THE UNIVERSITY
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MATERIALS CONSIDERED FOR PAVEMENT AND EMBANKMENT DESIGN

by

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ABSTRACT

Bangladesh has about has about 21,900 km of road about 65% of this network comprises rural roads. The remainder comprises both National and Regional roads in about equal proportions. This infrastructure in built using local materials which includes alluvial sands and silts, which at time contain mica and varying amount of clay, and organic materials. In the main, crushed brick aggregate is pavement construction due to scarcity of natural stone. In addition to this, substantial sections of the network are subjected to flooding for about three months annually. These conditions pose challenges to designers, who invariable have limited methods of evaluating relevant soil properties for undertaking suitable design. This often means that some form of index test is conducted and then suitable soil parameter is ascertained through some correlation. The correlations used, in the main have not been developed for soils and condition in Bangladesh.

In order to overcome some of the challenges posed by the existing design and construction methodologies, an investigation was undertaken to evaluate properties of soils through various means, which in the main included locally available equipment and evaluating correlations for better estimating required properties. This work comprised of a ground investigations (using trial pits and hand auger) of all three road types that included a significant embankment. Insitu properties were determined using standard penetration tests, density determination and Benkleman beam tests and laboratory investigations included determination of moisture content, index properties of soil, dry density/moisture content relationship and strength tests. Some cyclic load tests were also conducted. The study also included assessment of previously available information from ground investigations.

Results of the investigations showed that there was limited amount of previous information and that which was available did not often contain the required information. It was also found that very limited range of apparatus was available in Bangladesh for assessing properties of soils relevant to pavement design. These limitations pointed to the importance of ensuring that only correlations that were developed for local soils should be used. During this investigations limited amount of correlations were confirmed due to time constraints. One of the most useful outputs was the evaluation of resilient modulus of the subgrade soils, which will be used in pavement design.

In terms of the black top there seems to have been and adequate range of tests that could be conducted. These showed that some of the surfacing materials had aged and no longer complied with the specifications.

To My Parents

And

Wife

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List of ABBREVIATIONS

AASHTO	: American Association of State Highway and Transportation Officials
ACV	: Aggregate Crushing Value
AIV	: Aggregate Impact Value
ASTM	: American Society for Testing Materials
BRRL	: Bangladesh Road Research Laboratory
BS	: British Standard
CBR	: California Bearing Ratio
Cft	: Cubic foot
Cum	: Cubic meter
DCP	: Dynamic Cone Penetrometer
DoT, UK	: Department of Transportation, U.K.
ESA	: Equivalent Standard Axle
FM	: Fineness Modulus
Ib	: Pound
I_P	: Plasticity Index
I_L	: Liquidity Index
Kg	: Kilo Gram
kPa	: Kilo Pascle
km	: Kilo Meter
kN	: Kilo- Newton
LAAV	: Los Angles Abrasion Test
m^2	: Square meter
m^3	: Cubic meter
MDD	: Maximum Dry Density
MPa	: Mega Pascle
OMC	: Optimum Moisture Content
Overseas RN31	: Overseas Road Note 31
Psi	: Pound per Square inch.
RHD	: Roads and Highways Department

RMSS : Road Materials and Standard Study

STP : Standard Testing Procedures, RHD

TFV : Ten Percent Fines Value

TRL : Transport Research Laboratory, U.K.

UCS : Unconfined Compressive Strength

UK : United Kingdom

USA : United States of America

WBM : Water Bound Macadum.

w : Moisture content

w_L : Liquid Limit

w_P : Plastic Limit

Chapter 1: General Introduction

1.1 Introduction

The development of communication infrastructure (road networks and railways) is the key elements of development of a nation such as Bangladesh. Improved, effective and efficient road network plays a pivotal role in development as well as economic activities. In Bangladesh, Roads and Highways Department (RHD) is primarily responsible for design, construction and maintenance of 20,889 km roads which comprise National highways (3476 km), Regional highways (4165 km) and [1]Zilla roads (13248 km).

All RHD roads are built on high embankments because a large part of the country is inundated during the monsoon season. So it is a little surprising that whilst there are specifications for roads, none exist for the construction of earthworks for embankment. There are some construction implementation manuals, but these manuals have been prepared based on overseas design standards and specifications which may not always be suitable for Bangladesh. Besides these, there are scarcities of conventional road construction materials. For this reason, non conventional materials such as brick aggregate are used in the construction of roads in Bangladesh. These materials characteristics do not comply with the overseas materials specifications for pavement construction. Need to contract more economic needs with low whale life cost is a pavement in Bangladesh as it is elsewhere. Thus there is a need to develop specifications, utilizing local materials and available technology for both investigation and construction of pavements. This also applies to the construction of embankments, where seasonal flooding has a major influence on the behaviour of the embankment.

This study was undertaken for an eighteen month period from March, 2008 to August, 2009. It comprised of a literature review of available soils and materials properties, available correlations between simple measurement techniques and engineering properties, continuation of same and modifications of some correlations based on the both laboratory and field testing in Bangladesh and in the UK in the laboratories of the school of civil engineering in the University of Birmingham. This data and test results

[1] Zilla road means thana connecting (rural) road.

from the various investigations were also included in investigation undertaken by Bhattachayya (2009) and Haider (2009) in their respective studies such as embankment design and data analysis & risk assessment.

The work undertaken during the investigation is described in detail together with the aims and objectives in the following sections.

1.2 Background

In Bangladesh, most of the roads especially in the flood prone area of the country are built on embankment. To date there is no integrated specification based on sound engineering principles that relates to the construction of embankments to support roads. There are some standard and specifications for pavement but nothing of embankment. There is no comprehensive and integrated research works regarding evaluation of design input parameters supported by field and laboratory testing for pavement as well as embankment design for Bangladesh. However, some scattered works have been done regarding locally available pavement materials. This study reports a review of the available studies and proceeds to develop procedure of data / results evaluation, data gathering features. Previous works include characterization of unbound granular brick aggregates by Zakaria (1986) for purposes of pavement construction. This was followed by Alam (2002) who investigated the use of cement stabilized brick layer in pavement constructions in order to overcome the problem of moisture effects. In addition to this, Rahman (2004) investigated the possibility of using stabilized locally available sand in pavement instead of bricks. Both Alam (2002) and Rahman (2004) proved the constructability of their materials through full scale construction.

Various researchers such as Serajuddin (1996), Ameen (1985), Ullah (2001) and Ferthous (2006) evaluated the properties of soils in Bangladesh. However, engineering properties required for the design of embankment were not investigated. In addition to this, although many embankments have been constructed in Bangladesh for many years, no suitable geotechnical information was available. Although it was investigated that lack of the information from the previous properties could be used, since was further duly this study field investigation data and laboratory investigation were required. Much of this work was in addition to the originally envisaged at the start of the study. This study was financed by DFID.

1.3 Aim of the Research:

The aim of the research was to develop a methodology for the evaluation of the properties of soils and granular materials used in the design of embankment and pavement for Bangladesh. This research work was focused on the establishment of data base as well as correlations for the design input parameters for future design of pavement built on embankment. This data base and correlations was developed by appropriate field or laboratory or both testing techniques. Moreover, existing correlations were also reviewed and compared with the developed ones.

1.4 Objective of the Research:

The objective of this research programme has to make a compressive evaluation of pavement materials as well as embankment soils in terms of fundamental engineering parameters for the purposes of facilitating the sustainable and economic design of pavement and embankment.

The specific research objectives formulated for the programme are as follows:

- To ascertain the design input parameters for the design of pavement and embankment design.
- To identify the appropriate testing techniques such as field and laboratory testing for the determination of those input parameters based on the accuracy and availability of the equipment at Bangladesh.
- To identify the appropriate site for road pavement and embankment for performing the intensive insitu and exsitu testing.
- To carry out the above techniques for the materials likely to be found in the road pavements of the trails sections used in the overall pavement and embankment design project for Bangladesh.
- To establish the database and correlations for the design parameters that will be used for future pavement and embankment design project for Bangladesh.

1.5 Thesis layout

Chapter one consists of general introduction along with a brief of the problem background, aims and objectives of the study. Chapter two deals with methodology of the research work which contains the suitable testing method and reasons for selecting the field & laboratory testing methods, selection criteria and methods for determinations of all design input parameters. Chapter three consists of review of the availability of soils and materials and their properties. Chapter four described the existing correlations of the properties of materials found from literature and also discussed suitability of particular one for use in Bangladesh conditions. Chapter five describes the review of field and laboratory testing techniques, their advantages, disadvantages and limitation. Chapter six deals with the review of site investigation composed of available methods of ground explorations, sampling and specifications to be followed for the investigation. Chapter seven describes the testing programmes and procedure for different field and laboratory testing based on the criteria noted in methodology. Chapter eight, nine and ten summarises the test results, analysis and discussions made on embankment soils, both subgrade & improved subgrade and granular & bituminous materials respectively. Finally, chapter eleven deals with the conclusion and recommendations for future studies.

1.6 Summary

In Bangladesh many roads are constructed on embankments. However, there is a lack of procedures and specifications for construction of embankment although main information is unavailable for road pavements. In addition to this, although much information exists from the design and construction of roads it is not used to make a database that may be used for future projects. In addition to this although many correlations exist, it is not clear which are more suitable for soils of Bangladesh. In order to overcome these drawbacks, this study is designed to explore availability of existing information’s and its usefulness, where there is a lack of information, ground information was gathered to agreement existing knowledge and made good gaps.

Chapter 2: Methodology of the Research

2.1 Introduction

Various field and laboratory testing methods are available for evaluating the properties of soils associated with the design of road pavements and embankments. Although it is possible to identify the most suitable tests and number of tests required for ascertaining the required properties, it is generally necessary to make compromises based on availability of resources (testing equipment, skilled technicians, funding and time amongst many others). Thus it may be necessary to conduct index tests such that required properties are obtained through correlations. Alternatively only a limited number of tests can be done. In such cases, correctness of the results should be considered with respect to published works and correlations that are most appropriate for Bangladesh.

This chapter also describes the parameters required for the design of pavement and embankments in Bangladesh. The parameters related to embankment and pavement design were identified in a parallel study undertaken by Bhattacharyya (2009) and this study respectively. Furthermore, a methodology is given by which suitable field and laboratory tests can be selected to determine the required parameters. Finally some correlations will be developed for Bangladesh soils and non conventional granular materials that can be used for future RHD road and embankment design project.

2.2 Parameters required for design

One of the purposes of this study was to identify suite of tests which may be carried out in Bangladesh to determine parameters required for the analytical design of the road pavement and the embankment. The input parameters, which have been identified for pavement and embankment design, are summarized in Table 2.1.

