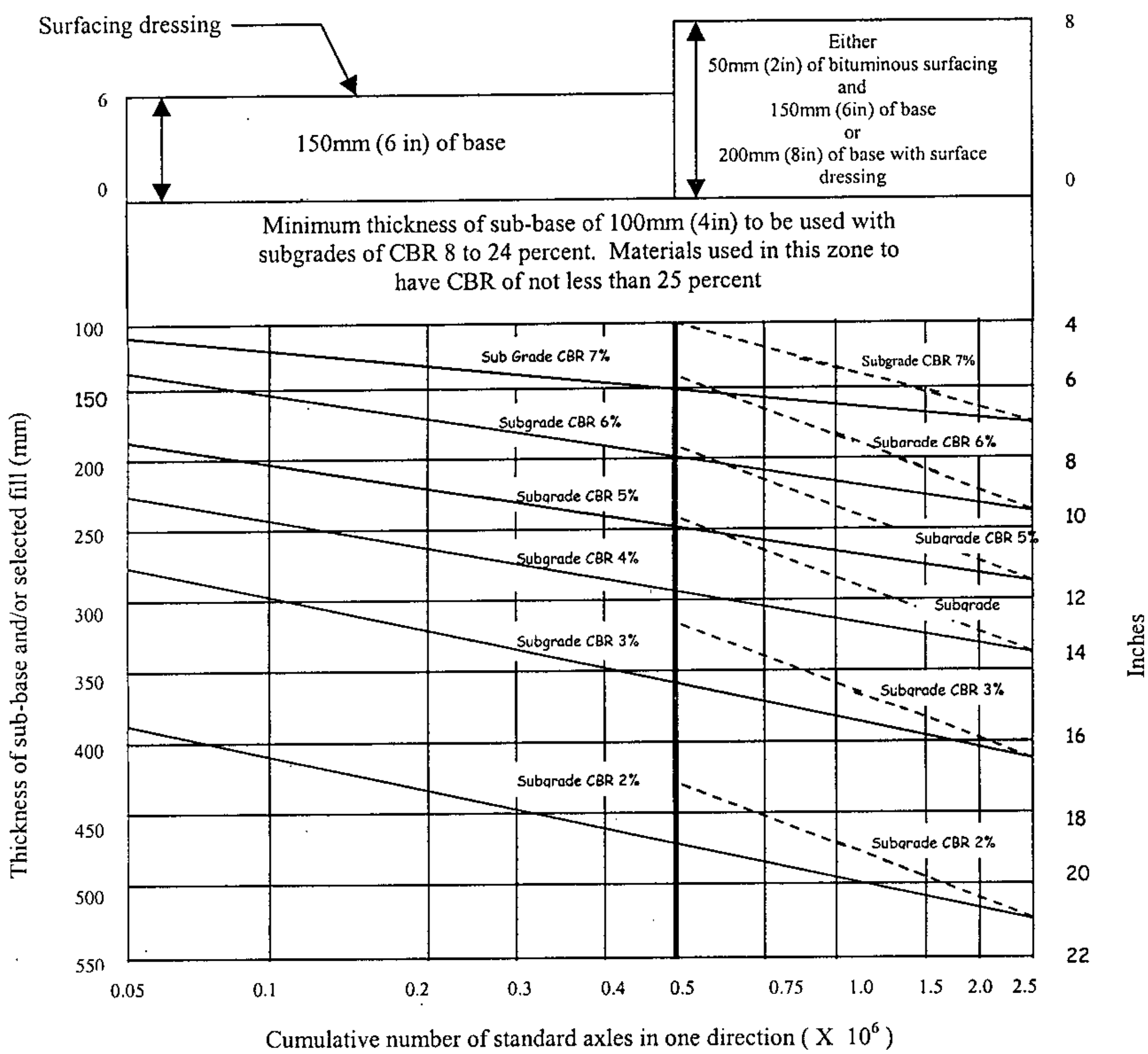


Figure 2.4 Pavement Design Chart Based on Subgrade CBR Value (Road Note 31)

Table 2.3 Estimated Minimum Subgrade CBR Value at Different Depth of Water Table (Road Note 31)

Estimated minimum design subgrade CBR value under paved roads for subgrade compacted to 95 percent of British Standard maximum dry density :

Depth of water table from formation level	Minimum CBR (percent)					
	Non plastic sand	Sandy clay PI=10	Sandy clay PI=20	Silty clay PI=30	Heavy clay PI≥ 40	Silt
0.6m (2 ft)	8	5	4	3	2	1
1.0m (3.3 ft)	25	6	5	4	3	2
1.5m (4.9ft)	25	8	6	5	3	Laboratory CBR is required
2.0m (6.5ft)	25	8	7	5	3	
2.5m (8.2ft)	25	8	8	6	4	
3.0m (9.8 ft)	25	25	8	7	4	
3.5m (11.5ft)	25	25	8	8	4	
5.0m (16.4 ft)	25	25	8	8	5	
7.0m (23 ft or more)	25	25	8	8	7	

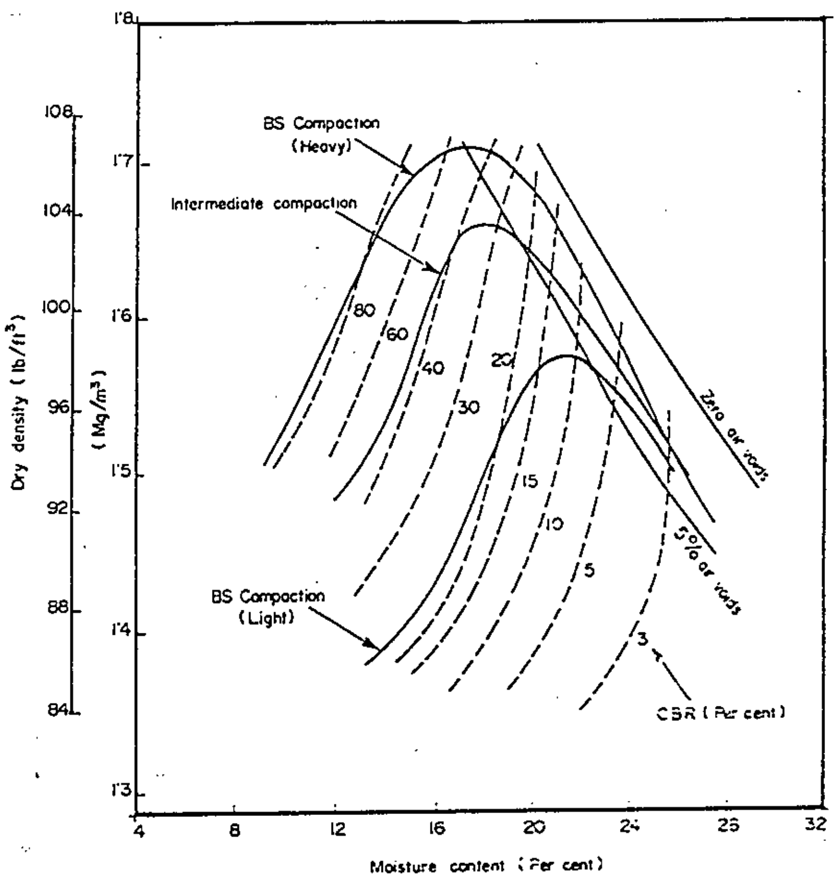


Figure 2.5 Dry Density-Moisture Content-CBR Relationships (Road Note 31)

2.8 Determining the Pavement Thickness

The thickness of pavement will vary due to CBR value of subgrade. CBR value also varies due to change of compaction. If the same material is compacted to 100% of MDD instead of 95%, then CBR value will be more to provide lesser pavement thickness as well as more longevity.

The subgrade CBR and the cumulative number of standard axles expected to be carried by the pavement during its design life having been determined, the required pavement thickness can be obtained from the design chart given in Fig 2.4 (Road note 31). This chart has been prepared on the basis that a standard base thickness of 150mm with a variable thickness of sub base to allow for different subgrade strength, is the most economical design for flexible pavements. The thickness of sub base required is governed by the CBR of subgrade and the cumulative number of standard axles to be carried. The appropriate thickness can be obtained from the design chart in Figure 2.4. If the CBR of subgrade is 25 percent or more, no sub base is required. If it is less than 25 percent, a thickness of 100mm of sub base is required for CBR of 8 to 24 percent and a greater thickness for CBR less than 8 percent. If the subgrade is composed of a swelling clay the thickness of the pavement is not governed by the strength of the sub grade, but the need to reduce the volume change resulting from the seasonal changes in the subgrade moisture content. The sub base material, which is normally be a naturally occurring gravel or gravel-sand-clay, should have a CBR of 25 percent or more at the density and moisture content expected in the field. The base materials constitute crushed stone or gravel or open textured macadam with less than 15 percent of materials finer than 0.075 mm ((No. 200 sieve) or bitumen bound or cemented materials. Except for dense bitumen bound materials, the differences in the load spreading properties of these materials are too small and nominal base thickness for those materials is 150mm (6inch) and for bitumen bound materials nominal thickness of base may be reduced to 125 mm (5 inch). Table 2.4 shows the estimated minimum soaked CBR value for different type of soil sample in Bangladesh (T. Hunt, 1976). Serajuddin and Azmal (1991) suggested minimum CBR values for subgrade of AASHO designated soil sub groups A-4, A-6 and A-7 with more than 35 percent passing No. 200 (0.075mm) sieve, which is presented in Table 2.5.

Table 2.4 The estimated minimum soaked CBR value from a number of samples compacted at 95% modified AASHTO (T. Hunt, 1976):

Soil Unit	Topographic Unit	Predominant Soil Type	Recommended Preliminary Design Subgrade CBR
Alluvial floodplain deposits	Ridges, and Shallow basins	Silt with trace sand and clay ML-CL	6
As above but including depression deposits	Lakes, Channel fills and deeper basins	Silty clay ML-CL	3
Himalayan piedmont deposits	Ridges, plains and shallow basins	Sand with silt SM-ML	10
Estuarine & tidal flood plain deposits	Ridges and basins	Silt and silty clay ML-CL	3
Raised alluvial terrace deposits	Terrace and Valleys	Silty clay and Clay CL-CH	4
Hill soil	Hills and valleys	Extremely variable	Assume 10 as an average value

Table 2.5 Suggested Minimum Subgrade CBR Values (Serajuddin and Azmal, 1991)

AASHO Soil Sub-Groups	Reqd. Dry Desity, Kg/Cum	Reqd. Plasticity Index (%)	Reqd. Relative Compaction (%)	Unsoaked CBR (%)	Soaked CBR (%)
A-4, A-6	≥ 1800	≤ 25	≥ 95	25	10
A-7	1700 to 1800	≤ 25	90-95	12	4

2.9 California Bearing Ratio (CBR) Test and Its Development

The California Bearing Ration (CBR) method of road design was first used by the California Division of highways (Yoder and Witczak, 1975) as a result of surveys made during the years 1928 and 1929 (12). The investigations brought out that the principal types of pavement failure were: I) lateral displacement of the subgrade material as a result of pavement absorbing water, II) differential settlement of materials underneath the pavement and II) the excessive deflection of the materials under the pavement. To predict the behavior of paving materials, the CBR test was devised in 1929. Tests were performed on a large number of typical crusher-run materials, which were considered representative of base course material. The averages of these test result, then was designated CBR 100 percent.

At the beginning of the second world war, the Corps of Engineers made an extensive survey of the different methods of flexible pavement design (Yoder and Witczak, 1975). As a result of this investigation, the CBR method was adopted. The thickness of the different elements comprising a pavement is determined by the CBR values. This method has some advantages and disadvantages. One of the advantages is the simplicity with which the design test can be performed, but a disadvantage of the procedure is that the test is empirical and, therefore, the design is based upon correlation. However, much research has been done which permits extrapolation of the data from one wheel load gear configuration to another.

Samples of soil from failed and from satisfactory pavements were tested and design curves were developed as indicated in figure 2.6 (Yoder and Witczak, 1975). Curve A on this figure was developed on the basis of 1942 practice and was considered to be adequate for average traffic conditions, whereas curve B represents values developed from the original survey made by California Division of Highways.

The California Bearing Ratio (CBR) test is a penetration test. The CBR is expressed as a percentage of the penetration resistance to that of a standard value for crushed stone.

$$CBR = \frac{\text{Load Carried by Specimen for a Specific Penetration}}{\text{Standard Load Carried by Crushed Stone for the Same Penetration}} \times 100$$

Since the CBR is a percent of a standard load, it is possible in some cases to measure CBR values in excess of 100 percent. Generally, the CBR at 2.5 mm (0.1 inch) penetration is used for design purposes. However, if the bearing ratio at 5.0 mm (0.2 inch) penetration is greater in that case this value is used.

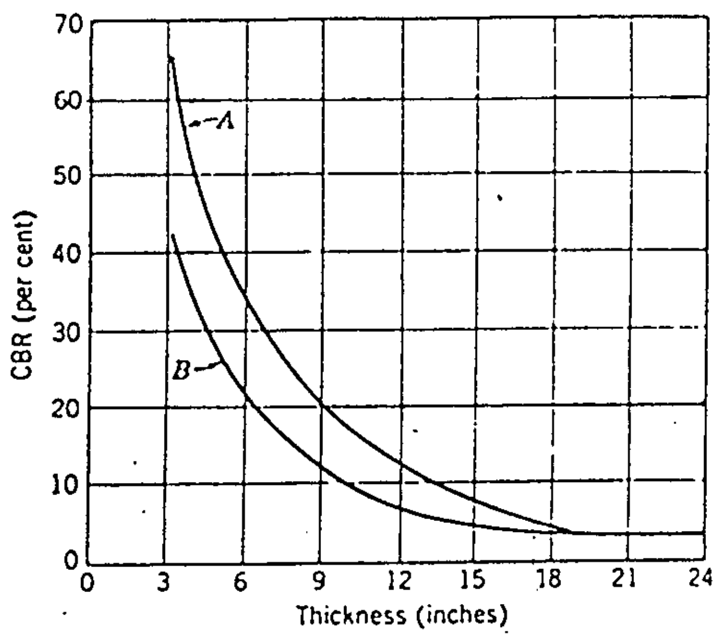


Figure 2.6 Original CBR Curves for Pavement Thickness (Yoder and Witczak, 1975)

2.10 Shear Strength and Unconfined Compression Test

When soil is loaded, shearing stresses are induced in it. When the shearing stresses reach a limiting value, shear deformation takes place, leading to the failure of the soil mass. The failure may be in the form of sinking of a footing, or movement of a wedge of soil behind a retaining wall forcing it to move out, or the slide in an earth embankment. The shear strength of soil is the resistance to deformation by continuous shear displacement of soil particles or on masses upon the action of a shear stress. The failure conditions for a soil may be expressed in terms of limiting shear stress, called shear strength, or as a function of the principal stresses. The two forms of the failure conditions are often but not always interchangeable. All stability analysis in soil mechanics involves a basic knowledge of the shearing properties and shearing resistance of the soil. The shear strength is the most difficult to comprehend and one of the most important of the soil characteristics.