Table 2.1: Summarization of input data & respective test for pavement and embankment design:

Name of input parameter for design	**Description & purposes of the parameter**	**Name of the test**	**Appropriate standard (BS /ASTM/ others)**	**Pavement or embankment or both.**
Liquid limit (w_L), Plastic Limit (w_P),	Index parameters of the soils, used in soil classifications and estimation of some engineering properties of clay soils.	Atterberg limit test with Casagrande's apparatus (lab. test)	BS 1377-2:1990 or ASTM D 4318 – 84	Stability and settlement analysis (determination of Cu, Cc, Cs etc) of embankment design.
Moisture content (w)	The measurement of the loss of weight of a soil sample after oven drying to constant weight at a temperature of 105 to 110 ^{0} C. and used in many soil tests	Moisture content test. (lab. test)	BS 1377-2:1990	Stability analysis of Embankment design specially used to determine the liquidity index and hence strength.
Organic content test	Organic matter is destroyed by loss on ignition. Used in preparation of soil for sedimentation test and also gives an indication of amount of organic matter present.	Loss on ignition (lab. test)	BS 1377- 3 :1990 or ASTM : D 2974 - 87	Stability and settlement analysis of Embankment design, as strength and settlement depends on organic content of soil.
Density(γ), dry density (γ_d)and OMC	Used for determining the relative compaction of soils.	Proctor test	BS1377-4:1990	Used in stability analysis of embankment design.
Specific gravity (Gs) of soil	Used in many soil problem computations.	Specific gravity test (lab test)	BS 1377 – 2:1990, Clause 8	Used in settlement analysis (Initial void ratio determination) of embankment design
Particle size distribution	Used to determine the percent of gravel, sand, silt and clay particles in a soil sample and also used to know the type of distribution of various sizes.	Particle size distribution test (sieve analysis & Hydrometer analysis)	BS1377-2:1990, Clause 9	Used in the classification of soil for embankment design.
Undrained shear strength (Cu)	Shear strength determined from undisturbed soil sample at undrained conditions.	1. Triaxial test (UU) 2. UCS test 3. Vane shear 4. Penetrometer test	1. BS1377-7:1990 2. BS1377-7:1990 3. BS1377-7:1990	Used in the stability & settlement analysis of embankment design.
Undrained modulus (Eu)	Determined from the stress-strain relationship of lab. Triaxial test.	1.Triaxial test	BS1377-7:1990	Used in the settlement analysis of embankment design.

Continued..

Name of input parameter for design	Description & purposes of the parameter	Name of the test	Appropriate standard (BS /ASTM/ others)	Pavement or embankment or both.
Angle of internal friction (ϕ) (Undrained)	Determined from undisturbed soil sample by quick undrained triaxial test.	1.Triaxial test (UU)	BS1377-7:1990	Used in the stability and settlement analysis of embankment design.
Peak & Residual Drained Cohesion (c') and angle of internal friction (ϕ ')	These effective stress parameters determined by laboratory test.	1. Direct shear box test 2. Ring shear test 3. Consolidated drained and consolidated undrained (with pore pressure measurement) triaxial test.	1.BS1377-7:1990, Sec -4 2.BS1377-7:1990, Sec -6 3. BS 1377-8:1990	Used in the stability and settlement analysis of embankment design.
Compression index (Cc)	The slope of the e – log p curve and used in the analysis of settlement.	1.Consolidation test 2.Correlations	BS1377-5:1990	Used in the estimation of total settlement of embankment design.
Secondary compression index (Cα)	The slope of the straight line portion of the e – logt curve and used in the analysis of settlement. Most useful if the soil is organic.	1.Consolidation test 2.Correlations	BS1377-5:1990	Used in the total settlement calculation of embankment design where soils are susceptible to secondary consolidation.
Initial void ratio (e_o)	The ratio between volume of void to solid at the starting of the test and used in the calculation of compression index.	1. Relationship among unit wt, void ratio, moisture content & specific gravity.	BS1377-5:1990	Used in the settlement analysis of embankment design.
Coefficient of consolidation (C_v)	Used to estimate the amount of settlement for a given time period under a given load.	1.Consolidation test	BS1377-5:1990	Used in the settlement analysis of embankment design.
Coefficient of volume compressibility (M_v)	It depends on the pressure increments adopted and not a true soil property.	1.Consolidation test	BS1377-5:1990	Used in the settlement analysis of embankment design.
Recompression index (Cr)	It is the slope of the recompression portion of the e- log p curve.	1.Consolidation test	BS1377-5:1990	Used in the settlement analysis of embankment design.
Swell index (C_s)	To know the swell shrink behavior of clayey soil.	1.Consolidation test	BS1377-5:1990	Used in the settlement analysis of embankment design.

Continued..

Name of input parameter for design	Description & purposes of the parameter	Name of the test	Appropriate standard (BS /ASTM/ others)	Pavement or embankment or both.
Permeability (K)	Access ability of materials to transmit water.	1.Consolidation test	BS1377-5:1990	Embankment settlement design
CBR value	CBR value of the subgrade soil and other granular layers are used in the design of pavements for roads and airfields. The value may be obtained from DCP test from the field with empirical correlation and from laboratory test. CBR value at different moisture content is required to establish relationship between them.	1.DCP test and correlation 2. lab. Soaked CBR.	1.STP -5.2 of RHD Specifications 2. BS 1377 – 4: 1990. 3. ASTM D 1883 - 99	Pavement layers thickness design
Resilient Modulus (M_r)	Resilient modulus is used for analytical pavement design. It can be obtained from CBR value with correlations or from cyclic triaxial test.	1. Cyclic Triaxial test. 2.Correlations	AASHTO T307	Pavement layer thickness design
Particle size distribution of aggregate	Gradation of granular materials is closely related to stability & engineering properties.	Sieve analysis	BS1377-2:1990,	Used in pavement granular layers thickness design.
Elastic modulus of bituminous layer	Used in the design of bituminous layer thickness.	1.Using van der Poel & Shell chart 2.Witczak model	-	Used to determine the thickness design of bituminous layer.
Bitumen content of the bituminous mixtures	The properties of the bituminous mixes like durability, compactibility, rutting, bleeding and raveling are controlled by the quantity of bitumen in the mixes.	Bitumen extraction test	STP 10.4 of RHD specification	Used in the design and analysis of bituminous mixes.
Grades of bitumen	Determination of grade of bitumen and indirect determination of high tempt. Viscosity and low tempt. stiffness	Bitumen Penetration test	ASTM D 5	Used in selection of bituminous materials as per specifications.
Viscosity of bitumen	Viscosity of bitumen is used to determine its stiffness modulus.	Viscosity test of bitumen	ASTM D 2170	Used in the mix design of bituminous materials.

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Name of input parameter for design	Description & purposes of the parameter	Name of the test	Appropriate standard (BS /ASTM/ others)	Pavement or embankment or both.
Bitumen properties test like ductility, softening point, loss of heating etc	These are the routine laboratory test of bituminous materials & gives idea about the properties of bitumen.	Ductility test, Ring & ball test, Loss on heating test.	ASTM D 113, D 36, D6 – 96(2006)	Used in the mix design of bituminous materials.
Particle size distribution of aggregates	The aggregate gradation is used as per required specifications	Sieving test	BS1377-2:1990,	Used in the mix design of bituminous materials.
[2]ACV & [3]TFV of aggregates	Strength test of aggregates	ACV & TFV test	BS812-110:1990, BS812-111:1990	Used in selecting the aggregate for mix design.
Marshall stability and flow	Optimum bitumen content is determined	Marshall test	ASTM D 1559	Used for the design and control of the bituminous mixes
Thickness of pavement layer	Thickness of different layers is required to determine the elastic modulus from back calculation.	Coring test	-	Pavement design
Pavement deflection	Total deflection of the pavement layers is determined by Benkelman beam test.	Benkelman beam	AASHTO T - 256	Used in pavement design, analysis & strengthening of Pavement.

For each of the parameters given in Table 2.1 there are a number of procedures which could be used in order to determine the required properties. Available procedures were evaluated according to the criteria given in Table 2.2. This was done by attributing a score for each procedure to identify the most suitable one and given in Appendix G, Table A- 2.1 to 2.6. Some design [4]parameters can be determined by test and or using correlations.

[2] ACV means Aggregate Crushing Value
[3] TFV means Ten percent Fines Value
[4] Parameters mean materials properties used as design input.

Table 2.2: Weightage and Priority range of different criteria

Criteria consideration	Applications & drawbacks	Weighting (%)	Priority[5] range
Accuracy	The parameters evaluated from the test should be more accurate which represent the actual field conditions. However it is not always possible to conduct field test due to the lack of equipment, its availability or expertise to undertake the test. Any design depends on the accuracy of the data. So accuracy should be given in highest weightage.	100	0 – 5
Availability of the equipment	In Bangladesh, there is lack of both field and laboratory equipment. Very basic equipment mainly for classification tests are available in RHD owned Bangladesh Road Research Laboratory (BRRL). For example, no test apparatus is available for determining insitu CBR, cyclic load triaxial test to ascertain the resilient modulus. There is very limited availability of equipment (one apparatus) is available for undertaking effective stress parameters of soils. Thus where suitable equipment is available for conducting the most suitable test, highest weighting is awarded.	100	0 – 5
Available expertise for carrying out the test	Some tests are ease to perform and some are more sophisticated. If the technical personnel are experienced in doing the test, it will take less time and resources. In this case weightage should be based on use of suitably trained technicians. Weighting should neither too low nor higher say medium percentage. Since regardless of the quality of the test equipment of the suitable staff has no experience, the test apparatus is pretty more uses.	50	0 – 5
Cost	Since execution of test is associated with cost, so it is desirable to do the cheapest test that gives the required engineering properties. Optimum utilization of existing equipments should be made for minimizing the cost for the project. So weightage should be given in accordance with the economy of the test. The cheapest test should be awarded cent percent weightage. However, considering	75	0 – 5

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[5] Priority range depends on the test methods and conditions and the values used in this investigation; see Appendix G, table A – 2.1 to 2.6 for example of total score using priority.

	the suitability of the test, cost is not always prime factors. So weighting should be in between medium and cent percent.		
Ease of use	It was felt that the equipment should be easy to use such that low level expertise is required. In addition to this, easy of repair (few unsophisticated parts) and portability are important and desirable facts. Considering this, medium weighting is awarded.	50	0 - 5
Training in use of the equipment	Some equipment is more sophisticated and requires considerable training to raise staff competence in using equipment. For example, a sophisticated CU triaxial apparatus with pore pressure measurement is available at DUET, Bangladesh; but it is fully unutilized due to the lack of training of technical staff. So, training is the subject of considerations for selecting the test and must be awarded to the highest weighting.	100	0 - 5

In the above table priority ranges from zero for no priority to five for highest priority. This priority should be awarded based on criteria of each tests.

For each parameter, the procedures identified from the literature noted in chapter 3, 4, 5 and 6 are described below under the following headings: field investigation, field tests, laboratory tests and correlations.

2.3 Field investigations and sample collections

Criteria for the selection of the properties of the pavements are discussed in chapter six: review of site investigation. For details laboratory investigations some trial pits were excavated manually to collect granular materials as well as subgrade soils. Moreover, bituminous layer thickness and both strength and stability of bituminous materials were evaluated from core of bituminous layer which were extracted by coring. Meanwhile, subsurface investigation was undertaken for the determination of the properties of soil. Hence boring and samples collection for selected road embankment were done as per selection criteria given in the Appendix G, Table A - 2.1. From that criterion, hand auger boring was used for recovering samples from different depths for exploration of highway embankment in Bangladesh. Soil samples collected were slightly disturbed. On the other hand, two other methods are available for soil sample collections such as cable percussion boring and wash boring. In case of wash boring, finer particles were washed away with the water and moisture content may not be constant and cable percussion boring equipment was not available in BRRL. Considering accuracy and availability, hand auger equipment was used to collect soil samples from the site.