The shearing resistance of soil is constituted basically of the following components:

(I) the structural resistance to displacement of the soil because of the interlocking of the particles,

(II) the frictional resistance to translocation between the individual soil particles at their contact points, and

(III) cohesion or adhesion between the surface of the soil particles.

The shear strength in cohesionless soil results from intergranular friction alone, while in all other soils it results both from internal friction as well as cohesion. However, plastic undrained clay possesses very negligible internal friction.

The measurement of shear strength of soil involves certain test observations at failure with the help of which the failure envelope or strength envelope can be plotted. Shearing resistance can be determined in the laboratory by different methods like; Unconfined compression test, Direct shear test, Triaxial shear test, Vane shear test etc.

Out of these methods Unconfined Compression test is mostly used because of simplicity and less time required for the test. In this method due to absence of confining pressure, the cylindrical specimen of soil 71.1 mm (2.8 inch) height and 35.6 mm (1.4 inch) diameter is subjected to increase in major principal stress till the specimen fails due to shearing along a critical plane of failure. The apparatus consists of a small load frame fitted with a proving ring to measure the vertical stress applied to the soil specimen. The deformation of the sample is measured with the help of a separate dial gauge. During the test, load versus deformation readings are taken and a graph is plotted. When a brittle failure occurs, the proving ring dial indicates a definite maximum load, which drops rapidly with the further increase on strain. In the plastic failure, no definite maximum load is indicated. In such a case, the load corresponding to 20% strain is arbitrarily taken as the failure load.

2.11 Theory of Compaction

Compaction is the process of packing the soil particles closer together by external forces with addition of water with the view to increase density and reduce air void. The main purpose of a theory of compaction is to explain the effect of moisture on the compacted dry density.

2.11.1 Proctor's Theory

Proctor published the theory of soil compaction in 1933 and explained the compaction phenomenon as the effect of two factors, namely, a) Capillarity and b) Lubrication. Of the two factors he put more emphasis on the concept of lubrication.

Capillarity, the first effect of moisture content was stated by him as "The moisture contained in a very dry soil surrounds each particle as a thin film held in place by the forces of surface tension. Where the films come in contact, the capillary force caused by the surface tension of the joined films draws the particles firmly together, causing a high frictional resistance between them'.

The second one is the effect of lubrication. When water is added to the dry soil, each particle is covered with a thin film of adsorbed water. With the addition of more water the thickness of the water film on a particle increases. Increased thickness of the water films reduces the interparticle friction, and also reduces the shearing strength, which permits the particles to slide past one another more easily. This process is called lubrication. The reduction in shearing strength causes the compactive dry density to increase because with the application of loads soil particles slide over one another and arrange themselves according to their sizes in a more stable position giving a denser compacted structure. Up to a certain limit, water replaces the air voids in the soil particles, but there is a limiting amount of water content beyond which water occupies the space, which could be filled up by soil particles. Due to compaction pressure and the effect of lubrication, water forces into these particles, therefore, unit weight of the compacted specimen decreases when water is increased above the optimum point.

2.11.2 Lambe's Theory

Lambe in 1958 published the physico-chemical theory of soil compaction. He attempted to explain the shape of the compaction curve in terms of surface chemical theories. When water is added to a dry soil, the small amount of water gives a very high concentration of electrolyte, which depresses the double layer of the soil colloids. "The double layer depression reduces the interparticle repulsion, thereby causing a tendency toward flocculation of the colloids. This results in low density".

An increase of the moulding water content to optimum results in expansion of the double layer around the soil particles and also reduction in the electrolyte concentration. This reduces the degree of flocculation, which permits a more orderly arrangement of particles and a higher density. Lambe used the term 'lubrication' to describe properly the effect of molding water content of the sliding phenomenon of soil particles to attain the high density of soils. When moisture content is increased beyond the optimum, double layer expands to a further extent. This results in decrease of soil strength, because double layer of the two adjacent colloids reduces the net attractive forces between them. Soil particles exists in a more orderly arrangement at a moisture content higher than the optimum, but the density become low because of the 'dilution of the concentration of soil particles per volume'.

2.12 Moisture, Density and Strength of Soil

Proctor in 1933 was perhaps the first to show the relationships between compaction energy, moisture contents, and density of soils. These results were later confirmed by many investigators. Foster (1953) investigated into the compacted characteristics of a silty clay of liquid limit 33 and plasticity index 13. From both laboratory and field tests, it was shown that for a certain moisture content, shear strength as measured by CBR increased with the increase of density up to a certain limit. Beyond this limit shear strength decreased with the increase in density. 'This behaviour has been noted in a wide range of soil types, but is most prevalent in silty materials'. He also observed that generally, this decrease in strength occurred with the water content-density condition beyond the line of optimum moisture content.

The decrease of strength was explained by him to be the result of development of pressure in void phase of the soil structure. His opinion was contradicted by Barber and according to him reduction in strength was due to the differences in structure produced by compaction rather than by the development of pore pressure. Foster observed from the results of investigations conducted by the U.S. Waterways Experiment Station that the variation of density with unconfined compressive strength was similar to that of CBR. Triaxial tests showed the same behaviour, only when the deviator stress at a low percentage of strain, was used.

Turnbull and Foster (1956) studied the relationship between strength, water content and density of cohesive soils. They observed that increases in compactive effort produced increase in density and strength (measured by CBR) and a decreases in optimum moisture content. But after a certain limit, a high degree of compaction did not necessarily lead to high strength. They showed that shear strength (measured by CBR) of compacted cohesive soils both in the unsoaked and soaked conditions might decrease beyond a certain limit of density. Curves of unconfined compression strength versus moisture content and density showed similar results as with CBR.

Water content of the compacted soil determined the strength and density of soils. For clay soil, moisture content on the dry side of optimum, produced high density and strength and on the wet side of optimum, a high compactive effort produced lowering of the shear strength. Moreover, a clay soil when compacted on the wet side of optimum by a given type and amount of compactive effort would lose less strength on coming in contact with water than if it were to be compacted on the dry side of optimum. Therefore, they concluded that cohesive soils could be stabilized to yield a given strength by proper consideration of moisture content control during compaction to attain the specified density and strength.

In 1958 Lambe made the following observations of the compacted clay strength:

A) Increase of compactive effort on the dry side of optimum moisture content increases shear strength.

B) Increase of compactive effort of the wet side of optimum may increase or decrease the strength.

C) 'For same compactive effort and same compacted density, dry-side compaction gives a higher strength than does wet-side compaction'. Soil samples of equal density when soaked and the volume is kept constant, the dry-side sample gives higher strength than the wet side one. If free swelling is permitted, the wet-side sample gives higher strength.

According to him, these phenomena could be explained by the compaction theory 'clays compacted dry of optimum usually have negative pore water pressure.

These negative pressures result in higher intergranular stresses and therefore, higher strength. Wet-side compaction of a soil attains better saturation, which gives more compressible structure and higher pore water pressures are developed during shear. This built up pore pressure accounts for part of the lowering of strength'.

Langfelder and Nivargikar (1967) made a review of forty research publications concerning the factors affecting the engineering behavior of compacted soils (cohesive and non cohesive) published up to 1966. Some of the conclusions on shear strength and compressibility of compacted soils made by them are enumerated below:

1) 'Cohesive soils are found to have differences in shear strength that are caused by differences in soil structure. A flocculated soil structure is more rigid and produces smaller initial pore water pressures during shear than the same soil with a dispersed soil structure. This leads to increased strengths particularly at low strains. The soil structure that is produced by compaction is governed by the soil type, the molding water content and the compaction method'.
2) 'Based on effective stress theory, it can be shown that the initial effective stress may either increase or decrease with increasing water content along a compaction curve on the dry side of optimum, but that the effective stress will always decrease with increasing water content along the compaction curve on the wet side of optimum'.
3) 'The as-compacted shear strength of a cohesive soil for a constant dry density will always exhibit a decrease in shear strength with an increase in water content'.
4) 'The as-compacted shear strength of a cohesive soil, for a constant water content, will exhibit an increase in shear strength for all water contents with an increase in dry density only when the strength is defined at large strains. At low strain levels the strength may increase or decrease with dry density depending on the water content and the mould of compaction'
5) 'For soaked conditions, the resulting shear strength is determined by combined effect of swelling during soaking, initial water content, and as-compacted soil

structure. For CBR-type tests that allow swelling to take place it appears that the maximum soaked shear strength occurs at approximately the as-compacted optimum water content'.

6) 'The strength of a compacted cohesive soil may change significantly with time after compaction because of thixotropic effects'.
7) ' Compressibility of compacted cohesive materials is influenced by the soil type, molding water content, as-compacted dry density, initial degree of saturation and compaction method'.

Serajuddin and Azmal (1991) conducted a study on the mostly occurring alluvial deposits of Bangladesh used as embankment fill and subgrade in national highways from two to three meter depth of borrow pit for necessary physical and geotechnical properties like; plasticity characteristics, particle size distribution, dry density- moisture content relation (by Standard and modified compactive efforts) and California Bearing Ratio (CBR).

The effect of variation of moulding water content on soaked CBR at modified and standard compactive efforts (Serajuddin and Azmal, 1991) for those types of soil is shown in Figure 2.7 (a). Figure 2.7 (b) shows the correlation between soaked CBR and soaked Unconfined Compressive strength. Relationship of both soaked and unsoaked CBR with dry densities is shown in figure 2.8.

The compaction and CBR test results after Serajuddin and Azmal (1991) for alluvial soil of Bangladesh of different regions indicate the following:

I) Compactibility of the soils is fair to good, specially when subjected to modified compaction.

II) Standard MDD and OMC of the soils vary from about 1390 kg /m^3 to 1970 kg/m^3 and about 11 to 27% respectively; and modified MDD and OMC vary from about 1590 kg/m3 to 2100 kg/m3 and about 8 to 23% respectively.

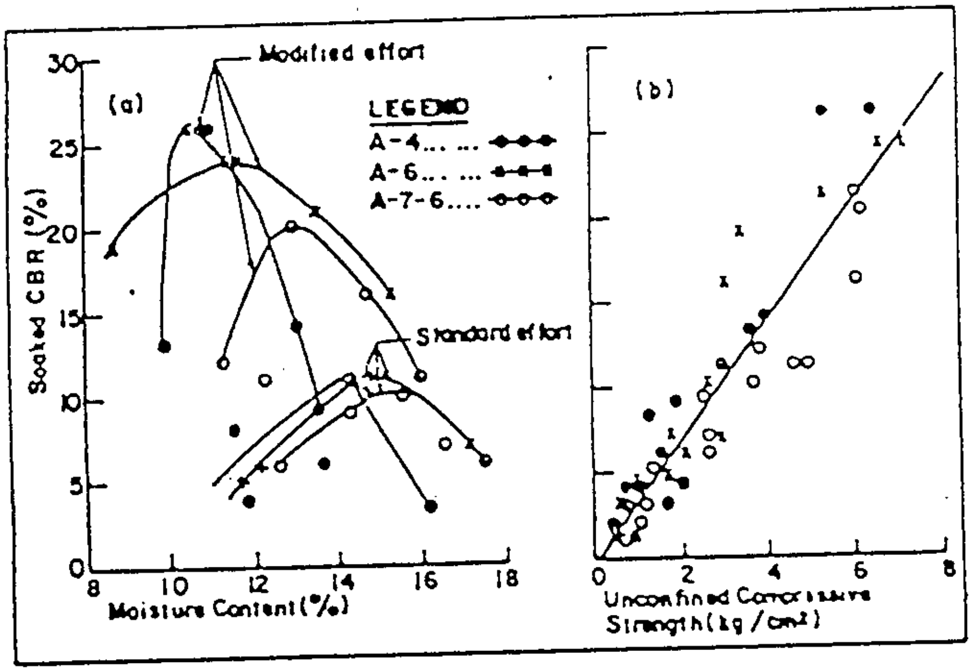


Figure 2.7 Showing (a) Effect of Variation of Moulding Water Content on Soaked CBR at Modified and Standard Compactive efforts and (b) Correlation between Soaked CBR and Soaked Unconfined Compressive Strength (Serajuddin and Azmal, 1991)