2.4 Field tests

Very limited pavement field testing equipments is available in Bangladesh. The Falling Weight Deflectometer (FWD) is the most reliable and sophisticated equipment that can measure the pavement deflection in the field. However, it was not available in Bangladesh. So, considering the availability of the equipments, suitable field tests were carried out. According to the scoring criteria shown in Appendix G, Table A-2.2, the Dynamic Cone Penetrometer test (DCP) is the preferable field test for estimating the CBR value using suitable correlations. Another field test for measuring the compaction of the different layers of pavement as well as embankment earthworks is the insitu density test which is widely used to control the field compaction. There are a number of methods available for measuring the insitu density but most common methods is the sand replacement method that is selected in accordance with the criteria given in Appendix G, Table A- 2.3.

Pavement deflection can be measured by three methods. According to the criteria given in Appendix G, Table A-2.4, Benkelman beam test is the most suitable method for measuring the deflection of the pavement and details are discussed in chapter seven.

2.5 Laboratory tests

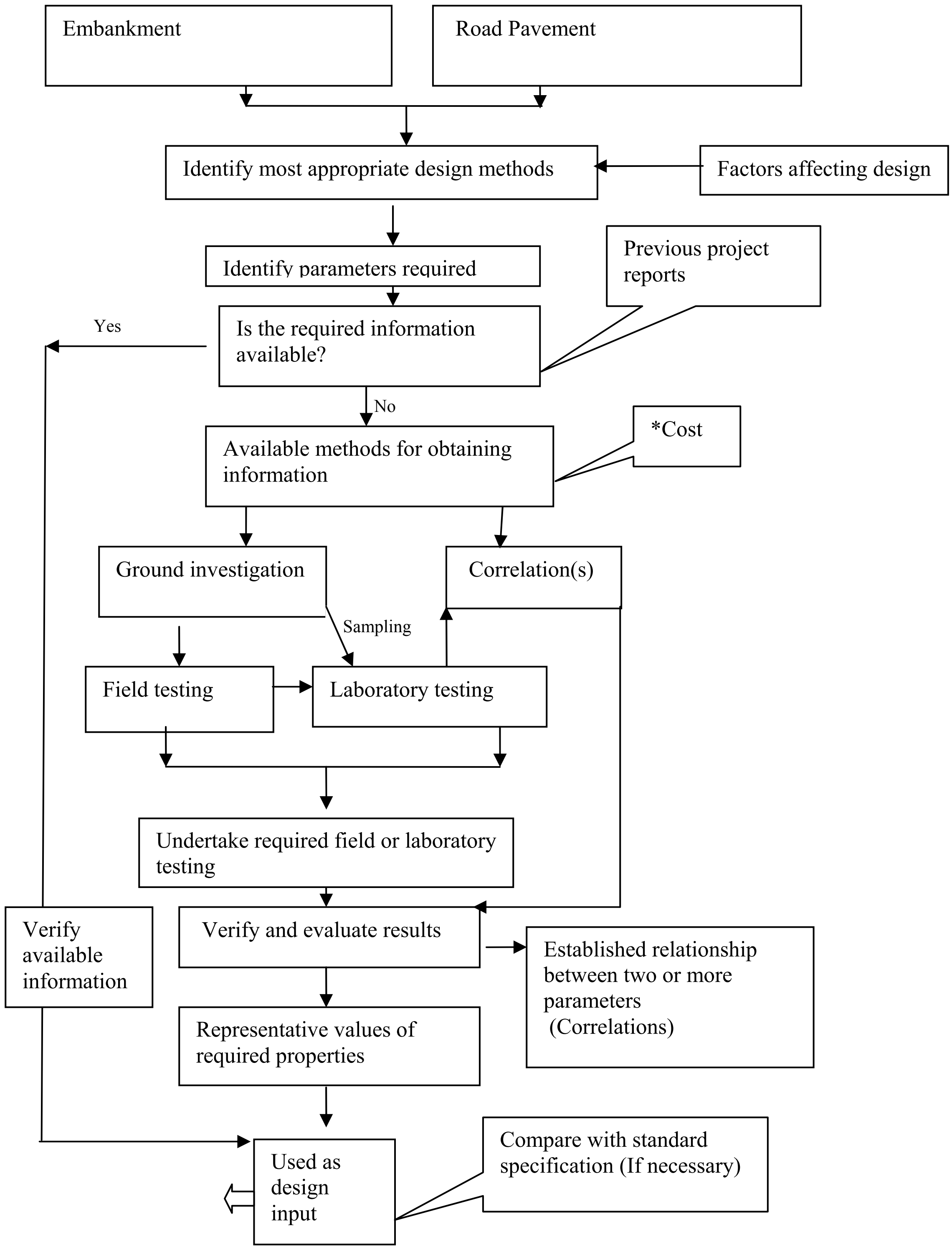
The index parameters of the soil such as Atterberg limit (Liquid limit and Plastic limit), unit weight, moisture content, and organic content were determined. The particle size distribution test was done with the help of hydrometer and sieve analysis method. The undrained shear strength parameter like undrained shear strength can be found from the quick undrained triaxial (UU) test, unconfined compressive strength (UCS) test, vane shear test, penetration test. But the most suitable test is selected based on the criteria given in Appendix G, Table A- 2.5. Besides this, some simple tests were done to compare the test results and to correlate the properties. The UCS test was done for undisturbed soil samples collected from different boreholes at different depths.

The drained shear parameter such as drained cohesion and angle of internal friction can be determined by drained shear box test, consolidated drained triaxial test and consolidated undrained (CU) triaxial test with pore pressure measurement. So, considering the availability of the equipment direct shear box test was selected based on the selection criteria given in Appendix G, Table A- 2.6 and hence that test was conducted. Besides this, the residual strength parameters were determined by ring shear test. For determination of settlement parameters, oedometer test is inevitable for evaluating these design parameters. So, oedometer is conducted for determination of the compressibility behavior of soil samples. Besides this, there are some empirical equations available and these can be used to compare the parameter that was found from the test. The cyclic triaxial test was performed for determining the resilient modulus of subgrade soils (silty clay of Bangladesh), improved subgrade materials (medium sand) and sand mica mixture. Besides these, the test was also conducted on non conventional brick aggregates used in base layer and mixture of brick aggregate with sand as per specified gradation used in subbase layer of pavement in Bangladesh. Resilient modulus of subgrade soils as well as granular materials will be used in analytical pavement design in Bangladesh.

2.6 Correlations

Correlations for different properties are described in the chapter four. However, there are still doubts about applicability of some correlations. So there will be a need to ascertain design values from simple tests. For examples there are lots of correlations are available for determining the CBR value from DCP value but the most appropriate correlation will be used that are derived for similar materials likely to be found in Bangladesh.

2.7 Flow chart for the Research Work of the Project



*cost: it may be necessary to re- evaluate the scope of the investigation if the costs are too high. On end it may need to be reconsidered.

2.8 Summary

Evaluation of existing embankment and pavements were done by appropriate field and laboratory tests as selected by the criteria given above. The available soils and granular materials properties are discussed in the following chapter three. Besides these, some of the engineering properties can be estimated by correlations discussed in chapter four. Moreover, these properties are determined by appropriate testing methods. Some of these tests are laboratory based, some are field based and others use correlations. Finally, testing methods will be recommended for evaluating the design input parameters used in pavement and embankment design for Bangladesh.

Chapter 3: Review of the available Soils & Materials and their Properties

3.1 Introduction

The availability of soils, granular and bituminous materials found in Bangladesh and their properties are discussed in this chapter. It also deals with the available information about index properties, shear strength properties and settlement behaviour of fills and subsoil. This includes strength and stiffness properties of subgrade soils; granular as well as bituminous materials that are used in pavement construction are also discussed in this chapter. The engineering properties of soil are the crucial factors for the design of embankment and similarly the properties of granular materials (crushed bricks) are also important for analytical pavement design process.

3.2 Soils of Bangladesh

So far many researchers such as Morgan and McIntire (1959), Bramer (1971), Hunt (1976), Master Plan Organization (1986), Road materials and standard study (RMSS, 1994), Ameen (1985), Bashar (2000) and Islam (1999) had investigated geological characteristics of Bangladesh soil for many years.

Most of Bangladesh is an extremely flat delta area which consists of a large alluvial basin floored primarily with quaternary sediments deposited by the Ganges – Padma, the Brahmaputra – Jamuna and the Meghna river systems and their numerous tributaries and distributaries. The north-eastern and eastern boundaries of country follow the mountainous area in India and Myanmar. The alluvial deposits of the country have varying characteristics ranging firm piedmont deposits near the mountain to swamp and deltaic deposits near the southern sea- shore (Morgan and McIntire, 1959). Several studies (RMMS, 1994; Kabir et al. 1997; Uddin, 2001; Ferthous, 2007) were conducted to evaluate the properties of soils of Bangladesh. RMSS is one of the most important studies that were done in 1994 under the financial assistance of European Economic Community (Humphrey et al., 1994). RMSS study divided the whole country into six zones that are shown in Figure 3.1.The most of the soils are silty clay, clayey silt, silty fine sand, dark grey to brownish clay with organic fragments, dark grey to black silty clay & peat and organic rich silty clay. The RMSS study is quite worse and misses out the fact that in parts of the delta, ground is very swampy and in some areas soils described as black cotton soils exist. The black cotton soils contain montmorillonite and thus exhibit extreme shrinkage and swell characteristics. Serajuddin & Ahmed (1982) have indicated that upper soil strata in many areas of Bangladesh up to the depth of 6 to 7 m, are silts and clay of low to medium plasticity. Serajuddin and Azmal (1991) have observed the upper strata soils of about 2 to 3m depths and found silty and clayey soils.

Figure 3.1: Soil classification map of different RMSS zone Bangladesh (Humphrey, 1994)

3.3 Soil Properties

According to Terzaghi (1936), "*Soils are made by nature and not by man, and the products of nature are always complex.*"

So characteristics of soils are inherently complex and highly variable. However, according to Graham (1988) the properties of soil to be measured for a project should be within the four categories such as classifications of soils, identifications of strength parameters, identifications of compressibility or stiffness and permeability parameters.

A soil's characteristics are based on the particle size grading of the coarser particles and the plasticity of the finer particles that plays a major role in determining the engineering properties of the soil. For sustainable design and construction of embankment, proper characterization of both fill (soils from which embankment is constructed) and foundation soils are necessary. The most common soil properties such as index properties, shear strength properties and compressibility properties found from literature are presented in the following subsections.

3.3.1 Index properties

The Atterberg limits, natural moisture content, specific gravity, particle size distribution and organic content are the index properties of soils. These properties are discussed in the following subsections.

3.3.1.1 Atterberg limits

Atterberg limits are used in most soil classifications system i.e. to describe the soil and its name. Depending on the moisture content, the behaviour of soils can be divided into four basic states – solid state, semisolid state, plastic and liquid state. The limiting water content among these states is called shrinkage limit (w_S), plastic limit (w_P) and liquid limit (w_L) respectively. The Atterberg limits are widely used in the classification of fine grained soils. A cohesionless soil (sand) has zero plasticity index is called the non-plastic. Clays are highly plastic and possess high plasticity index. The lower the plasticity index, the higher the bearing capacity of soil (Das, 1997). These index properties of soft, organic and silty clay were determined for Bangladesh soils by Hossain & Rahman (2005) as reported by Ferthous, 2007 are presented in Table 3.1.