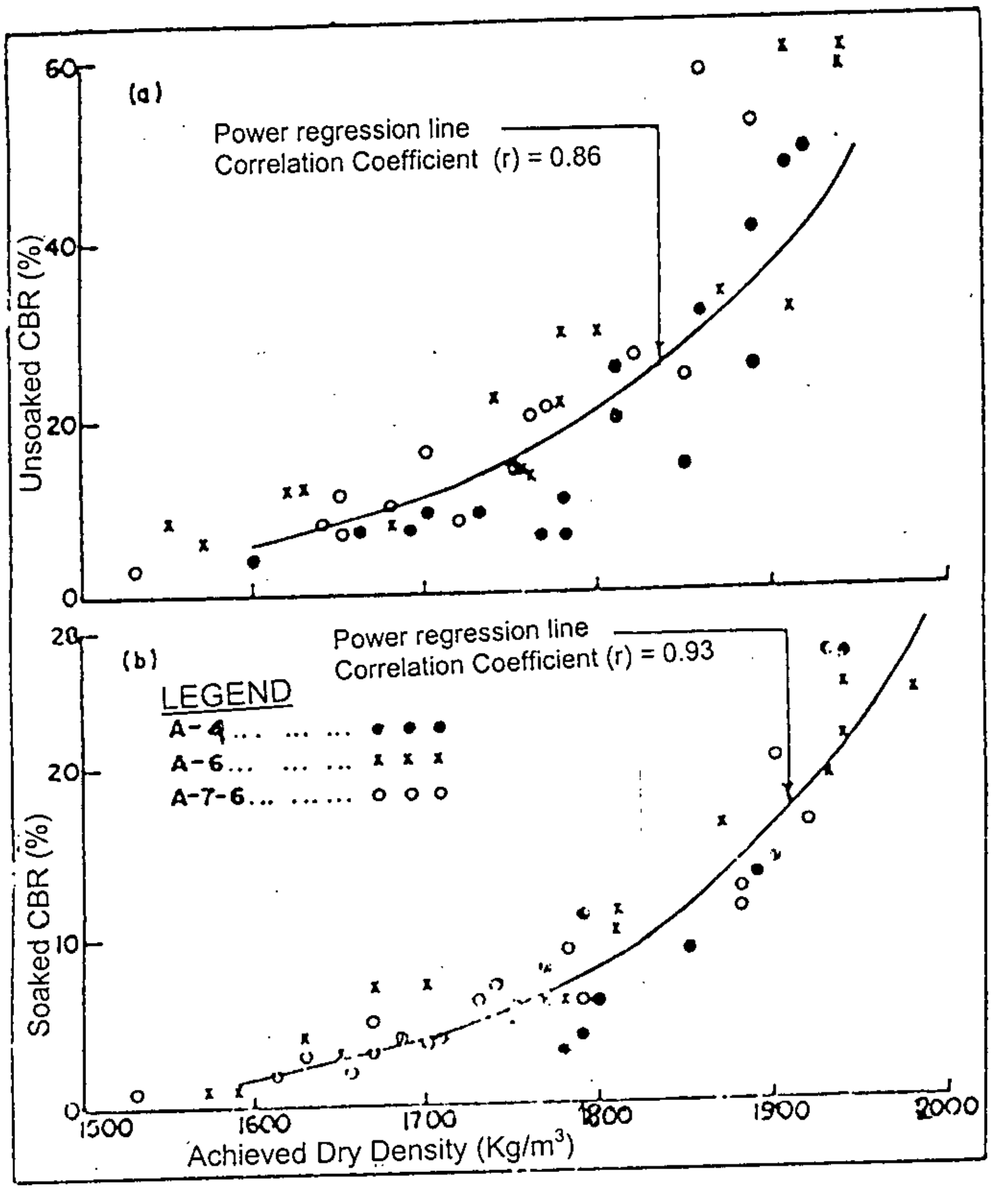


Figure 2.8 Relationship between CBR values and Dry Density (Serajuddin and Azmal, 1991)

III) Unsoaked CBR values are quite high (Fig 2.8) and on soaking for 4 days under 10 kg surcharge weight, the soaked CBR values become about 33 to 44 % of the unsoaked CBR value on the average.

IV) Swelling on soaking does not exceed about 2% (with few exception)

V) At equal level of compactive effort, CBR drops appreciably on variation of moulding moisture even within 2 to 3% wet and dry of OMC.

VI) The minimum CBR values are possible to be achieved at the dry densities and relative compactions (with respect modified MDD) as mentioned in Table 2.5. The climatic and hydrologic environments of Bangladesh are such that subgrade saturation is possible in rainy / flood seasons and consideration of soaked CBR values for subgrade would be appropriate in pavement design and construction. The relationship between CBR and achieved dry density of composite soil samples is shown in Figure 2.8.

Though the correlation between UCS and CBR (Serajuddin and Azmal, 1991) as shown in Figure 2.7 (b) would help engineers to make an estimate of CBR from UCS of similar silty and clayey soils of any area of the country in absence of CBR test data there are some limitations, which are as follows:

1. The investigation was conducted with alluvial soil from different parts of Bangladesh with wide range of varieties in particle sizes as well as index properties and correlation is made altogether. If the correlation could be made, dividing the soil into various sub groups it could be the best representative correlation.
2. These relationships are only for the alluvial soil of Bangladesh and will not be useful for other types of soil such as Dhaka clay, which also forms a substantial part of Bangladesh.
3. Soaked CBR value and soaked unconfined compressive strength (q_u) are correlated only. Relationship between unsoaked CBR and unconfined compressive strength (q_u) is not established. Relationship between other strength properties could also be developed.

CHAPTER 3

THE RESEARCH SCHEME

3.1 Introduction

The literature review reveals that relationship between CBR and strength characteristics exists for alluvium soil deposit of Bangladeshi soil. Some geotechnical properties related to road pavement also exits for alluvium soil. It appears that the strength behavior of soil depends on the water content, density and compaction energy. Different strength behavior at wet and dry side of optimum water content is also observed. From geological formation, Dhaka clay is under the group of Pleistocene soil and is overconsolidated.

The present research is aimed to evaluate the strength properties of compacted Dhaka clay through CBR (both soaked and unsoaked condition) and Unconfined Compression Strength test with varying water content so that the relationship among different strength parameters can help the engineers to prepare project proposal for road project within very short time in Modhupur clay region.

3.2 Soil Used

The soil sample for this experimental work was collected from the area of Pallabi Phase-II, Land Project of Eastern Housing Limited, which is just on the east of National Botanical Garden by digging an open cut. The used samples were from three to five feet depth. Location of the soil used is shown in Figure 3.1.

The colour of the soil was reddish brown and the consistency can be attributed as medium to stiff. The soil may be generally termed as the 'Older Alluvium' deposit or ' Pleistocene Madhupur deposit' at Dhaka or locally called 'Red Dhaka Clay'. The details of the physical and index properties of this soil are given in chapter 5.

The collected soil was air dried for several days in Geotechnical Engineering Laboratory of BUET. It was then broken down manually with the help of wooden hammer. The powdered sample was then sieved through No. 4 sieve and thus was made ready for different tests. Some times water was mixed with the sample or it was oven dried for specific requirements as specified by standard test methods.

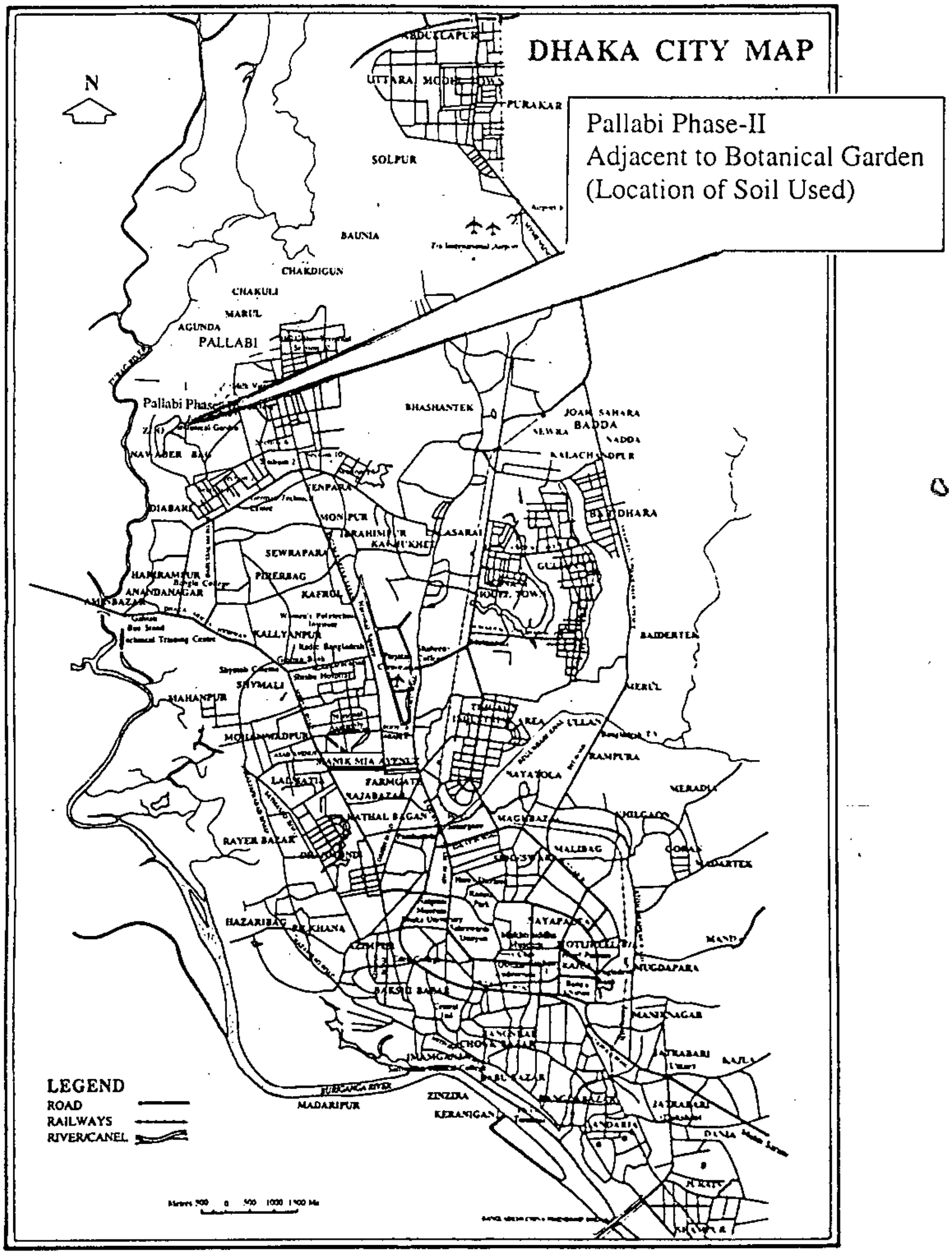


Figure 3.1 Location Map of Soil Used for the Study

3.3 Experimental Program

This experimental program for the collected Dhaka clay was carried out according to the following phases:

The soil sample was first undergone for index properties test i.e. Grain size analysis and Atterberg's limit tests were conducted. After necessary index tests Modified Proctor Density test was performed to obtain Maximum Dry Density and Optimum Moisture Content. Different water content for Modified Proctor Density test are 9.5%, 12%, 15.3%, 18.5%, 19.9%, 22.4% and 27.5%. These compacted soil samples were removed from the mould and used for Unconfined Compression Strength test. For each percentage of moulding water content one to two samples were prepared and tested.

At modified compaction energy California Bearing Ratio (CBR) test samples were prepared for both Soaked and Unsoaked condition at 9.5%, 12%, 15.3%, 18.5%, 19.9%, 22.4% and 27.5% water content. For soaking the samples were kept under water for 96 hours with 10 kg surcharge load. Swelling was measured during soaking.

Vane Shear test was also performed for the soaked CBR samples of different percentage of moulding water content using tore vane in the laboratory.

A flow chart for the experimental program is shown in Fig. 3.2. The details of the preparation of soil specimen and testing procedures are given in Chapter 4.

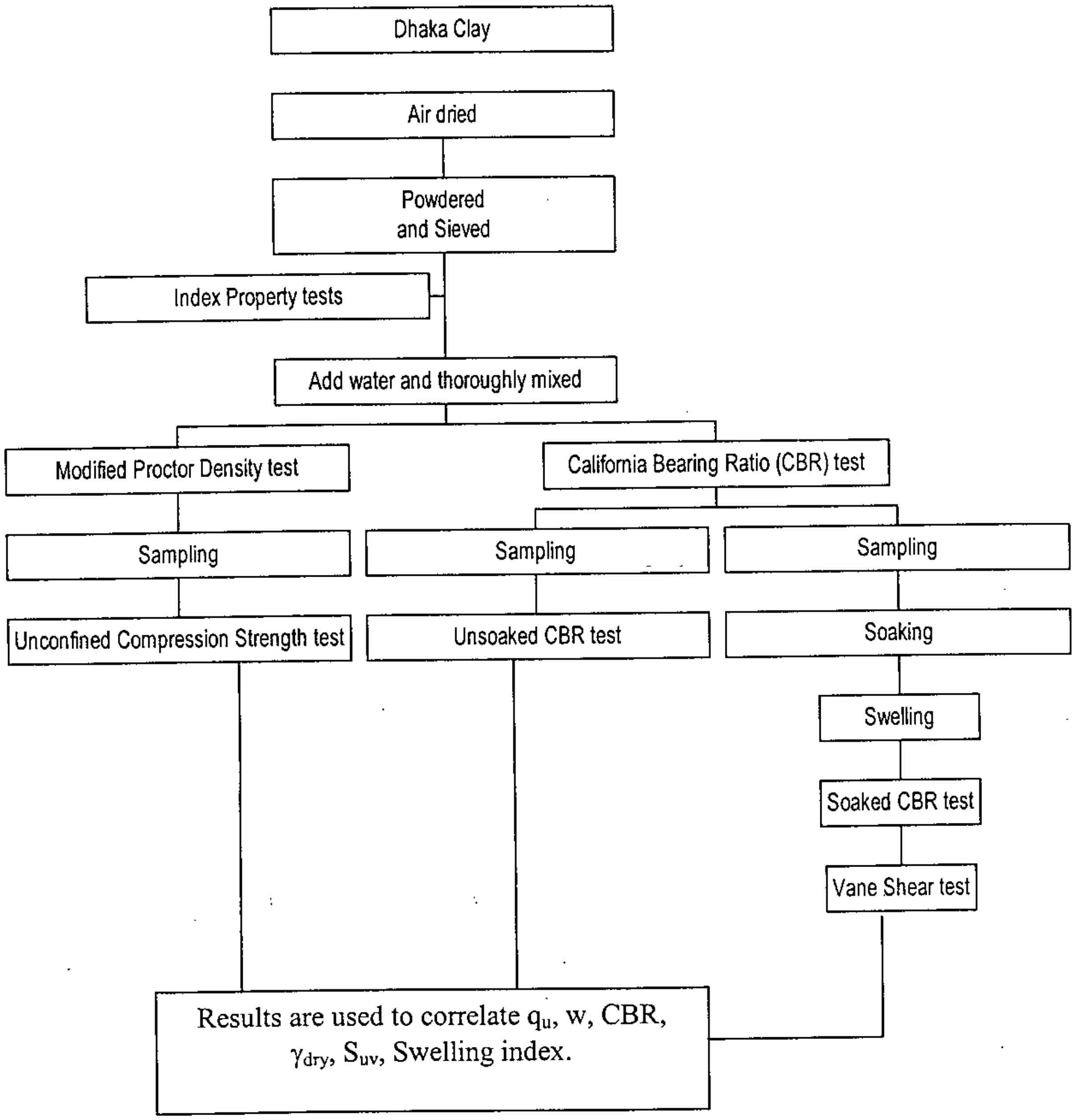


Figure 3.2 Flow Chart for Experimental Program

CHAPTER 4

LABORATORY INVESTIGATION

4.1 Introduction

This chapter describes, in details, the laboratory investigations made on the soil samples collected from the location described in Chapter 3. Flow chart in Fig 3.1 was followed for the laboratory investigations.