The table shows the moisture content of organic layer remains in the range of 89% to 370% while liquid limit and plasticity index range from 80 % to 352 % and 24 % to 181 % respectively. Besides these, the liquid limit and plastic limit of Montmorillonite clay mineral may vary 100 % to 900% and 50% to 100% respectively (Mitchell, 1993). However RMSS recommended that the soils having liquid limit exceeding 70% or Plasticity Index exceeding 40% should be rejected as an earth fill materials for embankment. The reasons for rejection of such soil are that they contain Montmorillonite clay mineral and hence exhibit high swelling & shrinkage properties.

Another index property of soils is liquidity index which can be defined as the relative consistency of a cohesive soil in the natural state by a ratio as follows:

$$I_L = \frac{w - w_P}{w_L - w_P}$$

If the value of I_L is zero and one, the natural water content will be equal to the plastic limit and liquid limit of the soil respectively. Although these properties have been determined for different project in RHD of Bangladesh but lack of proper recording system, data are unavailable in archives.

Table 3.1: Atterberg limits with respect to depth for Bangladesh soils (after Hossain & Rahman, 2005)

Location	Soil type	Depth (m)	w_n(%)	w_L (%)	w_P (%)	I_P (%)
South West part of Bangladesh	Soft clay	1.50	42	82	57	25
	Organic soil	3.05	89	352	171	181
		4.57	370	87	55	32
	Silty clay	9.14	45	32	25	7
		10.67	44	31	22	9
		12.19	44	37	28	9
		13.72	46	40	31	9
	Soft clay	1.52	41	53	26	27
		9.15	53	56	40	16
	Organic soil	3.05	107	80	56	24
	Silty clay	17	68	57	39	18

3.3.1.2 Specific Gravity:

The specific gravity of soils is used for various calculations in soil mechanics and can be determined in the laboratory using tests outline described in ASTM D792 and BS1377. Bowles (1997) reported that specific gravity of the inorganic clay (CL, CH, CH / CL and CL- ML) and inorganic silt (ML, MH, ML / CL) may vary from 2.68 to 2.75 and from 2.60 to 2.68 respectively. Bowles (1978) also reported that specific gravity of organic clay is variable but may be below 2.0. Ferthous (2007) has evaluated the specific gravity of Bangladesh soils ranges from 2.58 to 2.86 and 1.61 to 2.29 for inorganic (silt & clay) and organic soils respectively.

3.3.1.3 Natural Moisture Content:

Moisture content is one of the important parameter for a soil that plays a pivotal role on its properties including permeability, strength and settlement. In Bangladesh, most of the soil samples are collected by wash boring method and in this method the moisture content of soils is influenced by water and hence moisture content as reported by some researchers in Bangladesh may not represent the actual moisture content. The moisture content of different clayey soils in Bangladesh is also reported in Table 3.1. So the moisture content of organic soils ranges from 89% to 370% (Humphrey et al. 1994) and inorganic soft clay and silty clay soils ranges from 42% to 68% (Hossain, 2005).

3.3.1.4 Particle Size Distribution:

Particle size distribution of a soil has a major bearing on its properties. In classifying soils, particle sizes in clay, silt, sand and gravel are determined. This shape of the distribution curve can be used to decide if the soils are uniformly graded, well graded, poorly graded and gap graded particles (Das, 1997). It is possible to estimate permeability and particle packing properties from the nature of the particle size distribution. Various indices can also be used to estimate the nature of the packing of particles. These are coefficient of uniformity (C_u) and the coefficient of curvature (C_z), defined as follows:

C_u =[6]D_{60}/[7]D_{10} & C_z = [8]D^2_{30}/ $D_{60}D_{10}$.

Well graded soils have $C_u > 3$, greater the value better the grading (Murthy, 2003). If the value of C_u is < 3, soils are uniformly graded. Ideal grading for best packing resulting in high strength, low permeability follow Fullers curve that shows various coefficient of permeability (k) curve in gradation chart.

The particle size distribution test has done by Bangladesh Road Research Laboratory (BRRL) during Feb/2007 on the embankment of Tongi – Ashulia – EPZ Road (N302) of Bangladesh. Soil samples were collected from eleven boreholes up to the depth of 24

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[6] D_{60} = Particles diameter at which 60% is finer
[7] D_{10} = Particles diameter at which 10% is finer
[8] D_{30} = Particles diameter at which 30% is finer

meters from the ground level. The percentage of sand, silt and clay in the soil masses for three boreholes are given in Table 3.2. Table shows that most of the strata below 15 m depth are sandy soils.

Table 3.2: Particle Size Distribution of soil samples collected from Tongi – Ashulia – EPZ road (BRRL, 2007)

Depth (m) from [9]EGL	Borehole at Ch. 6+ 192.5			Borehole at Ch. 6+ 195			Borehole at Ch. 6+ 280		
	Sand (%)	Silt (%)	Clay (%)	Sand (%)	Silt (%)	Clay (%)	Sand (%)	Silt (%)	Clay (%)
3	2	76	22	12	74	14	11	59	30
6	31	39	30	4	74	22	15	61	24
9	50	30	20	9	74	17	15	61	24
12	8	91	1	38	54	8	15	61	24
15	87	13	0	52	36	12	61	29	10
18	87	13	0	81	19	0	97	3	0
21	95	5	0	81	19	0	97	3	0
24	92	8	0	92	8	0	97	3	0

3.3.1.5 Organic Content

The presence of organic matter in a soil results increase in its high plasticity, high compressibility and makes the soils more susceptible to shrinkage and swelling, lower the strength and increases its hydraulic conductivity (Santagata et al., 2008). Soil containing significant amount of decomposed organic matter may change its colour to dark gray or black.

In accordance with BS EN ISO 14688 – 2:2004, soils were classified based on the percentage of organic content is shown in Table 3.3.

Table 3.3: Classification of soils with organic constituents (BS EN ISO 14688 - 2:2004)

Soil	Organic content (< 2mm) % of dry mass
Low - organic	2 to 6
Medium – organic	6 to 20
High - organic	> 20

[9] EGL means existing ground level.

RMSS recommended that soils from swamps, marshes having organic content more than 12% when tested with Dichromate Oxidation Method (BS 1377:1975, Test 8), should be rejected for filling materials for embankment construction. However, Ferthous (2007) reported that the percentage of organic content in Bangladesh (south west part) ranges from 13% to 43%. Munshi (2003) and Islam et al. (2003) reported that organic content of Bangladesh soils at a depth of up to 12 m ranges from 5 % to 30% and 0% to 30% respectively. So as per British Standard classification, these soils are classified as medium to high organic soils. Presence of significant amount of semi or fully decomposed organic matter in a soil sample has high water content, high void ratio, high compressibility and low shearing strength (Ferthous, 2007).

3.3.2 Shear strength properties

Shear strength of a soil is used in bearing capacity, slope stability, retaining wall design and pile design. In most of the cases Mohr Coulomb equation defined as $\tau = C + \sigma \tan \phi$ (where τ is the shear strength, C is the cohesion, ϕ is the angle of internal friction and б is the normal strength) is used to estimate the shear strength of soils. Different values of shear strength parameters depending on the type of soils determined by the different authors are reported. Among them, BS EN ISO14688-2:2004 suggested some typical value of undrained shear strength for clayey soils is given in Table 3.4. Ferthous (2007) and Serajuddin (1998) have worked on Bangladesh cohesive soils found that the undrained shear strength ranged from 11 kPa to 60 kPa and 1 to 83 kPa at the depth between 3.5 to 9.5 m in the south west and south east zone of Bangladesh respectively. Again Das (1985) reported the drained angle of friction for sand and silts are ranged from 27^o to 30^o and 26^o to 35^o respectively. Murthy (2003) reported that the drained angle of friction for clay of low to medium plasticity is 22 to 28 degree. Moreover, Peck et al. (1974) has suggested the drained angle of friction for loose silts ranges from 29 to 30 degree.

3.3.3 Undrained Modulus

The undrained modulus (Eu) is one of the embankment design parameter. This parameter can be determined in the laboratory by triaxial test. This value for soils usually determined as secant modulus between a deviator stresses of 0 and 1/2 to 1/3 peak deviator stress in the triaxial test (Lambe & Whitman, 1969). Kabir et al. (1992) have determined the values of Eu for Bangladesh sand and clay that are 50 MPa and 3 MPa respectively.

3.3.4 Consolidation Properties

The embankment is constructed on ground that is known as foundation of the embankment. Most of the roads in Bangladesh are constructed on high embankment. As spelled out earlier most of the subsoil in Bangladesh is silty clay, clayey silt, sandy silty clay and expansive soil such as black cotton soil. When the embankment is constructed on that kind of cohesive soils, behave in an undrained manner with very little consolidation occurring during construction and the major portion of the consolidation settlements occurs after the end of the construction resulting failure of the embankment as well as pavement (BS 6031:1981). The properties of consolidation depend on the types of the soils and overburden pressure. The consolidation properties used in the estimation of primary consolidation settlement for [10]fine grained soils of Bangladesh are presented in Table 3.5 (Serajuddin, 1998). The properties are discussed in details in the following subsections. At early stage, Terzaghi & Peck (1967) suggested that the liquid limit and soil compressibility is directly proportional.

Table 3.4: Undrained Shear Strength of fine soils (BS EN ISO 14688 – 2:2004).

Undrained shear strength of clays	Undrained shear strength, Cu (kPa)
Extremely low	<10
Very low	10 to 20
Low	20 to 40
Medium	40 to 75
High	75 to 150
Very high	150 to 300
Extremely high	> 300
Materials with shear strength greater than 300 may behave as weak rocks and should be described as rocks according to ISO 14689 – 1.	

[10] Fine grained soils means silty and clayey soils

Table 3.5: Primary consolidation parameters of fine grained soils (Serajuddin, 1998)

Location	Depth (m)	USCS	W_n (%)	e_o	C_c	Cv (m^2 / yr)
Bangladesh	3.5 – 9.5	CL,ML,CH	24 - 47	0.706 – 1.32	0.080 – 0.52	1.73 – 100.91

3.3.4.1 Compression Index (C_c):

It is the principal values obtained from the consolidation test and is calculated from test data. The slope of the linear portion of the e- log p curve (field curve) is designated as the compression index (C_c). It is a dimensionless parameter used to analyse the total settlement. There are numerous empirical equations available for determinations of Cc are spelled out in chapter four.

The typical values of Cc for silty clay and clay soils are normally ranged from 0.1 to 0.3 and 0.2 to 1.0 respectively (O' Flaherty, C.A, 1991). The value of compression index in saturated clays varies from 0.1 to 0.5 depending on their plasticity characteristics and the value increases with increasing plasticity (Aysen, 2002). Besides these, in organic soils and peat, the value of compression index may be 3. In some cases such as Mexico City clay, its value is almost 10 (Mesri et al., 1975). The value of Cc for Bangladesh soils (silty clay) determined by Aminullah (2004) ranges from 0.11 to 0.43.

3.3.4.2 Recompression Index (C_r):

During consolidation test, unloading and reloading are done to determine the swell and recompression behaviour of the soils respectively. The slope of the recompression portion of e- log p curve is known as recompression index (Cr). So it depends on the types of the soils as well as loading. Leonard's (1976) has suggested some typical values of recompression index which are within in the range of 0.015 to 0.035 and also mentioned that its value decreases with the decrease in plasticity. According to Aysen (2002), disturbed samples exhibit high values of recompression index. However, a reasonable value of recompression and compression indices may be found if unloading and reloading cycles for void ratios less than 0.42 e_o (Schmertmann, 1953). In overconsolidated clays, the recompression index is smaller than compression index (Das, 1997).

3.3.4.3 Secondary Compression Index ($C\alpha$):

In some soil types continuing settlement have been observed after complete dissipation of excess pore water pressure i.e. at the end of the primary consolidation. This is attributed to the plastic adjustment of the soil particles. Secondary consolidation characteristics were determined from conventional laboratory consolidation test to provide the assessment of secondary settlement.

The secondary compression index is significantly affected by organic content especially peat, clay mineralogy as represented by high plasticity and metastable mineral grain structures as represented by sensitive clays (Mesri, 1973). Hence Das (1985) has mentioned that the determination of secondary settlement is more important than primary consolidation for organic and highly compressible inorganic soils. In overconsolidated inorganic clays this value is very small and of practically insignificant. However, some typical values of the secondary compression index C_α suggested by Lambe & Whitman (1979) and Cernica (1995) are given in Table 3.6.

Mesri and Godlewski (1977) reported that the ratio of secondary and primary compression index lies in between 0.025 to 0.1 and average value is 0.05(Mesri, 1973). The ratio has been derived for 12 different types of clay such as Maxico clay, New Zealand clay, Norwegian clay, Boston & Chicago blue clay, organic clay, organic silty clay and silty. But Tarzaghi (1996) reported that for all geotechnical materials this ratio ranges from 0.01 to 0.07 and the most common value for inorganic clays and silts is 0.04.

Table 3.6: Typical values of secondary compression index (C_α)

Soil type	$C\alpha$ (Lambe and Whitman, 1979)	$C\alpha$ (Cernica, 1995)
Normally consolidated clays	0.005 – 0.02	0.005 – 0.03
Very plastic clays	0.03 or higher	-
Organic clays	0.03 or higher	0.04 – 0.1
Overconsolidated clays	Less than 0.001	0.0005 – 0.0015

3.3.4.4 Swelling characteristics of soil (Cs):

The direct determinations of swelling characteristics such as swelling pressure, swell potential and swelling index require considerable time and effort. Several investigators have developed correlations to predict the swelling characteristics reported in chapter four are used to estimate not only the swelling characteristics but also used to cross checking on field and laboratory testing conducted on such kinds of soils. Some typical values of swelling index for Bangladesh soils (silty clay) range 0.007 to 0.049 (Aminullah, 2004).

The swelling characteristics of expansive soils are decreased with the increased of coarse fractions (sand) based on correlation (Rao, Babu and Rani, 2006).

3.3.4.5 The Coefficient of Consolidation (Cv):

It is one of the principal values obtained from the consolidation test. It is used to estimate the amount of settlement for a given period of time under a given load. Some typical values of coefficient of consolidation for different types of soils suggested by different researchers are shown in Table 3.7. Besides this, some typical values of Cv for silty clay and clayey soils are also shown in that figure. This value were determined according to the soil mineralogy are also reported in Table 3.7. Table shows that kaolinite soil mineral have high coefficient of consolidation than other soil mineral. The coefficient of consolidation can be determined by means of laboratory test by plotting settlement curves on two types of graphs.

1. Casagrande's logarithm of time vs settlement
2. Taylor Square root of time vs settlement.

Both methods may be used for determination of time for degree of consolidation and hence calculation of Cv. According to Muni Budh (2007) the log time method makes use of the early (primary consolidation) and later time responses (secondary compression) while the root time method only utilizes the early time response which is expected to be a straight line. In theory, the root time method give good results except when nonlinearities arising from secondary compression cause substantial deviations from the expected straight line. These deviations are most pronounced in fine – grained soils with organic materials.

3.3.4.6 The Coefficient of Volume Compressibility (Mv):

The volume change per unit volume per unit increase in effective stress is known as the coefficient of volume compressibility. The value of M_v for a particular soil is not constant but depends on the stress range over which it is calculated. The British standard (1377:1990) specifies the use of the coefficient Mv calculated for a stress increment of 100 KPa in excess of the effective overburden pressure of the insitu soil at the depth of interest, although the coefficient may also be calculated, if required, for any other stress range. However, some typical values of Mv suggested by Barnes, (2001) for different types of soil are given in Table 3.8. Table shows that the value ranges from 0.05 m^2 / MN to 0.5 m^2 / MN for very stiff heavily overconsolidated clay to soft organic clay respectively.

Table 3.7: Some typical values of coefficient of consolidation (Cv) for different types of soils.

Soil types	The value of Cv		Name of Researchers
	cm^2 / sec (x10E-4)	m^2 / yr	
Boston blue clay (CL)	40 ± 20	12 ± 6	Ladd and Luscher, 1965
Organic silt (OH)	2 – 10	0.60 – 3	Lowe, Zaccheo and Feldman, 1964
Glacial lake clays (CL)	6.5 – 8.7	2.0 – 2.7	Wallace and Otta, 1964
Chicago silty clay (CL)	8.5	2.7	Terzaghi and Peck, 1967
Swedish medium sensitive clays (CL- CH) 1. Field 2. laboratory	 0.4 – 0.7 0.7 – 3.0	 0.10 – 0.2 0.2 – 1.0	Holtz and Broms, 1972
San Francisco Bay Mud (CL)	2 – 4	0.60 – 1.2	Leonards and Girault (1961)
Mexico city clay (MH)	0.9 – 1.5	0.30 – 0.5	Leonards and Girault (1961)
Silty clay	6.6 – 66.6	2 - 20	O' Flaherty (1991)
Clayey soils	0.6 – 30	0.2 - 10	O' Flaherty (1991)
montmorillonite	0.2 - 1	0.06 – 0.3	Cornell (1950)
Illite	1 - 8	0.3 – 2.4	Cornell (1950)
Kaolinite	12 – 90	3.6 - 30	Cornell (1950)
Silty clay of Bangladesh	1.9 to 50	0.6 to 15	Aminullah (2004)

3.3.4.7 Pre - Consolidation Pressure (Pc)

Pre - consolidation pressure (P_c) is the maximum pressure which the soil subjected in the past. It is one of the most important properties of soft clay and used in embankment design. It is the pressure at which major structural changes including the breakdown of inter - particle bonds and inter - particle displacement begin to occur (Ralph et al, 1967). The ratio of pre - consolidation pressure to present effective overburden pressure is known as overconsolidated ratio (OCR). This ratio ranges from 1.2 to 3 for variety of natural soft clay (Mesri et al., 1967). In soft clay and silts, fluctuation of the water table, under drainage, minor erosion of sediments may have contributed to pre - consolidation.

Table 3.8: Typical values of Mv (Barnes, 2001)

Types of clay	M_v (m^2 / MN)
Very stiff heavily over - consolidated clay	< 0.05
Stiff over - consolidated clay	0.05 – 0.1
Firm over - consolidated clay, laminated clay, weathered clay	0.1 – 0.3
Soft normally consolidated clay	0.30 – 1.0
Soft organic clay, sensitive clay	0.5 – 2.0
Peat	> 1.5

3.3.4.8 Pore Pressure Coefficient (A)

If a soil sample is subjected to instantaneous loading, the excess pore water pressure will be developed if hydraulic conductivity of the soil is poor. There is an increase in pore pressure in excess of the hydrostatic pressure. The pore pressure coefficient is used to express the response of pore pressure to changes in total stress under undrained conditions. This value can be determined in the laboratory by consolidated undrained shear strength test with pore pressure measurements.

Some typical values of 'A' for different kinds of soil determined Skempton (1954) are given in Table 3.9. Moreover, Murthy (2003) reported that the value of 'A' for normally, over consolidated clay and compacted sandy clay ranges from 0.5 to 1, - 0.5 to 0 and 0.25 to 0.75 respectively. Besides this, Craig (2004) has established a typical relationship between pore pressure coefficient 'A' at failure condition and Overconsolidation ratio (OCR) is given in Figure 3.2.

Table 3.9: Some values of pore pressure parameter (Skempton, 1954)

soil	Pore pressure coefficient, A
Loose fine sand	2 -3
Sensitive clay	1.5 – 2.5
Normally consolidated clay	0.7 – 1.3
Lightly over consolidated clay	0.3 – 0.7
Heavily over consolidated clay	- 0.5 – 0.00
Compacted sandy clay	0.25 – 0.75

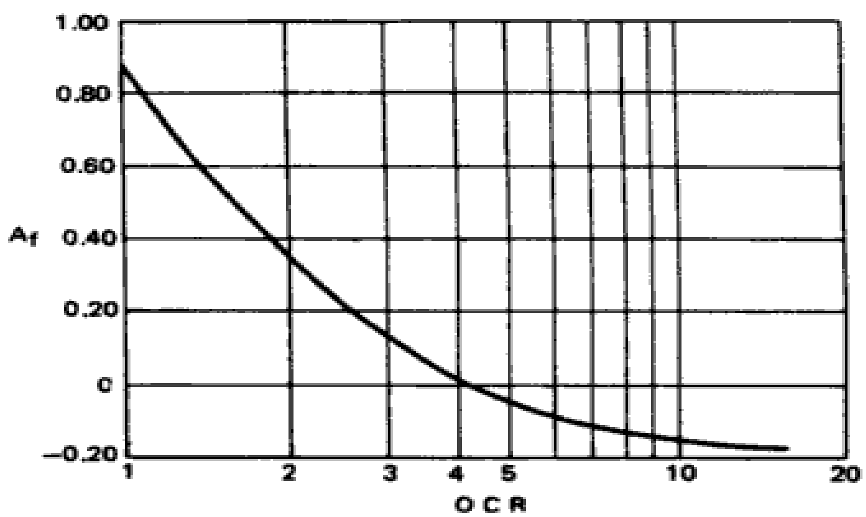


Figure 3.2: Typical relationship between pore pressure coefficient, A at failure and over consolidation ratio (Craig, 2004)

3.4 Subgrade Soil Properties

The soil immediately below the formation level is generally referred to as the subgrade. It is also known as the foundation of pavement. As discussed earlier, upper soil strata in Bangladesh are silt and clay of low to medium plasticity occur predominately in many areas. These predominantly occurring silty and clayey soils, generally rated as fair to poor subgrade materials, are commonly used in Bangladesh in most areas in the construction of road embankment and road subgrade. These silty and clayey soils are often improved by blending with local fine sand at different proportion or may be sometimes stabilized with small percentage of cement or lime for increasing the strength of the road subgrade. The strength of subgrade has a great influence on the thickness design of pavement (Yoder and Witczak, 1975). Thus if the subgrade is strong, the thinner pavement layers may be used. Therefore, improvement of subgrade soils can make the pavement cost effective. RMSS report states that a good subgrade can increase the long life of a road. However subgrade quality must meet the conditions of short term rigidity. In order to get long term performance, good drainage must be ensured. Subgrade materials are characterised by their resistance to deformation under load which is measured by their strength or stiffness. In general, the more resistance to deformation of a subgrade soils can support more loads before reaching a critical deformation value. Selig et al. (2003) have established a chart shown in Figure 3.3 for estimating the UK subgrade properties. The chart shows relationships among subgrade types, its strength properties (soaked CBR) and stiffness properties (resilient modulus).