4.2 Preliminary Investigations for Physical and Index Properties

The soil samples collected were air-dried. Then lumps were broken carefully with a wooden hammer so as to avoid breakage of soil particle. The coarse particles were removed by screening with a No. 4 sieve. The following tests were performed to determine grain size and index properties of the soil:

I) Atterberg limits
II) Specific gravity
III) Grain size distribution

For the determination of physical and index properties of the soil, the procedures recommended by the American Society for Testing of Materials (ASTM) were used. ASTM standard D4318 was followed for liquid limit and plastic limit test, ASTM D854 for specific gravity test and ASTM D422 for grain size distribution by Hydrometer test. The values thus obtained are shown in Chapter 5.

4.3 Modified Proctor Density Test

The air dried soil samples were passed through No. 4 sieve. For determining the Optimum Moisture Content and Maximum Dry Density water at 6%, 9%, 12% 15%, 18%, 21% and 24% were added with the air dried samples and compacted following the Modified Proctor Density test. AASHTO T 180 standard test method was followed. After the test the water content as measured from oven dried sample becomes 9.5%, 12%, 15.3%, 18.5%, 19.9%, 22.4% and 27.5% respectively. The measured density is plotted against the percentage of water content to establish a moisture density relationship for the collected Dhaka clay. This figure will help to estimate different water contents on both dry and wet side of the Optimum Moisture Content.

4.4 Unconfined Compression Strength Test

The compacted soil at modified compaction energy was removed from the compaction mould and was carefully trimmed. Samples 35.6 mm (1.4 inch) diameter and 71.1 mm (2.8 inch) height for Unconfined Compression Strength test were prepared for these different percentage of water content following ASTM designated standard procedure. ASTM standard D 2166 was followed for the test of sample. Two to three samples were prepared and tested for each percentage of water i.e. 9.5%, 12%, 15.3%, 18.5%, 19.9%, 22.4% and 27.5%. The samples were tested to failure or up to 20 percent strain. Stress – Strain curve is plotted for each test specimen. Unconfined Compression Strength at different water content is illustrated in Chapter 5.

4.5 California Bearing Ratio (CBR) Test

California Bearing Ratio (CBR) tests were performed for both soaked and unsoaked condition mixing water with the air dried samples to keep percentage of water same of Unconfined Compression Strength test and same energy of compaction. At first water content of the air dried samples were determined and then required amount of water was mixed to give ultimately the required percentage of water i.e. 9.5%, 12%, 15.3%, 18.5%, 19.9%, 22.4% and 27.5%. Water mixed soil samples were then compacted at modified compaction energy following AASHTO T 193 standard method. Test results for both soaked and unsoaked condition are illustrated in Chapter 5.

For CBR test in soaked condition soil samples after compaction were kept in water bath with the mould, setting filter paper on both sides of mould and soaking was allowed for 96 hours with standard surcharge load of 10 kg as per AASHTO T 193 standard method. During the soaking stage swelling of soil was measured with the help of preset dial guage with the mold. After the soaking is completed the specimen was tested as per AASHTO T 193 standard method. Vane shear test was also performed with the help of torvane.

For CBR test in unsoaked condition soil samples after compaction at modified compaction energy was tested directly according to AASHTO T 193 standard method. Vane shear test was not possible due to difficulty in penetrating the tore vane blade into compacted unsoaked CBR sample.

CHAPTER 5

EXPERIMENTAL RESULTS AND DISCUSSION

5.1 Introduction

The results of the laboratory investigations described in chapter 4 on Dhaka clay are reported and discussed herein. The geotechnical properties especially the strength parameters of this Dhaka clay such as Unconfined Compression Strength, California Bearing Ratio (CBR) value, Vane Shear Strength, Swelling index have been determined for different percentage of water content. Other parameters like Optimum Moisture Content, Maximum Dry Density, Index properties and Grain Size Distribution are also presented here.

5.2 The Physical and Index Properties of the Soil Used

The index properties such as liquid limit, plastic limit and plasticity index and physical properties such as colour, consistency, specific gravity and grain size distribution of the investigated Dhaka clay are shown in Table 5.1. The position of the soil sample in plasticity chart is shown in Figure 5.1. According to the fine grained soil classification by BNBC (1993) the soil is CI type. Gradation curve for this soil is shown in Figure 5.2. Ameen (1985) found W_L 41, I_P 22 and specific gravity 2.63 for Dhaka clay with 4.2% sand, 70.5% silt and 25.3% clay content. Uddin (1990) found W_L 43, I_P 23 and specific gravity 2.69 for Dhaka clay with 1% sand, 77% silt and 22% clay content.

5.3 Moisture Content - Dry Density Relationship

The experimental results of Moisture – Dry Density relationship of Dhaka clay has been presented in Figure 5.3. This curve is identical with the typical Moisture-Dry Density curve referenced in various text book of Soil Mechanics. The dry density of the soil increases with the increase of water content up to a certain limit, beyond which the dry density decreases with the increase of moulding water percentage. The portion of the curve with water content from

Table 5.1 The Physical and Index Properties of the Dhaka Clay Used

Physical Description of the Soil		Specific Gravity	Atterberg's Limits			Grain Size Distribution			Soil Classi-fication
Colour	Consis-tency		Liquid limit (%)	Plastic limit (%)	Plasticity Index	Sand (%)	Silt (%)	Clay (%)	
Reddish Brown	Medium to Stiff	2.67	39	22	17	8	60	32	CI

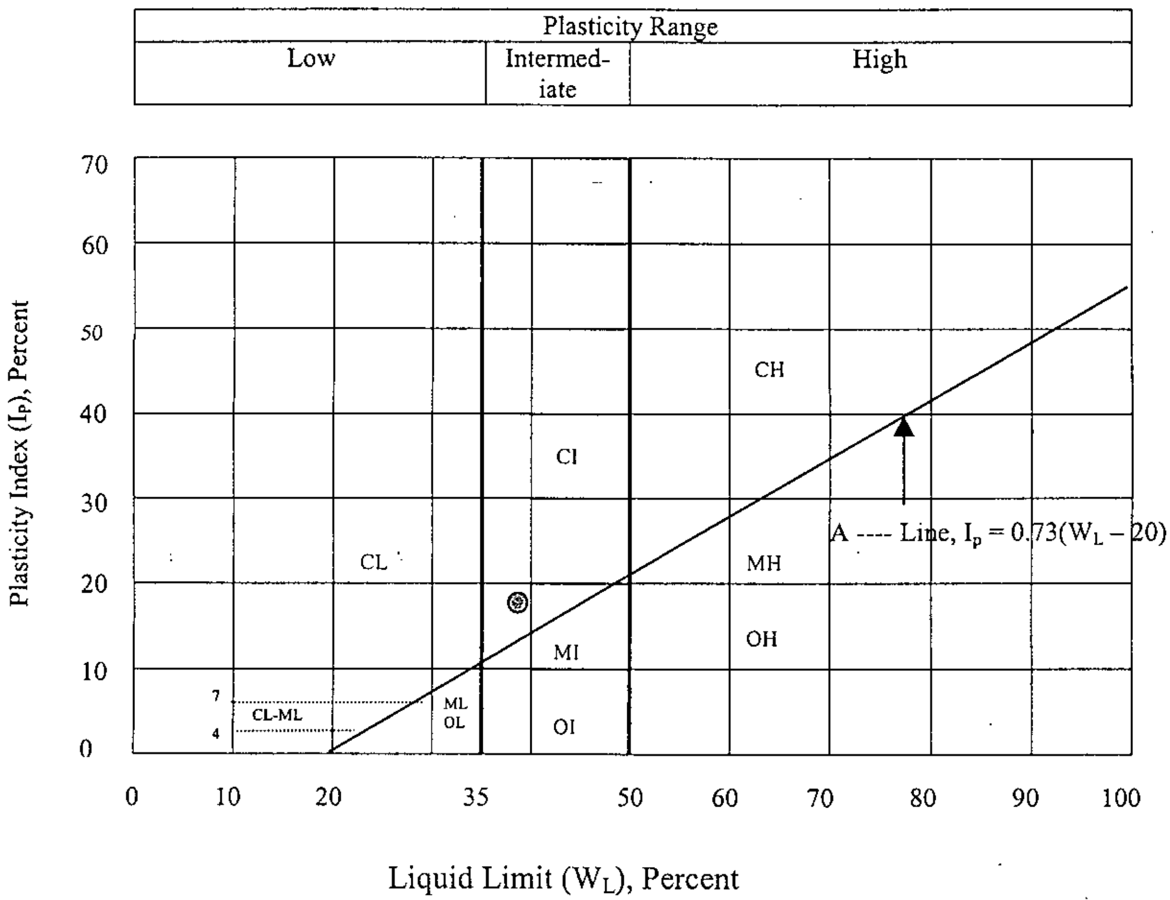


Figure: 5.1 Classification of Fine grained Soil by BNBC (1993)

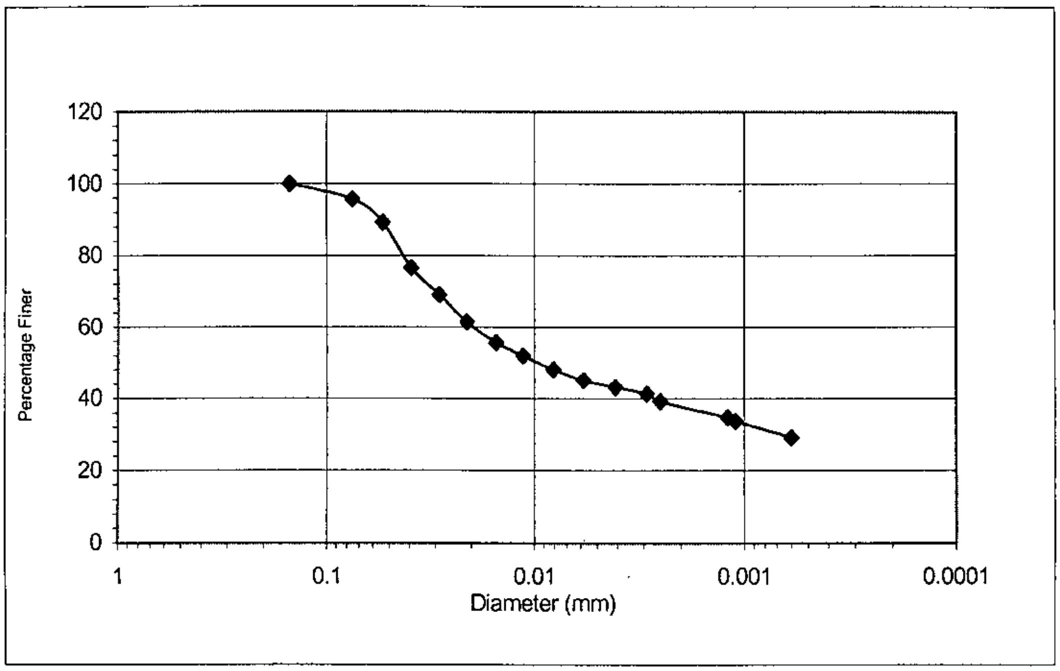


Figure 5.2 Gradation Curve for the Dhaka Clay used.

9.5% to 15.3% (OMC) is termed as dry side and the other portion from OMC up to the highest is termed as wet side. The soil compacted with modified compaction energy produces Maximum Dry Density (MDD) of 18.36 kN/m^3 and Optimum Moisture Content (OMC) of 15.3% water. Alam (1969) suggested that Maximum Dry Density increase with the increase of compaction energy, but corresponding Optimum Moisture Content decreases with the increase of compaction energy. Serajuddin and Azmal (1991) found similar type of moisture-density curve for alluvial soil from different parts of Bangladesh.

5.4 Unconfined Compression Strength

The Stress- Strain curve from unconfined compression strength test of soil at different percentage of moulding water is shown in Figure 5.4. The value of unconfined compressive strength (q_u) of the compacted Dhaka clay at different percentage of water content is shown in Figure 5.5 and Table 5.2 with degree of saturation. The stress-strain modulus (E_{50}) from unconfined compressive strength test is also plotted in Figure 5.5. As expected the strength on the dry side is more than the strength on the wet side. From the test results it is observed that the dry side sample is much stiffer than the wet side i.e. stress-strain curve is steeper for dry side. The unconfined compressive strength increases from lower value at lower percentage of water content upto a maximum at optimum moisture content and then decreases with the increase of moisture content. From test results it is observed that samples on the dry side with low water content have clear peak on stress-strain curve.

From a series of unconfined compressive strength tests at different percentage of water content and different compaction energy for alluvium soil of Bangladesh Alam (1969) found maximum value of unconfined compressive strength achieved at water content slightly lower than OMC. In present study for Dhaka clay maximum value of unconfined compressive strength is achieved at OMC.

From Figure 5.5 it is seen that initially with the increase of moulding water percentage q_u increases very rapidly and after reaching a maximum value it decreases rapidly with further increase of moisture content. It may be noticed that slope of the curve is steeper on the wet side just after optimum moisture content than the slope of the curve with 22.4% or more moulding water on wet side.

From the stress strain curve in Figure 5.4 it is also noticed that samples compacted at water content less than 20 percent developed distinct peak in unconfined compression strength test. For samples compacted with 20 percent or more moulding water content did not show any peak stress and q_u value at 20 percent strain can be considered the failure strength. The relationship between strain at peak stress and percentage of moulding water content is shown in Figure 5.6 and this relationship can be deduced as

$$\varepsilon = 0.5412\,\mathrm{Exp}^{0.185w} \qquad (5.1)$$

Where, ε is axial strain in percentage and w is percentage of moulding water content.