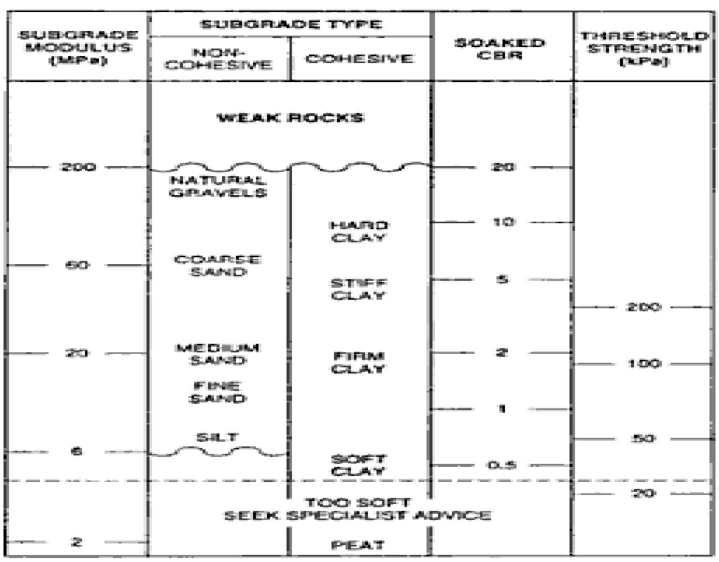


Figure 3.3: Basis for estimating Subgrade properties (Selig et al., 2003)

3.5 Soils Properties used for Embankment Design

Properties of both fill materials (for construction of earthworks) and foundation soil are needed for the design of embankment. The design parameters for the fill materials are the unit weight of soil, types of soils, particle size distribution, Atterberg limit and undrained as well as drained shear strength parameter like undrained cohesion (Cu), angle of internal friction (ϕ) and drained cohesion ($c^/$), drained angle of friction ($\phi^/$). Meanwhile the design parameters for the foundation soils are mainly the index properties, shear strength parameters and consolidation parameters. The details of the former one are the depth wise drained and undrained cohesion and angle of internal friction and the details of the later one are the depth wise compression index (C_c), secondary compression index (C_α), coefficient of consolidation (C_v), coefficient of volume compressibility (M_v), permeability (k), swell index (C_s), recompression index (C_r), undrained modulus (E), initial void ratio (e_o) and pore water pressure are needed for embankment design.

3.6 Granular Materials

The granular materials used in road construction of Bangladesh are boulders, gravels, pebbles, shingles, hard rock, bricks and sands. Most of the stone quarries situated in the north and north - east part of the country but road materials deposits are too scant and scattered. In some instance boulder are mined from rivers. Again this is a very limited source of rock. Besides this, construction of roads in other parts of the country will require long haulage distances that attribute more cost. So considering the availability and economy, non conventional brick aggregates termed as “brick khoa”, abundantly produced all over the country are used for road construction in Bangladesh. Bricks are manufactured across large area in Bangladesh and its annual production is over one billion. Aggregates are produced from crushing of bricks by machine or hand following specific gradation limits and widely used in road subbase and base (in some low cost road) construction in Bangladesh. The maximum size of brick khoa should pass through a 38 mm sieve (Alam, 2002). The quality of brick aggregate should be determined by its abrasion value and percentage of water absorption. The over – burnt picked Jama bricks are such kind of bricks that have low abrasion value as well as less percentage of water absorption value and are almost as good as stones in quality (Zakaria, 1986). Whereas inferior quality bricks known as Ama bricks which is yellow in colour should not be used in road construction as they have high abrasion value as well as higher percentage of water absorption. As noted earlier there are lots of rivers that flow through the country. These rivers carry huge amount of fine, medium and coarse sand from neighbour countries that include India, Myanmar during monsoon and deposited to Bangladesh. This sand can be properly utilized for construction of road and embankment. But

Ferthous (2007) has reported that the river bed sand of Bangladesh contains mica that effects on strength and stiffness property. The availability and deposit of road construction materials are discussed in the Appendix- A.

3.6.1 Materials properties

The size, shape, angularity and gradation are the main physical properties of aggregates. These surface properties of granular materials could affect the stiffness and permanent deformation characteristics of aggregates (Barksdale and Itani, 1989). Thom and Brown (1988) stated the significance of these physical properties of aggregates and pointed out that the surface properties of the large and small particles in the same material could be different. However, most of the researcher recommended well graded aggregates that provides dense packed mass (Lay, 1990). The particle shape and surface texture are those control the manner and degree of particles interlock upon which affects shearing resistance, crushing resistance and flexural & tensile strengths (Lees and Kennedy, 1975).

3.6.2 Strength Properties

The aggregate strength depends on the type of the rock from which it is produced (Lees and Kennedy, 1975). The strength of artificially produced brick aggregates depends on the properties of the soil from which brick is made. Bindra (1982) and zakaria (1986) were determined the ACV, AIV, TFV and LAAV value for brick aggregates shown in Table 3.10. Table shows that their findings are almost similar except TFV. Again Zakaria (1986) also reported that the ACV of the brick aggregates for dry and wet conditions are 36% and 39.5% respectively. However, ACV found by the both researchers is above 30% which indicates that this brick aggregates can not be used in the upper layer of a pavement because most of the highway agencies specified this value should be less than 30%.

Table 3.10: The Strength properties of brick aggregates found by Bindra (1982) and Zakaria (1986)

Properties	Bindra (1982)	Zakaria (1986)
ACV (%)	35	36
AIV (%)	30	32

TFV, KN	52	70
LAAV (%)	37	36

3.6.3 Resilient Modulus

Analytical pavement design depends on the traffic load, environment and properties of materials used in construction. All studies have found that for given loading and environmental conditions, pavement performance depends on resilient modulus and permanent deformation (Mohammad et al. 2006). These performance parameters used in analytical design are measured by cyclic triaxial test maintaining concurrently under loading & environmental conditions similar to those the soils are experienced in the field (Frost et al. (2005).

The soil and granular layers have non linear stress – strain relationship influenced by a range of factors such as compaction, properties of soils & materials and loading condition. Several factors such as compacted density, moisture content and gradation of materials affect the rate and magnitude of permanent deformation of pavement materials (Khogali & Mohammed, 2007). On the other hand, Puppala et al. (1999) stated that soil types, moisture content, dry unit weight and deviator stresses affect on the resilient modulus as well as permanent deformation of soils and materials. Andrew (2005) has shown the factors that affect on resilient modulus are deviator stress, confining pressure, moisture content and matrix suction which are highly dependent on temperature, water content as well as stress history. Fredlund et al. (1977) stated that resilient modulus of fine grained soils is dependent on deviator stress and metric suction and to a lesser extent on confining stress. Meanwhile, Li and Selig (1994) have stated that resilient modulus is a function of moisture content & compaction effort. Such relationship among deviator stress & resilient modulus, deviator stress & resilient strain and resilient modulus & moisture content for cohesive soils determined by Lee et al. (1997) are shown in Figure 3.4, 3.5 and 3.6 respectively.

Meanwhile, for granular materials resilient modulus is depended on gradation, density and moisture content. The resilient modulus of granular materials decreases significantly as the gradation changes from coarse to fine, as the density decreases, as the moisture content increases (Heydinger et al. 1996). The resilient modulus for granular materials is a function of confining pressure and the cyclic deviator stress level; thus a unique relationship cannot be determined (Flintsch et al., 2005). Meanwhile, several resilient modulus models can be successfully used to describe the stress - dependent behaviour of granular materials. Among them k- θ response model is widely used for granular materials (Flintsch et al., 2005).

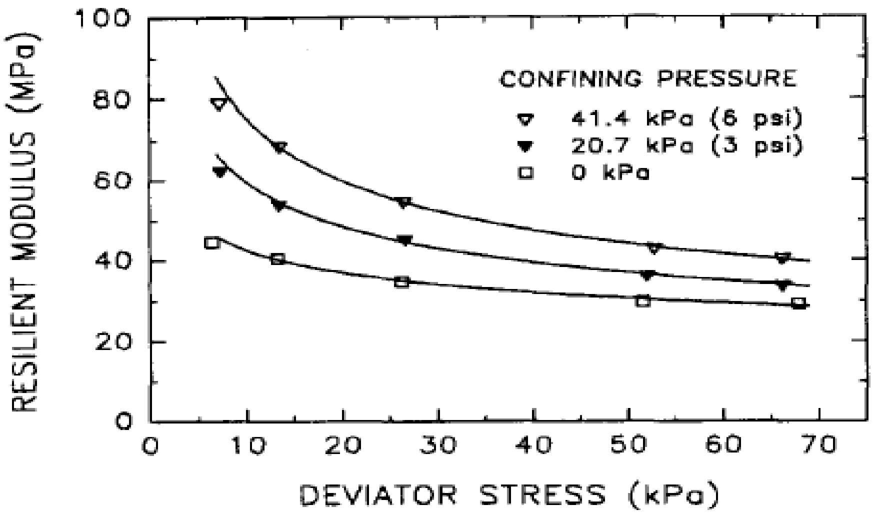


Figure 3.4: Relationship between Resilient Modulus and Deviator Stress for different confining pressure on cohesive soils (Lee et al, 1997)

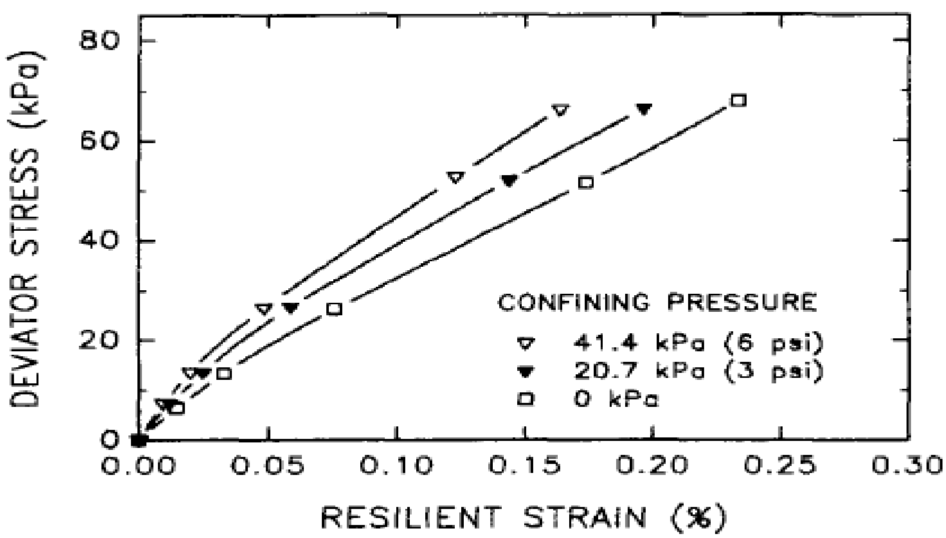


Figure 3.5: Relationship between Deviator Stress and Resilient strain (%) for different confining pressure on cohesive soils (Lee et al, 1997)

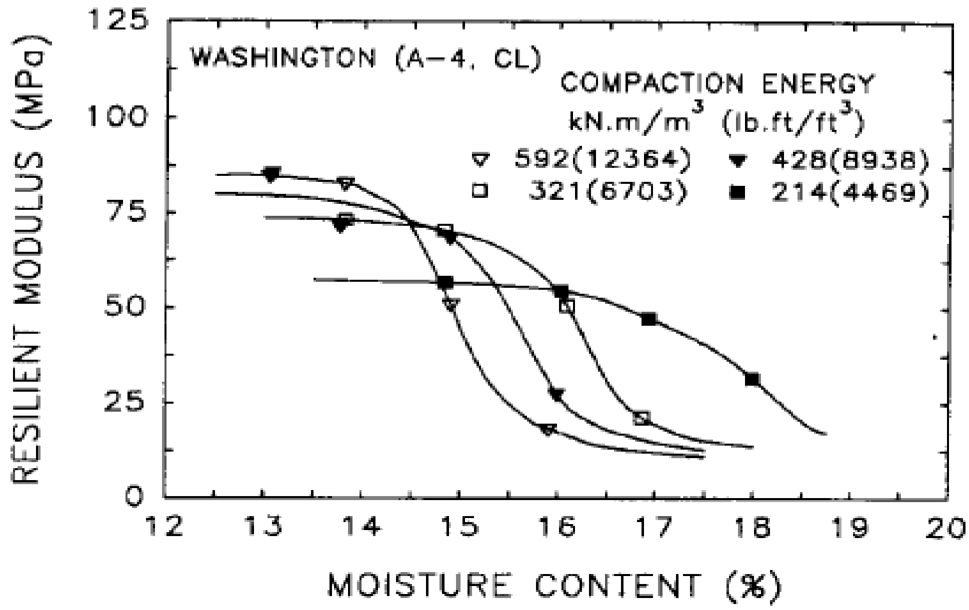


Figure 3.6: Variation of Resilient Modulus with moisture content for laboratory compacted soils (Lee et al., 1999)

3.6.4 Poisson's Ratio

Poisson's ratio is the ratio of transverse contraction strain to longitudinal extension strain in the direction to stretching force. It is an important parameter of material that is used in the pavement design. Its value varies with the strain level and becomes constant only at strains in the failure range (Lambe and Whitman, 1969). It is generally constant at cyclic loading; in cohesionless soils and cohesive soils the value ranges from 0.25 to 0.35 and 0.4 to 0.5 respectively (Hunt, 2007).

3.6.5 Particle Size Distribution of Granular materials

Particle size distribution is one of the important factors that govern the strength, stiffness and drainability of pavement base and subbase materials. Lay (1990) recommended a well- graded distribution of particles i.e. particles of all sizes in a proportion that gives a dense packed mass. In well graded materials larger particles are surrounded by the smaller particles that provide the better support and confinement to the interlocked larger particles (Bartley 1982). Some researchers (Tom and Brown 1988, Kamal et al. 1993) have reported that the resilient modulus generally decreases as the amount of fines increases. Hicks (1970) reported that increases in 2% to 10% fines, does not have a significant effect on resilient modulus. For aggregates with the same amount of fines and similar shape of particle size distribution, the resilient modulus was observed to increase with increase the size of maximum particle (Gray 1962; Kolisoja 1997). The explanation is that when the load is applied on the granular materials, major part of the load is transmitted by particles queues through coarse particles, the larger number of particles contracts results in less stress and hence less deformation. Thus the particle size distribution of granular materials seems to have considerable influence on materials stiffness.

Poisson's ratio is also influenced by the grading of particles. Hicks (1970) investigated the effect of fines content on Poisson's ratio and concluded that an increase in the amount of fines results in a decrease in Poisson's ratio. So considering the above mentioned factors, particle gradation should be selected in such a way that gives optimum strength and stiffness as well as provide good drainage facilities. The gradation requirement of pavement base and subbase materials suggested by the ASTM and Overseas Road Note – 31 (TRL) are given in Table 3.11 and 3.12 respectively. Tables show that TRL recommended three grading envelopes for base materials depending on the maximum sizes of the particles while ASTM recommended only one grading envelope for those materials.

Table 3.11: Grading requirements for bases and subbase for highways and airports to ASTM D 2940 –85)

Sieve size (mm)	Grading : percentage passing	
	Bases	Sub - bases
50	100	100
37.5	95 – 100	90 – 100
19	70 – 92	-
9.5	50 – 70	-
4.75	35 – 55	30 – 60
0.600	12 – 25	-

0.075	0 - 8	0 – 12
Other requirements: Fraction passing the 0.075 mm sieve should not exceed 60 percent of the fraction passing through 0.600mm sieve and coarse aggregate should be hard and durable.		

Table 3.12: Grading requirements for subbase and base granular materials (TRL, 1993)

Sieve size (mm)	Grading: Percentage passing			
	subbase	Base (nominal maximum particle size) (37.5)	(20)	(10)
50	100	100	-	
37.5	80 – 100	80 – 100	100	-
20	60 – 100	60 – 80	80 – 100	100
10	-	45 -65	55 – 80	80 – 100
5	30 – 100	30 – 50	40 – 60	50 – 70
2.36	-	20 – 40	30 – 50	35 – 50
1.18	17 – 75	-	-	-
0.425	-	10 - 25	12 – 27	12 – 30
0.30	9 – 50	-	-	-
0.075	5 - 25	5 - 15	5 - 15	5 - 15

3.6.6 Permeability

The Permeability of granular materials and soils is measured by the coefficient of permeability (K). The value of K depends on the size of the particles. Particle size distribution and packing of well graded materials gives low K value. If the soil contains more voids, the permeability will be higher. Besides this, the value of K also depends on the viscosity of the water as it varies with change in temperature. Permeability is further discussed below in two groups such as fine grained soils and granular materials.

3.6.6.1 Permeability for Fine Grained Soil

Usually the value of coefficient of permeability for fine grained soils is very extreme low and sometime it is impermeable. Different researchers have determined the k value for fine grained soils. Some typical values of k suggested by Aysen (2002) for different types of soil are given in Table 3.13. Aminullah (2004) worked on the Bangladesh soils (silty & clayey) and determined the K value from one dimensional consolidation test that ranged from 9.07 x 10^{-11} m/s to 4.18 x 10^{-9} m/s for effective overburden pressure 6.25 to 600 kPa.

Table 3.13: Some typical values of coefficient of permeability for different types of soils (Aysen, 2002)

Types of soil	Coefficient of Permeability (m/s)
Clean gravels	$1 – 10^{-2}$
Clean gravels, Clean sand and gravel	$10^{-2} – 10^{-5}$
Very fine sands, organic and inorganic silts, mixtures of sand, silt and clay	$10^{-5} – 10^{-9}$
Clays	$10^{-9} – 10^{-11}$
Well drained soils	$1 – 10^{-6}$
Poorly drained soils	$10^{-6} – 10^{-8}$
Practically impervious	$10^{-8} – 10^{-11}$

Siddique and Safiullah (1995) have done the permeability test on Dhaka clay by the Constant head permeability test method and one – dimensional Consolidation test method. The K value determined by the constant head permeability test ranged from 0.74 x 10^{-10} m /s to 7.35 x 10^{-10} m / s for the void ratio and dry density of the soil samples in the range of 0.51 to 0.84 and 14.2 kN/m^3 to 17.4 kN/m^3 respectively. On the other hand, the same parameter determined using the square root of time method are higher than those determined using logarithm of time fitting method for all ranges of stress (Ferthodus, 2007). The relationship between void ratio and coefficient of permeability determined by Constant head permeability test and One – dimensional consolidation test for Dhaka Clay (after Siddique and Safiullah, 1995) is shown in Figure 3.7 which shows the non linear relationship. The coefficient of permeability determined by constant head permeability test is higher than that found from consolidation test. It is also observed that the values of permeability decreases with decrease in the void ratio for all three curves.

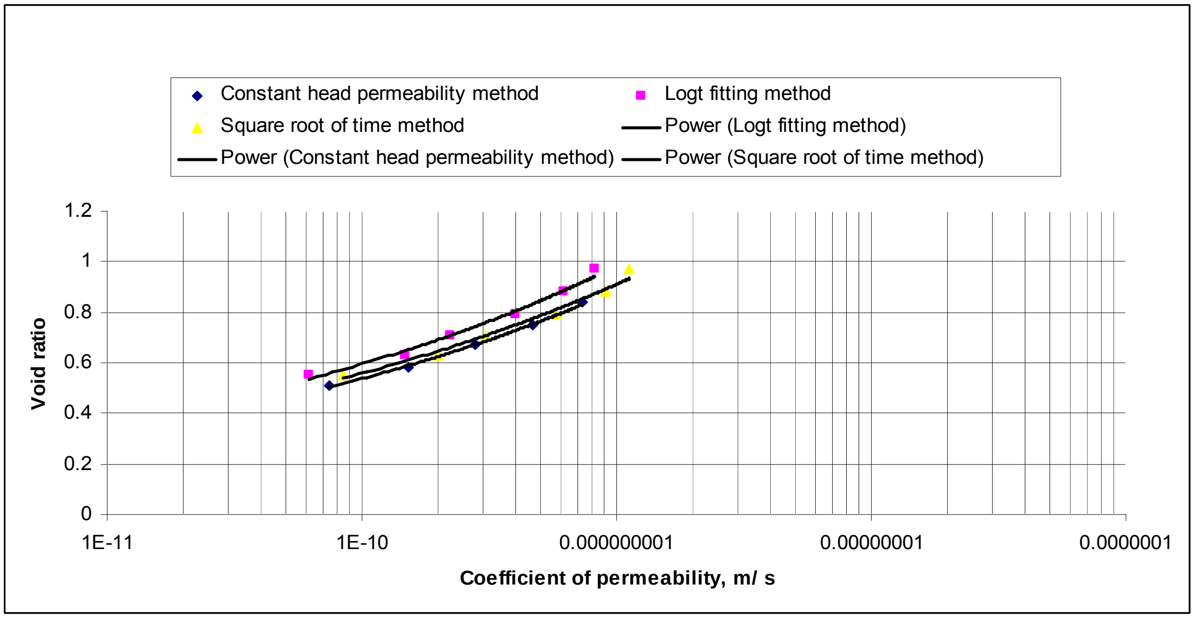


Figure 3.7: Relationship between void ratio and coefficient of permeability for Dhaka Clay (Siddique and Safiullah, 1995)

3.6.6.2 Permeability for Granular Materials

The permeability of granular materials is an important issue that should be properly addressed in the design of highway. This applies to all materials particularly soils and drainage media used in the construction of highway and related earthworks. More permeable materials have contributed to reduce the cost of road maintenance as well as life cycle cost (Bouchedid et al., 2005). Alam (2002) have determined the k value for unbound granular materials used in road base (Stone aggregate, sand and soil mix) and subbase (Brick aggregate, sand and soil mix) layers as used in the construction of pavement in Bangladesh are about 3.92 $x10^{-6}$ cm/s and 8.7 $x10^{-6}$ cm/s respectively whereas this value for capping layer (sand: sizes in the range of 0.06 mm to 2 mm) is 10^{-3} to 10^{-4} cm/s depending on the fineness of the sand. Many researchers (Mallela et al. 2000, Dawson et al. 1989, Kozlov 1984, Birgisson & Roberson 2000) recommended for a K of 0.35 cm/s to 1.75 cm/s to ensure proper drainage of the excess pavement water based on the environmental and physical conditions. To ensure this, it needs open graded drainage layer. The use of the open graded drainage layer is often leads to pavement rutting problems because of their low stability (Elsayed et al. 1996). On the other hand, dense graded materials possess low permeability and high strength as well as stiffness. Therefore, the two requirements are conflicting (Jones and Jones 1989, Bathurst and

Raymond 1990). However, in most cases the designer try to make a compromise between two opposite requirements.