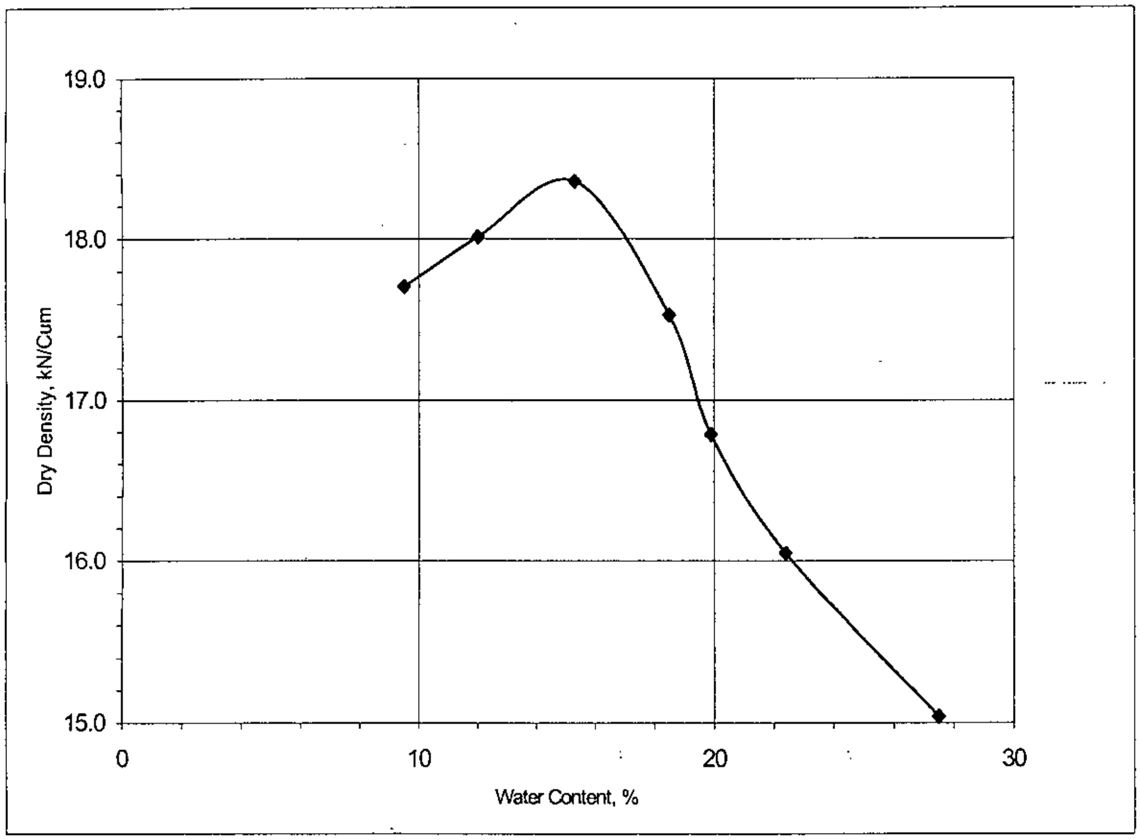


Figure 5.3 Moisture Content Vs Dry Density Relationship of Dhaka Clay at Modified Compaction Energy

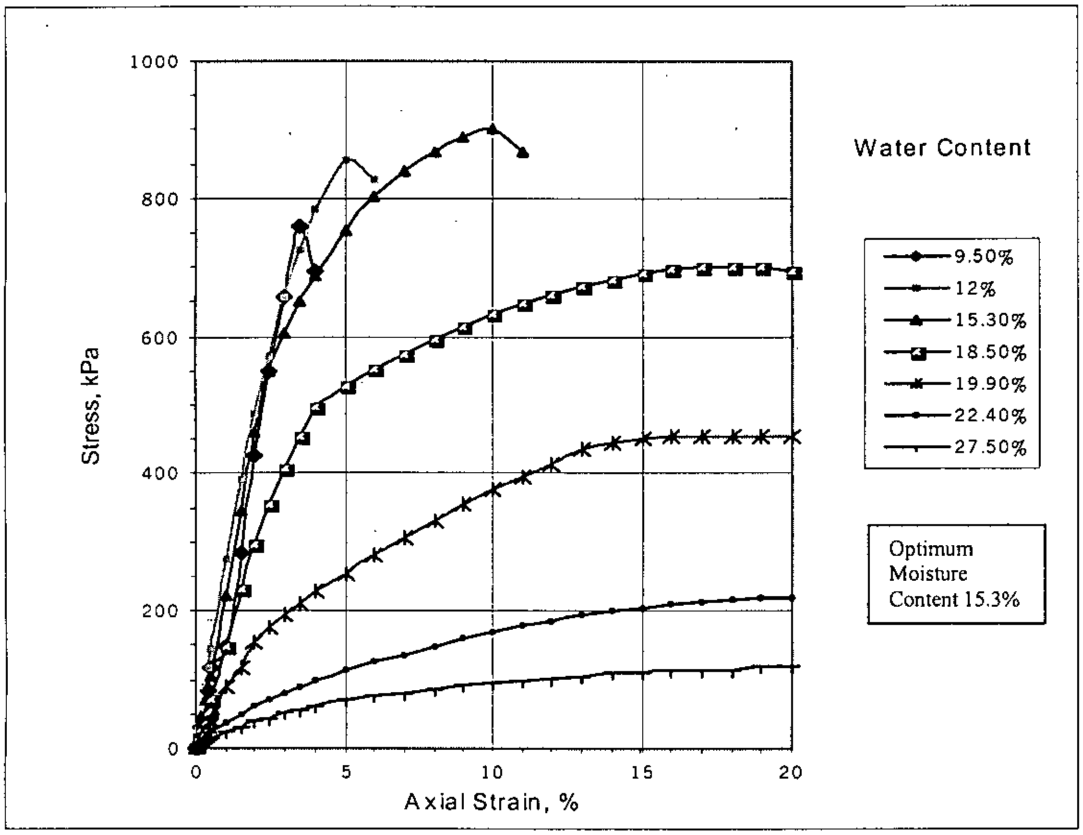


Figure 5.4 Stress - Strain Curve from Unconfined Compression Test for Compacted Dhaka Clay at Different percentage of Moulding Water Content

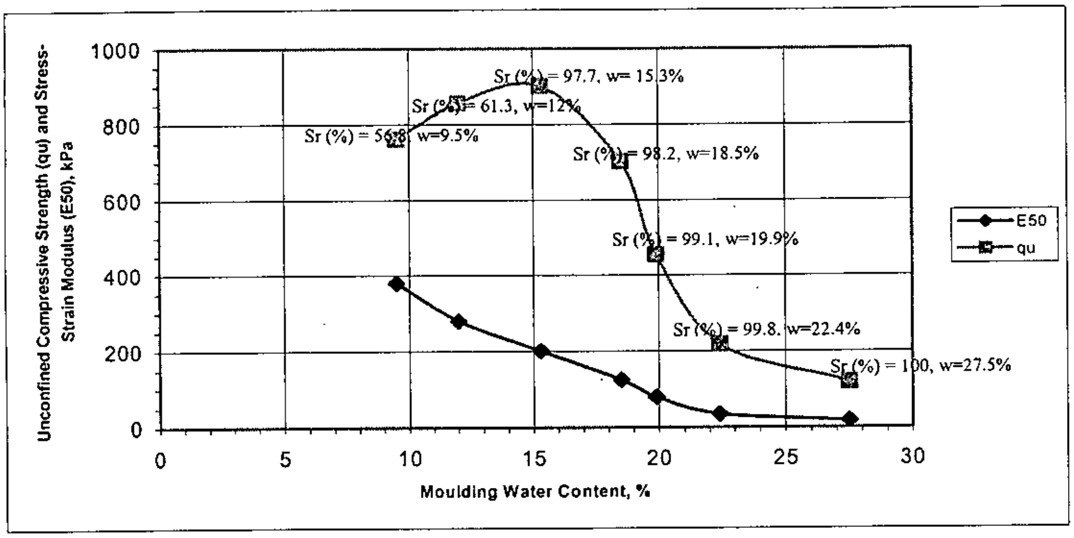


Figure 5.5 Unconfined Compressive Strength and Stress-Strain Modulus of Compacted Dhaka Clay at Different Percentage of Moulding Water and Degree of Saturation.

Table 5.2 Unconfined Compressive Strength at Different Percentage of Moulding Water.

Water Content (%)	9.5	12	15.3	18.5	19.9	22.4	27.5
Unconfined Compressive Strength, kPa	760.71	856.06	901.75	701.43	455.09	218.18	119.86
Degree of Saturation (S_r), %	56.8	61.3	97.7	98.2	99.1	99.8	100

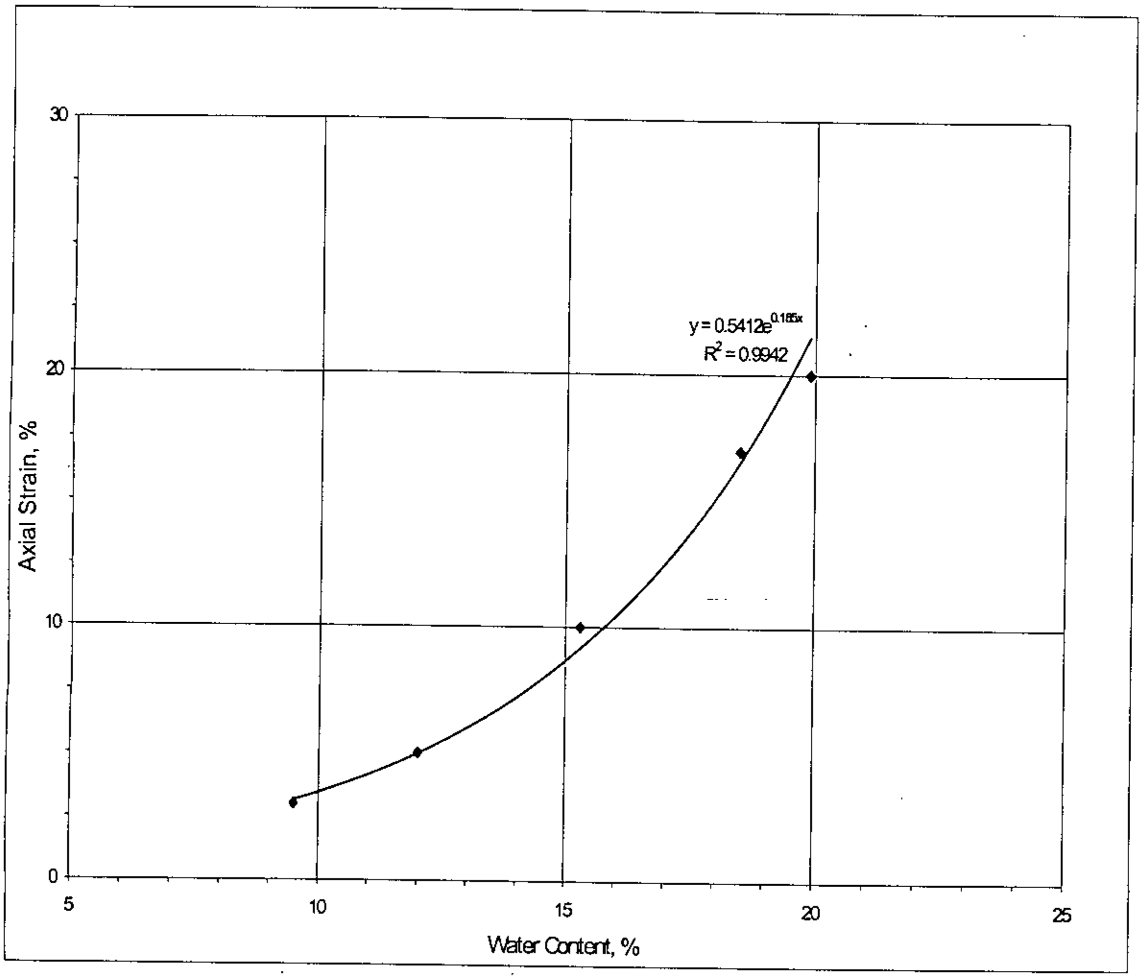


Figure 5.6 Axial Strain at Failure or Peak Stress during Unconfined Compression Test of Compacted Dhaka Clay at Different Moulding Water Contents.

5.5 Moisture- Dry Density and Unconfined Compression Strength

Figure 5.3 and Figure 5.5 show that both dry density and unconfined compression strength increase with the increase of moulding water content up to optimum moisture content (OMC). With moulding water content higher than OMC both dry density and unconfined compression strength decrease.

The relationship between Dry Density (γ_{dry}) and Unconfined compressive strength (q_u) relationship is shown in Figure 5.7. Figure 5.7 (a) shows the relationship on the dry side and Figure 5.7 (b) shows the relationship on the wet side. Both cases relationship is linear, but the correlation is different. For dry side the relationship can be deduced as:

$$q_u = 215.79\,\gamma_{dry} - 3050 \tag{5.2.a}$$

For wet side it is

$$q_u = 249.6\,\gamma_{dry} - 3701.9 \tag{5.2.b}$$

Where, q_u is unconfined compressive strength in kPa and γ_{dry} is dry density in kN/m^3.

5.6 California Bearing Ratio (CBR)

Figure 5.8 shows the California Bearing Ratio (CBR) value in both soaked and unsoaked condition at different percentage of moulding water for the experimented Dhaka clay. It indicates that unsoaked CBR values are quite high with respect to 4 days soaked CBR value. Also the slope of the unsoaked CBR curve is steeper than soaked CBR curve, which indicates that the samples in unsoaked condition is stiffer than the samples in soaked condition. For both cases maximum CBR value is obtained at optimum moisture content (OMC) starting from a lower CBR value at lower percentage of moulding water and increases with the increase of percentage of moulding water upto OMC. CBR value decrease with percentage of moulding water content more than OMC.

The relationship between soaked and unsoaked CBR is also shown in Figure 5.9. Figure 5.9 (a) is for the relationship on the dry side and Figure 5.9 (b) is for the relationship on wet side. For both dry and wet side the relationship is exponential. The equation as deduced from Figure 5.9 (a) for dry side of OMC is

$$CBR_u = 32.78Exp^{\,0.0217CBRs} \qquad (5.3.a)$$

and that for wet side from Figure 5.9 (b) is

$$CBR_u = 3.5844Exp^{\,0.1256CBRs} \qquad (5.3.b)$$

Where, CBR_u is CBR value in unsoaked condition and CBR_s is CBR value in soaked condition and for both cases values are in percentage.

Serajuddin and Azmal (1991) found soaked CBR value 33 to 44 percent of unsoaked CBR value for alluvium soil of Bangladesh. In present study soaked CBR value is 15 to 40 percent of unsoaked CBR value on dry side of OMC, but this value is 40 to 75 percent on wet side. From this statement it can be concluded that effect of soaking on wet side is less prominent than the effect of soaking on dry side.

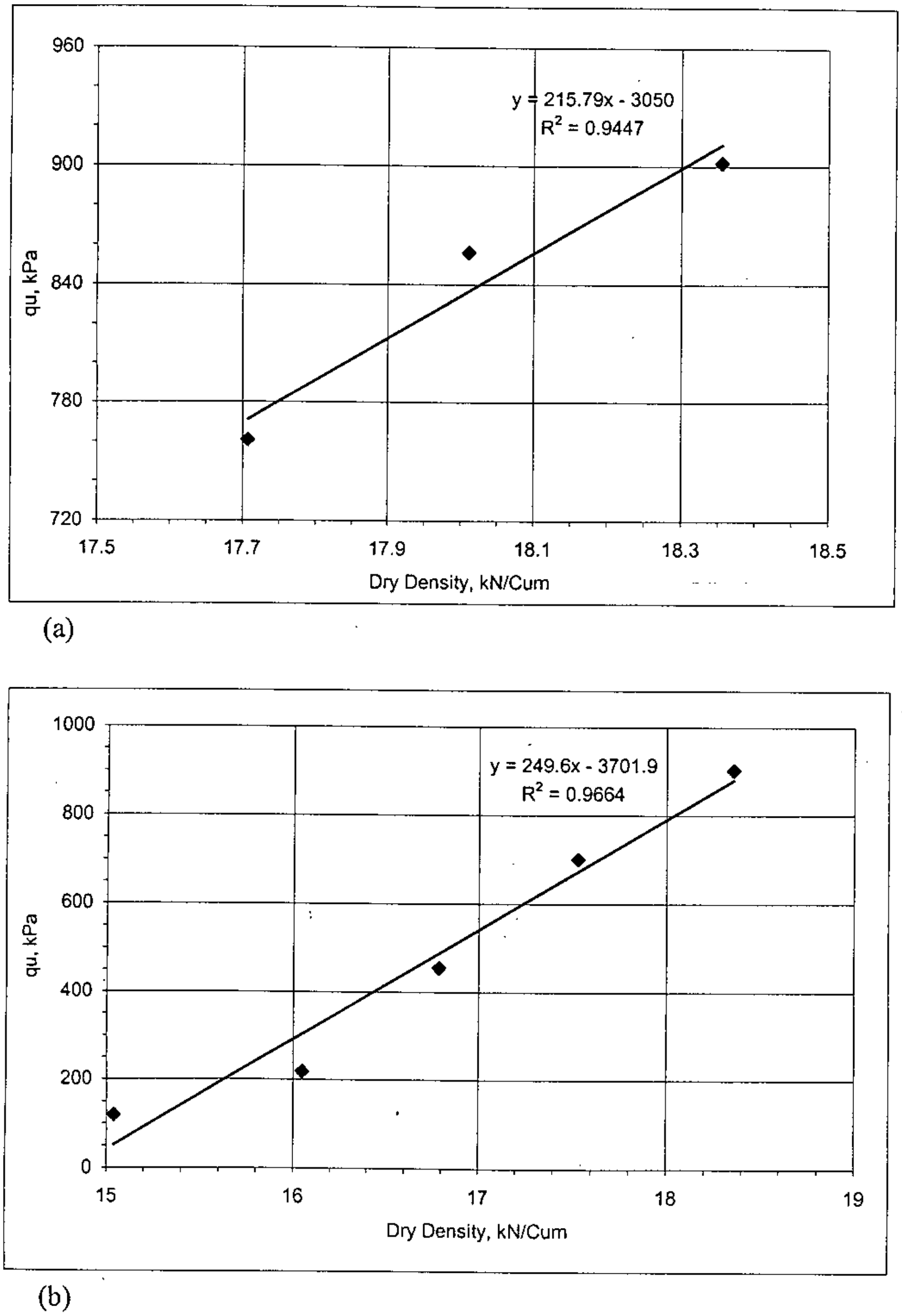


Figure 5.7 Relationship between Dry Density and Unconfined Compressive Strength on (a) Dry side and (b) Wet side of OMC.

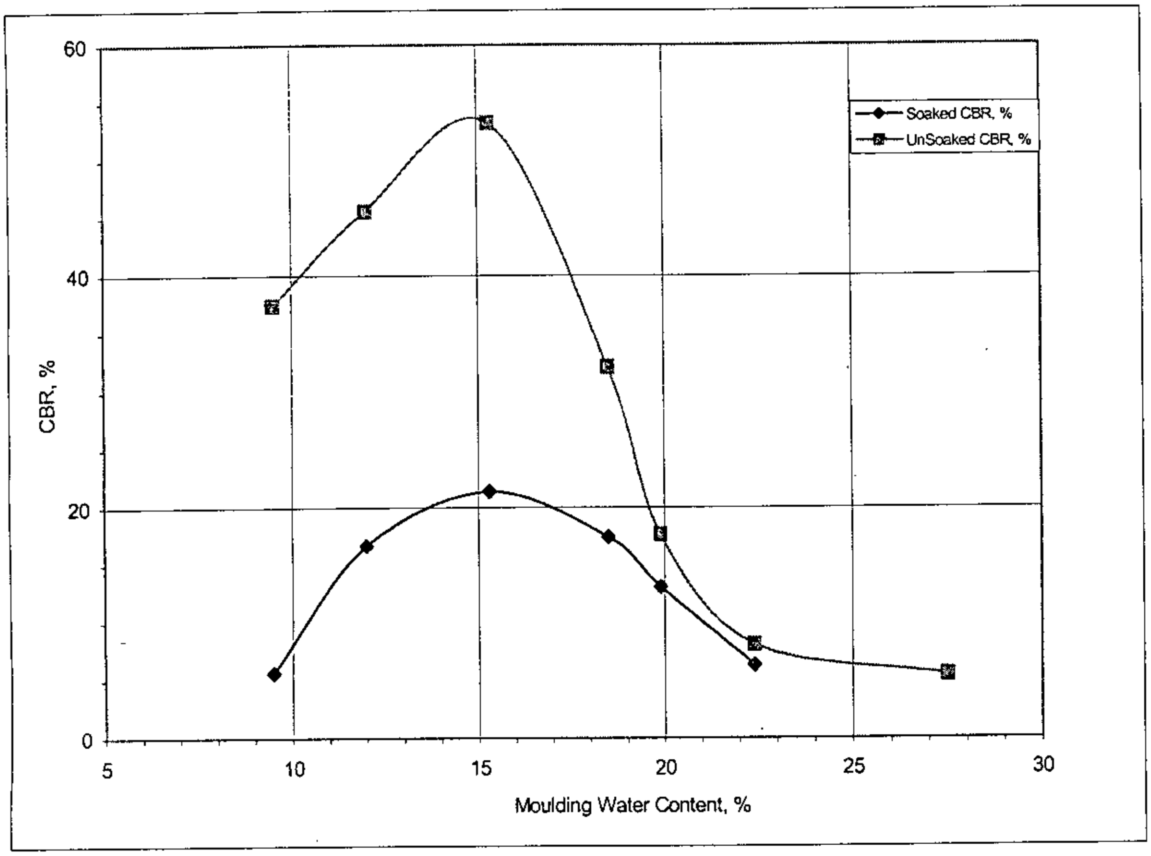
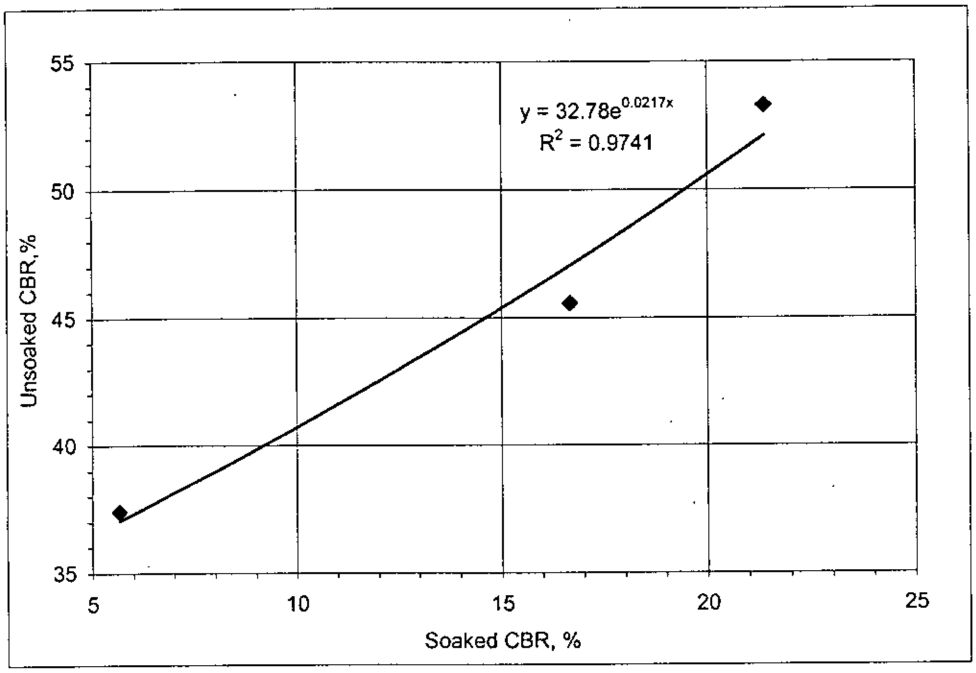
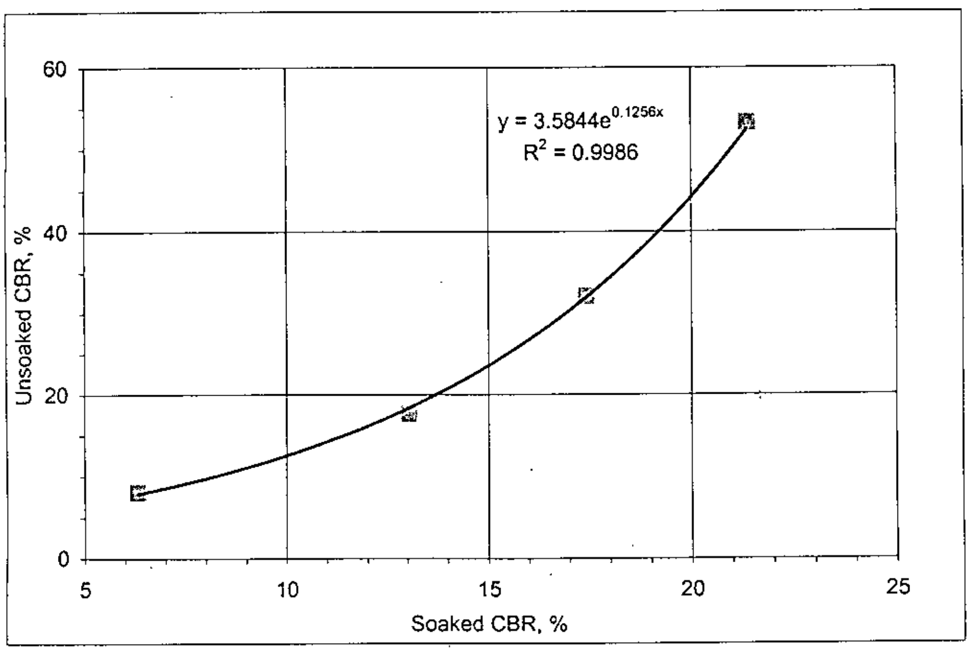


Figure 5.8 Plot of Soaked and Unsoaked CBR Value with respect to Different percentage of Moulding Water Content.



(a)



(b)

Figure 5.9 Relationship between Soaked and Unsoaked CBR value for both (a) Dry side and (b) Wet side of OMC.

5.7 Soaking and Swelling

The water content at different layer of the compacted CBR sample after soaking is shown in Figure 5.10, which indicates that specimen with lower percentage of moulding water absorbs more water than specimen with higher percentage of moulding water. Water content at the top layer of the specimen is the maximum. At the middle layer water content is the least. Percentage of swell at different percentage of moulding water content is shown in Figure 5.11. This figure shows that maximum percentage of swell is 2.8 and this happens for the specimen with the least percentage of moulding water i.e. 9.5%. The value of percentage of swell is gradually lower to negligible for specimen with higher percentage of moulding water. Serajuddin and Azmal (1991) found maximum 2.0 percentage of swelling for alluvium soil of Bangladesh. From Figure 5.11 the relationship between percentage of swell (C_s) with percentage of moulding water content (w) is logarithmic and can be written as:

$$C_s = 10.31 - 3.5145 \, Ln(w) \qquad (5.4)$$

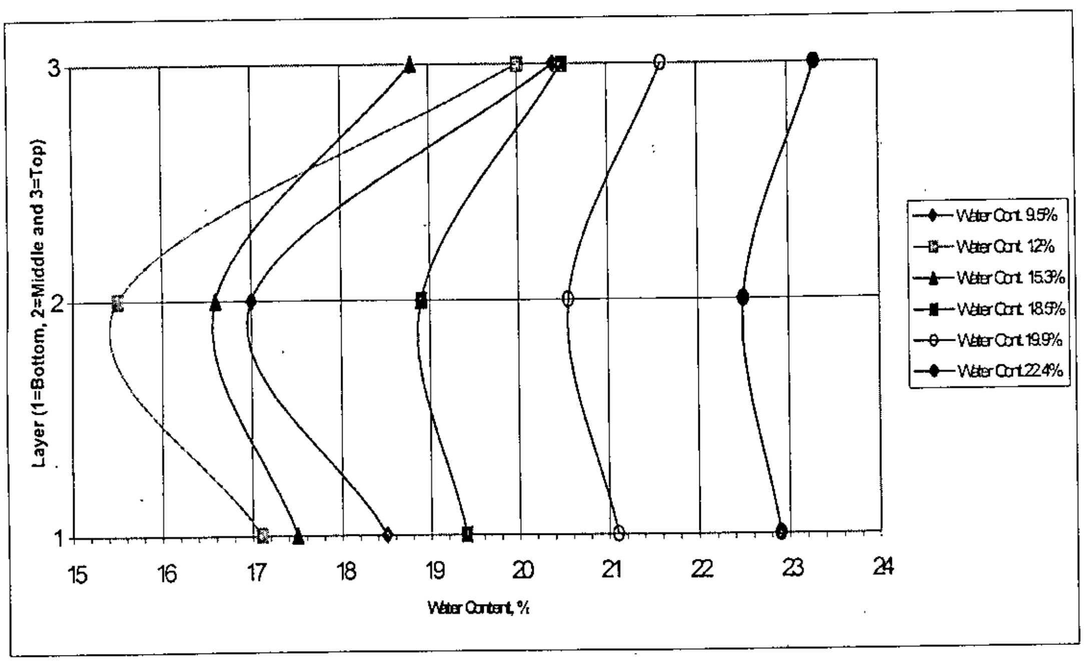


Figure 5.10 Water Content at Different Layer of Compacted CBR Sample after Soaking.

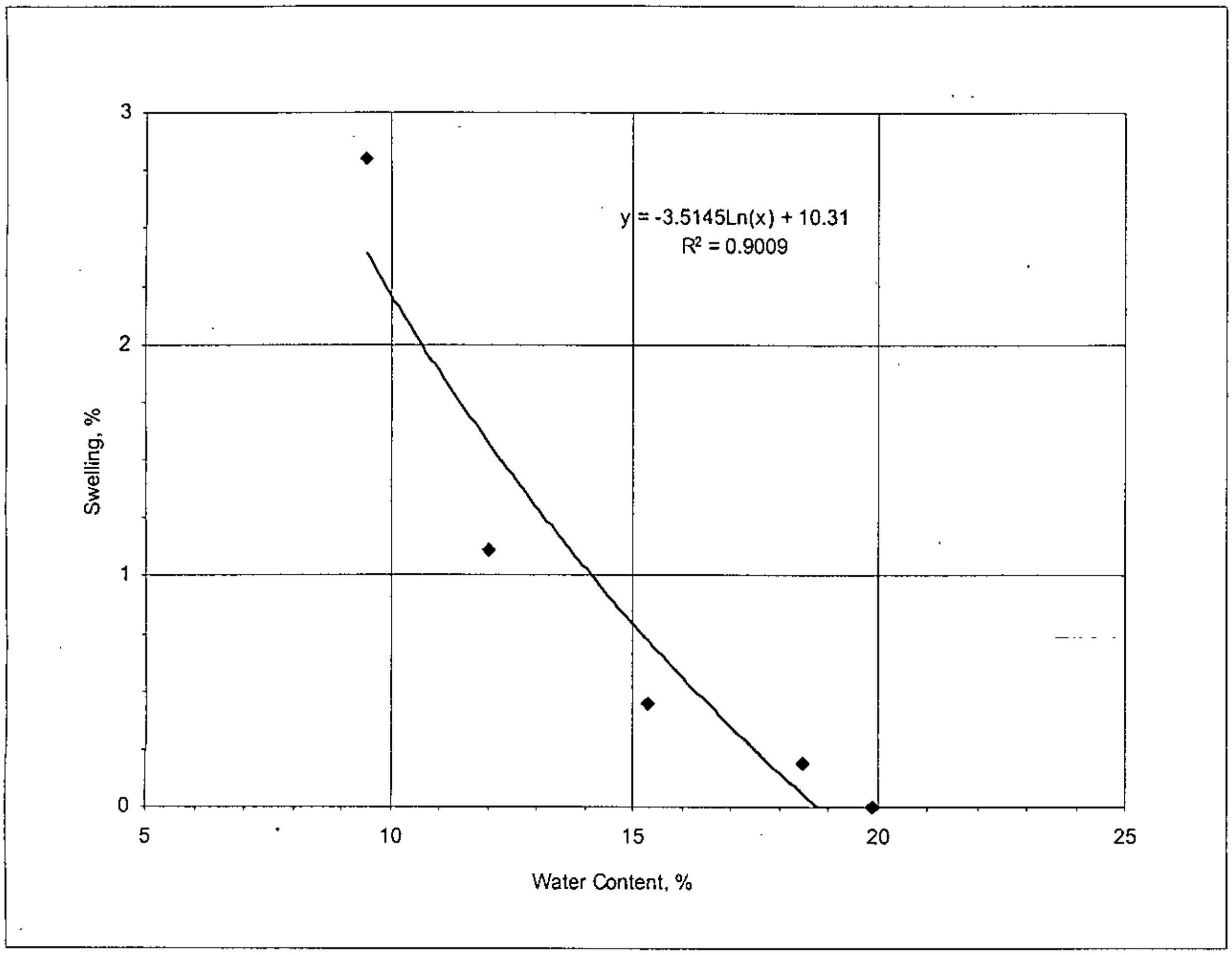


Figure 5.11 Percentage of Swelling at different Percentage of Moulding Water Content.

5.8 Relationship between Dry Density and CBR

For the Dhaka clay the relationship between dry density and CBR value for both soaked and unsoaked condition is shown in Figure 5.12(a) and 5.12(b) for dry side and wet side of OMC respectively. For dry side from Figure 5.12 (a) this relationship is linear for both soaked and unsoaked condition.

For CBR value in unsoaked condition the relationship is:

$$CBR_u = 24.461\ \gamma_{dry} - 395.47 \tag{5.5.a}$$

For CBR value in soaked condition the relationship is:

$$CBR_s = 24.028\ \gamma_{dry} - 418.53 \tag{5.5.b}$$

For wet side from Figure 5.12(b) this relationship for unsoaked CBR is exponential and expressed is by

$$CBR_u = 0.00009 Exp^{0.7241\ \gamma\ dry} \tag{5.6.a}$$

and that for soaked CBR is logarithmic which is expressed by

$$CBR_s = 111\ Ln(\gamma_{dry}) - 300.97 \tag{5.6.b}$$

Where, CBR values are in percentage and dry density in kN/m^3. Both cases Soaked CBR values are lower than unsoaked CBR value with respect to dry density.

5.9 Relationship between Unconfined Compression Strength and CBR

The relationships between unconfined compressive strength (q_u) and CBR value for both dry and wet side are presented in Figure 5.13(a) and 5.13(b) respectively. Due to higher value of unsoaked CBR in comparison with soaked CBR value the curve for unsoaked CBR stands above the curve of soaked CBR for both dry side and wet side of OMC. With the increase of q_u both soaked and unsoaked CBR value increase.

This relationship between CBR and q_u is linear for both soaked and unsoaked conditions on dry side of OMC and from Figure 5.13 (a) the rlationship can be written as-

For unsoaked CBR condition on dry side of OMC

$$CBR_u = 0.1084q_u - 45.604 \qquad (5.7.a)$$

and for Soaked CBR condition on dry side of OMC

$$CBR_s = 0.1121q_u - 79.518 \qquad (5.7.b)$$

The relationship between CBR and q_u is not linear on wet side. They are correlated by exponential equation for unsoaked CBR and logarithmic equation for soaked CBR.

For unsoaked CBR condition on wet side of OMC

$$CBR_u = 4.2495Exp^{0.0029qu} \qquad (5.8.a)$$

For soaked CBR condition on wet side of OMC

$$CBR_s = 10.327Ln(q_u) - 49.651 \qquad (5.8.b)$$

Where, CBR values are in percentage and q_u is in kPa. Serajuddin and Azmal (1991) found soaked CBR value (%) 3 to 3.5 times of soaked unconfined compressive strength in kg/cm^2.

5.10 Relationship between Vane Shear Strength and Soaked CBR

Vane (S_{uv}) shear strength produced from Torvane Shear test is correlated with Soaked CBR value and is presented in Figure 5.14. This relationship is exponential for both dry and wet side of OMC.

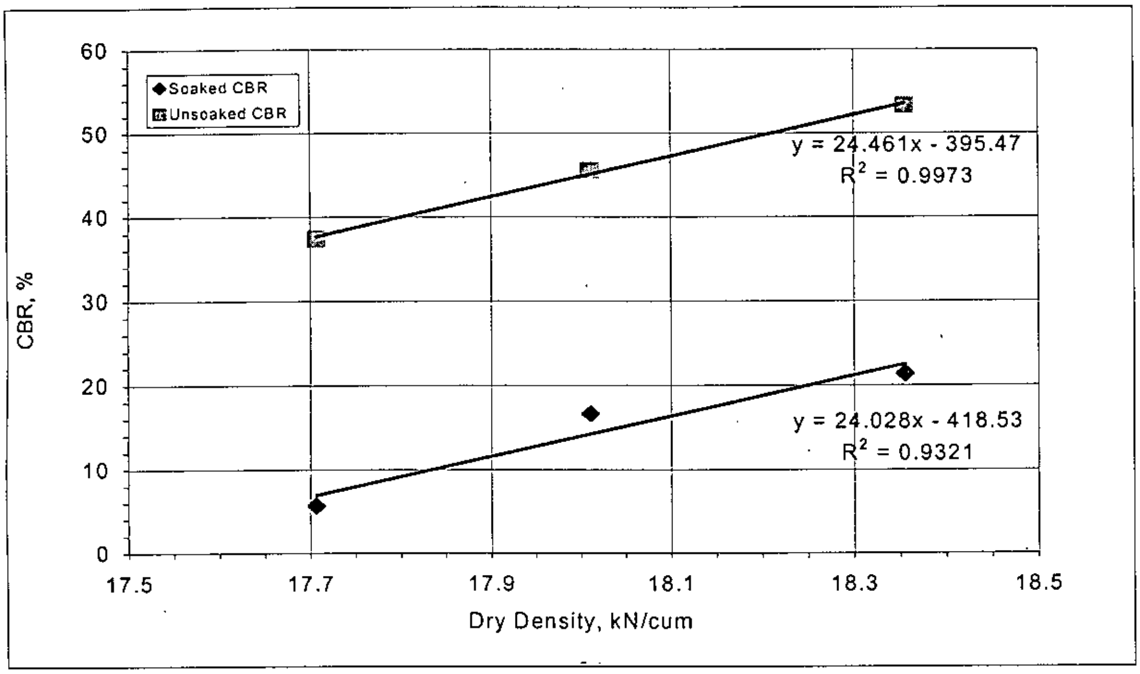
For dry side from Figure 5.14 (a) the relationship is expressed by

$$S_{uv} = 2.7474Exp^{0.0714\,CBRs} \qquad (5.9.a)$$

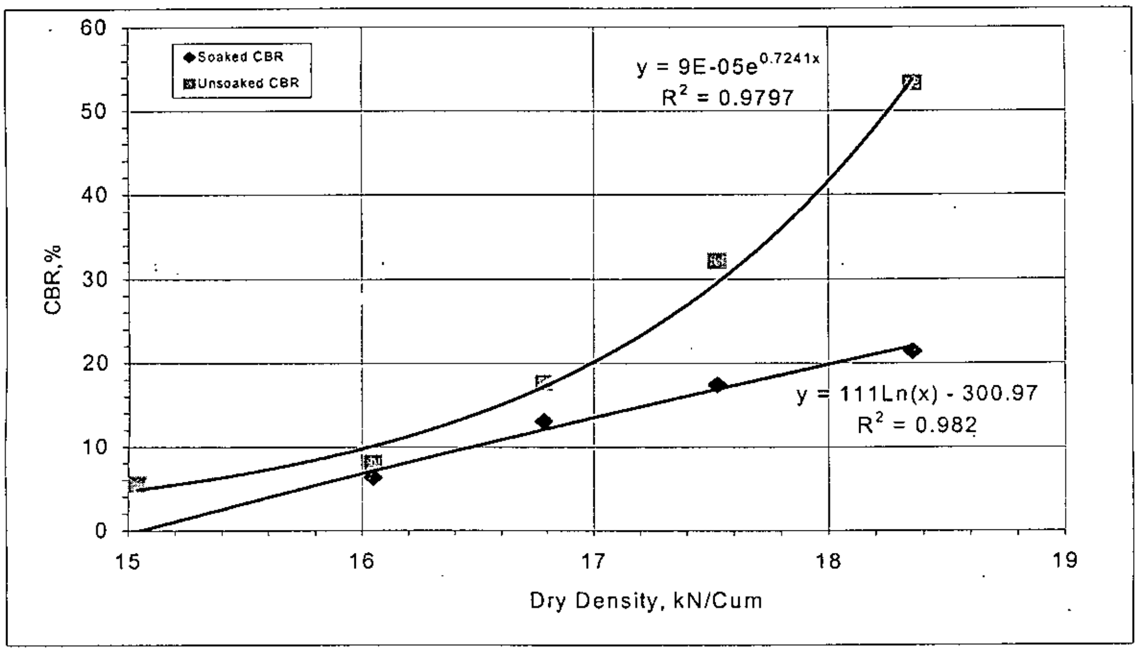
For wet side from Figure 5.14(b) the relationship is expressed by

$$S_{uv} = 2.1722Exp^{0.0945\,CBRs} \qquad (5.9.b)$$

Where, vane shear strength S_{uv} is in kg/cm^2 and soaked CBR value is in percentage.

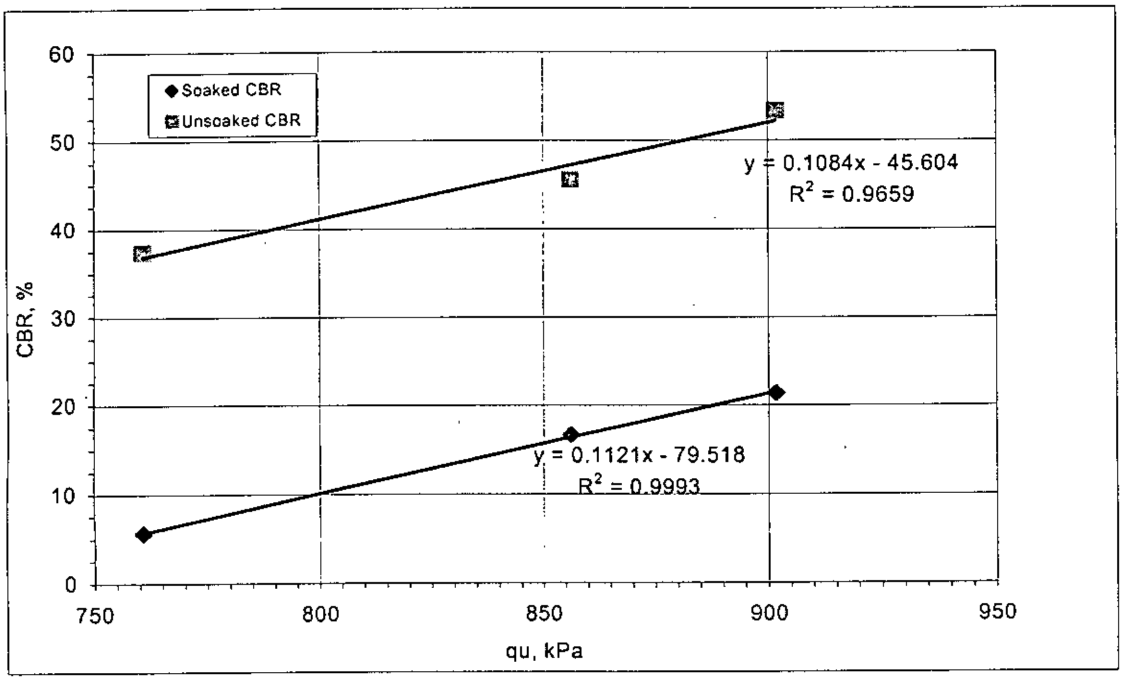


(a)

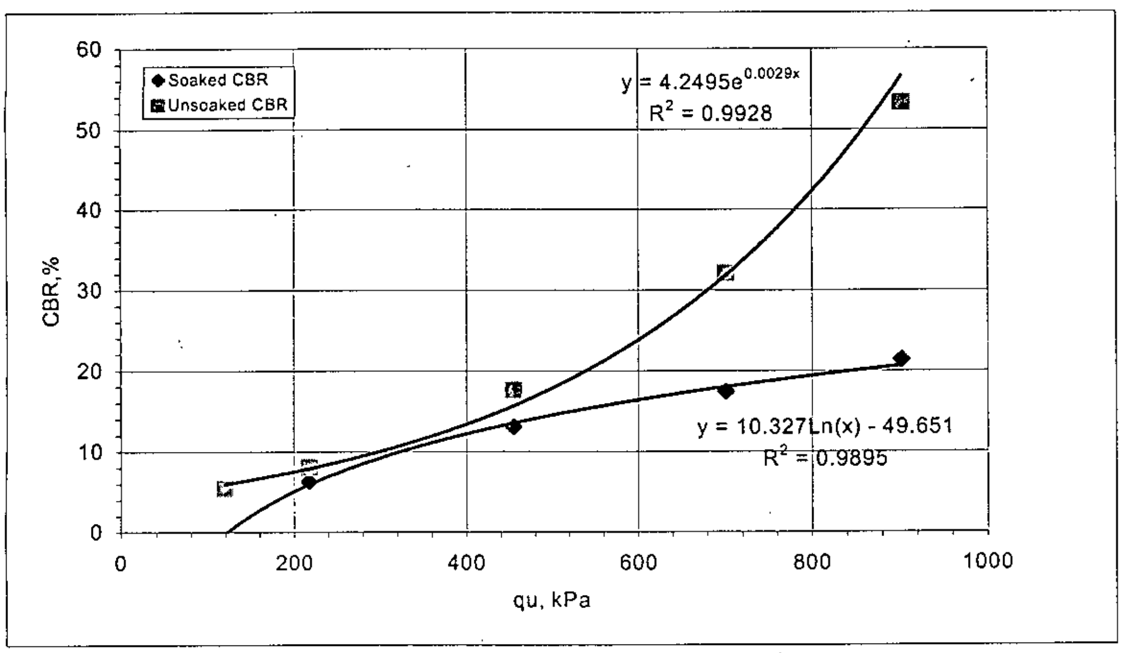


(b)

Figure 5.12 Relationship between Dry Density and CBR for both Soaked and Unsoaked conditions in (a) Dry side and (b) Wet side of OMC.

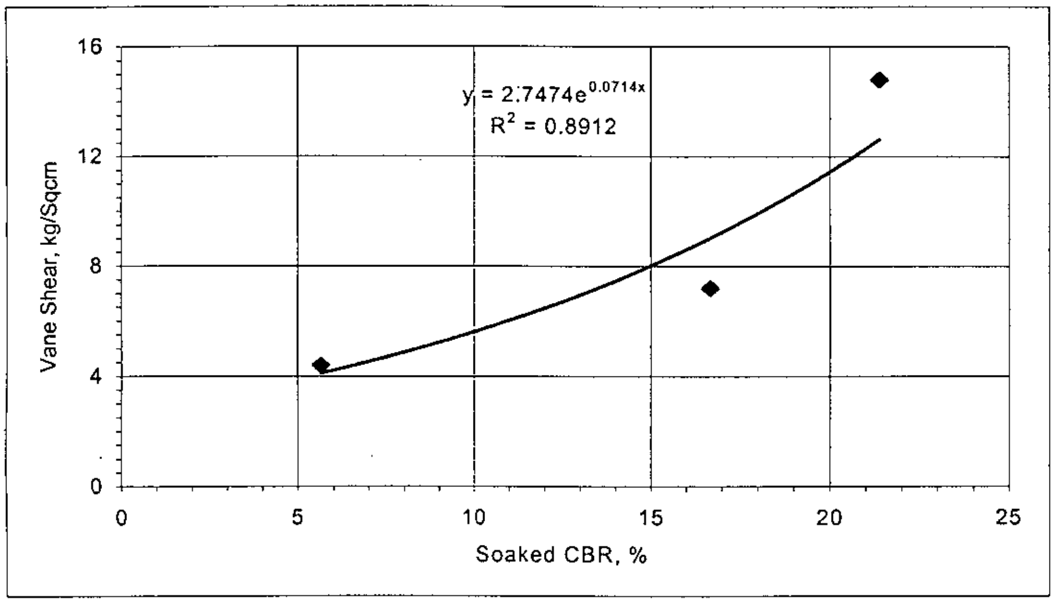


(a)

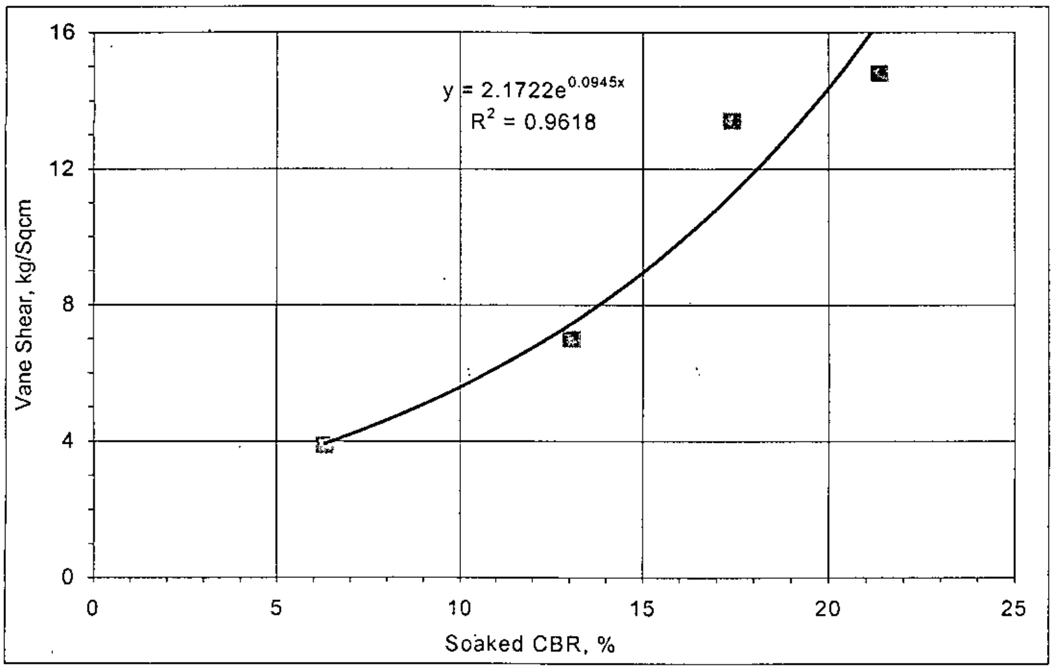


(b)

Figure 5.13 Relationship between CBR and Unconfined Compressive Strength at Different Percentage of Moulding Water Content for both Soaked and Unsoaked conditions in (a) Dry side and (b) Wet side of OMC.



(a)



(b)

Figure 5.14 Relationship between Soaked CBR and Vane Shear Strength for (a) Dry side and (b) Wet side of OMC.

CHAPTER 6
CONCLUSION AND RECOMMENDATION

6.1 Conclusions

Unconfined Compressive Strength, California Bearing Ratio, Dry Density, Vane Shear Strength, Plastic Limit, Liquid Limit and Grain Size Distribution of compacted Dhaka clay have been investigated in this present study. Attempts have also been made to evaluate the parameters with a view to have correlation between them. Based on the experimental results and limited number of data the following findings and conclusions may now be made:

1. The dry density, q_u and CBR values of compacted clay increase with the increase of water content in the dry side up to optimum moisture content, but decrease with the addition of more water in wet side. The pattern of Unconfined Compressive Strength curve (Figure 5.5) and California Bearing Ratio curve (Figure 5.8) at different percentage of moulding water is similar with the Moisture–Density curve (Figure 5.3). Both the q_u and CBR value is found to be maximum at moulding water content corresponding to optimum moisture content.

2. All samples compacted at water content less than 20 percentage developed distinct peak in the stress-strain curve (Figure 5.4) in the unconfined compression strength test. Samples with 20 percent or more moulding water content did not show any peak stress during the unconfined compression strength test. For these samples q_u value at 20 percent strain is assumed to be the failure strength. The relationship of strain at failure or peak stress with percentage of moulding water content for Dhaka clay can be expressed by the following equation-

$$\varepsilon = 0.5412\,\mathrm{Exp}^{0.185w} \qquad (5.1)$$

Where, ε is axial strain in percentage and w is percentage of moulding water content.

3. The plotted curve between dry density and q_u is linear (Figure 5.7) for both dry and wet side of optimum moisture content. Their relationships can be expressed by the following equations-

For dry side of compaction curve:

$$q_u = 215.79\ \gamma_{dry} - 3050 \qquad (5.2.a)$$

For wet side of compaction curve:

$$q_u = 249.6\ \gamma_{dry} - 3701.9 \qquad (5.2.b)$$

where, q_u is in kPa and γ_{dry} is in kN/m^3.

4. The relationship between dry density and CBR (Figure 5.12) is linear on dry side of optimum moisture content for both unsoaked and soaked CBR values. These relationships can be expressed by the following equations-

For unsoaked CBR values on dry side of compaction curve:

$$CBR_u = 24.461\ \gamma_{dry} - 395.47 \qquad (5.5.a)$$

For soaked CBR Values on dry side of compaction curve:

$$CBR_s = 24.028\ \gamma_{dry} - 418.53 \qquad (5.5.b)$$

This relationship on wet side of optimum moisture content is exponential for unsoaked CBR and logarithmic for soaked CBR and can be expressed by the following equations-

For unsoaked CBR vlaues on wet side of compaction curve :

$$CBR_u = 0.00009 Exp^{0.7241\ \gamma\ dry} \qquad (5.6.a)$$

For soaked CBR values on wet side of compaction curve:

$$CBR_s = 111\ Ln(\gamma_{dry}) - 300.97 \qquad (5.6.b)$$

where, CBR values are in percentage and γ_{dry} is in kN/m^3.

5. The relationship between q_u and CBR (Figure 5.13) is linear for both soaked and unsoaked CBR on dry side of optimum moisture content and can be expressed by the following equations-

   For unsoaked CBR values on dry side of compaction curve:

   $$CBR_u = 0.1084q_u - 45.604 \quad (5.7.a)$$

   For Soaked CBR values on dry side of compaction curve:

   $$CBR_s = 0.1121q_u - 79.518 \quad (5.7.b)$$

   This relationship on wet side of optimum moisture content is exponential for unsoaked CBR and logarithmic for soaked CBR and can be expressed by the following equations-

   For unsoaked CBR values on wet side of compaction curve:

   $$CBR_u = 4.2495Exp^{0.0029qu} \quad (5.8.a)$$

   For soaked CBR values on wet side of compaction curve:

   $$CBR_s = 10.327Ln(q_u) - 49.651 \quad (5.8.b)$$

   where, CBR values are in percentage and q_u is in kPa.

6. The relational pattern between unsoaked CBR and soaked CBR (Figure 5.9) values are nonlinear (exponential) for both dry and wet side of optimum moisture content. The relationships can be expressed by the following equations-

   For dry side of compaction curve:

   $$CBR_u = 32.78Exp^{0.0217CBRs} \quad (5.3.a)$$

   For wet side of compaction curve:

   $$CBR_u = 3.5844Exp^{0.1256CBRs} \quad (5.3.b)$$

   Where, CBR_u is CBR value in unsoaked condition and CBR_s is CBR value in soaked condition and for both cases values are in percentage.

7. Percentage of swell is higher for samples with lower percentage of moulding water content on dry side and it decreases with the increase of moulding water content. At 20 percent or more moulding water content percentage of swell is negligible. Maximum 2.8 percent swelling is observed at 9.5 percent of moulding water content. The relationship between percentage of swell and moulding water content is nonlinear (logarithmic in Figure 5.11) and can be expressed by the following equation-

$$C_s = 10.31 - 3.5145\ \mathrm{Ln}(w) \qquad (5.4)$$

8. The relationship between soaked CBR value and vane shear strength (Figure 5.14) is found nonlinear (exponential) for both dry and wet side of optimum moisture content. These relationships can be expressed by the following equations-

For dry side of compaction curve:

$$S_{uv} = 2.7474\mathrm{Exp}^{\,0.0714\ CBRs} \qquad (5.9.a)$$

For wet side of compaction curve:

$$S_{uv} = 2.1722\mathrm{Exp}^{0.0945\ CBRs} \qquad (5.9.b)$$

where, vane shear strength S_{uv} is in kg/cm^2 and soaked CBR value is in percentage.

6.2 Recommendations for Future Study

In this study Unconfined Compressive Strength, California Bearing Ratio, Dry Density, Vane Shear Strength, Plastic Limit, Liquid Limit and Grain Size Distribution of compacted Dhaka clay have been investigated collecting soil from one location only and attempts have made to establish correlation among q_u, w, CBR, γ_{dry}, S_{uv}, Swelling index. Several aspects of the work presented in the project require further study. Some of the important areas of further research may be listed as follows:

1. Density and strength parameters for compacted Dhaka clay are determined with modified compaction energy only. Further study can be conducted using different compaction energy.

2. Relationships of different strength parameters dry density, q_u, CBR for both soaked and unsoaked condition, strain, water content, vane shear strength, percentage of swell etc are made, but relationship between strength and compressibility characteristics can be studied further.

3. The present study is carried out using Dhaka clay collected from Pallabi Phase II, Land project of Eastern Housing Ltd. at the east side of National Botanical Garden. This study can be repeated using soil samples from other locations of the Modhupur tract.

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