Cedergren (1974) stated that this value varied with so wide range that no other engineering properties can vary like permeability. Laboratory permeability value is more accurate for use in design than reported values from any design manual (Faw et al, 2001).

3.7 Properties of Bituminous Materials

The bituminous layer of pavement consists of specified graded stone aggregates and bitumen as binder material. The notable properties of bituminous materials are the resilient modulus, stability and flow. The importance of resilient properties for pavement foundations was recognized and was associated them with the incidence of fatigue cracking in bituminous surfacing (Hveem, 1955).

The bitumen content depends upon the types of the layer such as base course & wearing course and gradation of the aggregates used in the mix design. The property tests of bitumen are specific gravity, penetration, softening point, fire point & flash point, solubility, ductility and viscosity. These properties play a vital role in the longevity of bituminous mixes. However, the overall strength and stiffness properties of mixture depend upon the bitumen properties and bitumen content in the mixes, properties of aggregates, temperature and compaction. The effect of temperature on bituminous is given in Table 3.14. Table shows the stiffness parameter of bitumen measured by elastic modulus is sharply decreased with increased in temperature.

Table 3.14: Typical values of Elastic modulus for bituminous materials (Newcomb et al. 2002)

Materials	Elastic Modulus (MPa)
Hot mix bitumen (0 ^{0}C)	14,000
Hot mix bitumen (21 ^{0}C)	3500
Hot mix bitumen (49 ^{0}C)	150
Crushed stone	150 - 300

3.8 Summary:

Some typical values of engineering properties of soils and materials are discussed in this chapter. This information is not sufficient for design of embankment and pavement for Bangladesh. There are inherent danger in extracting information for other parts of the world, since the prevailing climate and both construction & load regime may be different. Besides this, there is a scarcity of conventional materials used in the construction of road. So non conventional brick aggregates were used in pavement layers. But the resilient modulus and permanent deformation of that kind of aggregates as well as subgrade soils had not yet been determined. Hence, these important design parameters are not available in Bangladesh.

On the other hand, road embankment is made of soft silty clay or sandy silty clay. The subsoil is sometimes found black organic soils. All strength and settlement parameters for those kinds of soils are also unavailable. So, there is an inherent need to investigate the engineering properties of soils and materials found in Bangladesh.

Chapter 4: Correlations of the Properties of Materials

4.1 Introduction

This chapter deals with correlations for the characterisation of strength and stiffness properties of soils, granular materials together with bituminous materials. It also deals with the correlations for settlement behaviour of soils. In terms of engineering applications, the information concerning soil types and soil conditions; correlations for both soils and granular as well as bituminous materials obtained is limited due to the difficulties encountered in sampling, testing and available time. Therefore it is necessary to use the correlations by using a small number of soil parameters that can be easily obtained. These correlations are important to estimate engineering properties of soils particularly for a project where there are lack of testing equipment, limited time and financial limitations. It was rather helped to collect available data that have been collected to develop collections specific to Bangladesh.

Many correlations are available for determination of the different materials properties. These correlations were derived from different kinds of soils and from different parts of the world. However, for this study correlations that are most appropriate for Bangladesh will be selected based on some criteria such as soil classifications, mineralogy, climatic conditions and geographical conditions etc.

Moreover, due to the scarcity of laboratory equipment in Bangladesh, properties required for the design of embankments and pavements can not be determined directly. Hence some simple tests have to be done using the available equipment for soil as well as granular materials and using the suitable correlations the design input for pavement and embankment have been determined.

4.2 Properties of Soil

Soil may be characterised by their engineering properties. Such properties are usually determined by conducting laboratory and /or in situ tests. But these tests are usually found to be complicated, costly, time consuming, very often, in practice, use is made of correlative determinations or simple correlations (Taylor & Francis, 1996). Some properties may be found directly from testing and others may be found from the correlations. Different correlations that are discussed below; enable to predict the material properties without conducting time consuming and expensive standardized laboratory testing or the lack of equipment availability. Estimation of engineering properties through correlations is discussed in the following subsections.

4.2.1 Strength and Stiffness of Soil

The correlations between strength and index properties of soils suggested by different researchers for different kinds of soils and materials are discussed below:

4.2.1.1 Relationship between liquidity index and undrained shear strength

The undrained shear strength of soil can be estimated from the liquidity index using correlations developed by Tarzaghi (1996) and Skempton & Northey (1952) shown in Figure 4.1 and 4.2 respectively. The undrained shear strength results suggested by Terzaghi are derived for soft clay and silty soils from many parts of the world. Whereas, Skempton and Northey's (1952) figure is only related to the UK soil. Both sets of results however show very similar correlations.

Wroth, 1978 has shown the relationship between undrained shear strength and liquidity index in equation as follows:

$$C_u = 170\ e^{-4.6\,(I_L)} \qquad (4.2.1)$$

Where I_L = liquidity index, C_u = undrained shear strength, kPa, e = void ratio

Undrained shear strength (Vane shear strength) can be determined by knowing the water content and liquid limit of the material using Lee (2004) correlation.

$$\text{Vane shear strength (psf)} = 183\ e^{-2.3714(w\,/\,w_L)} \qquad (4.2.2)$$

Where, w = water content in percent and w_L = Liquid limit in percent, e = void ratio.

4.2.1.2 Relationship between Plasticity Index and undrained shear strength:

Whyte (1982) suggested that the ratio of the undrained shear strength at the plastic limit to strength at liquid limit is approximately 70. Skempton & Northy (1953) suggested the ratio is approximately 100. A comprehensive collection of equations relating to soil plasticity and compressibility indices is reported by Bowles (1996). The above mentioned relationship is very useful in guiding the feasibility study before conducting extensive soil exploration and strength tests.

Skempton & Henkel (1953) derived a correlation between Plasticity Index (I_P) and the ratio of undrained shear strength to effective overburden pressure (S_u/p') for normally consolidated clays. The equation is as follows:

$S_u / p' = 0.11 + 0.0037\ I_P$ (4.2.3)

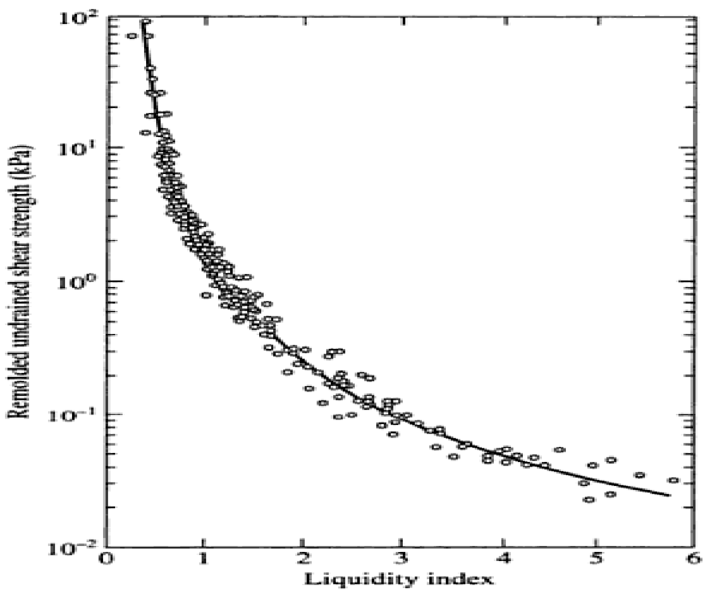


Figure 4.1: Relationship between Undrained shear strength and Liquidity Index of soils (Terzaghi et al. 1996)

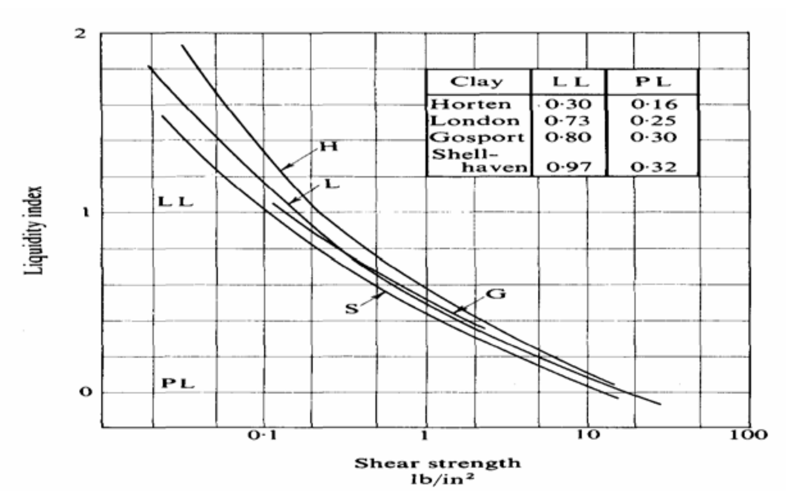


Figure 4.2: Relationship between Undrained shear strength and Liquidity Index for remoulded soils (Skemton and Northey, 1952).

Several researchers in the field of soil mechanics have confirmed the correlation. However, few researchers have discussed its limitations. Sridharam and Narasimha

(1973) re-examined the correlation in the light of the available published data, experimental results and certain theoretical considerations and concluded that no linear increased in S_u / p' occurs with increased in I_p but S_u / p' decreased with increased in I_p. They suggested that the result in a band rather than a single line for that relationship. On the other hand, Bjerrum and Simons (1960) have worked on the Norwegian clay and proposed a relationship between Cu / P' and I_p is shown in equation as follows

$$Cu / p' = 0.45 (I_P)^{0.5} \text{ for } I_P > 5\% \tag{4.2.4}$$

Another relationship is expressed

$$Cu / p' = 0.18 (I_L)^{-0.5} \text{ for } I_L > 5\% \tag{4.2.5}$$

Some of the factors that may influence the above mentioned equations are geological history and stress history during the test.

Karlsson and Viberg (1967) have proposed a relationship as

$$Cu / p' = 0.005 w_L \text{ for } w_L > 20\% \tag{4.2.6}$$

Bjerrum (1974) showed that as the plasticity of soils increases, undrained shear strength obtained from vane shear tests may give results that are unsafe for foundation design. Because field data gives the more results, for this reason he suggested the following correction:

$$C_{u\ design} = \lambda C_{u(vane\ shear)} \tag{4.2.7}$$

Where, λ = correction factor for soft clay = 1.7 – 0.54 log (I_P). (4.2.8)

Morris and Williams (1994) have suggested another correlation for estimating the correction factor:

$$\lambda = 1.18\ e\text{-}0.08(I_P) + 0.57 \tag{4.2.9}$$

Tavenas & Leroueil (1987) have established a chart between Cu / p' (p' is the effective overburden pressure) and I_p based on comprehensive test data analysis is given in Figure 4.3. But this C_u value is the field value which should be corrected by multiplying the C_u (vane) with correction factor (μ). Bjerrum (1973) has reported a relationship between correction factor (μ) and plasticity index (I_p); using this relationship field vane shear strength is calculated as: C_u (field) = μC_u (vane).

The correction factor have derived based on the back – analysis of embankment failures data and field vane shear test results, depending on plasticity index (Ladd et al.,1977).

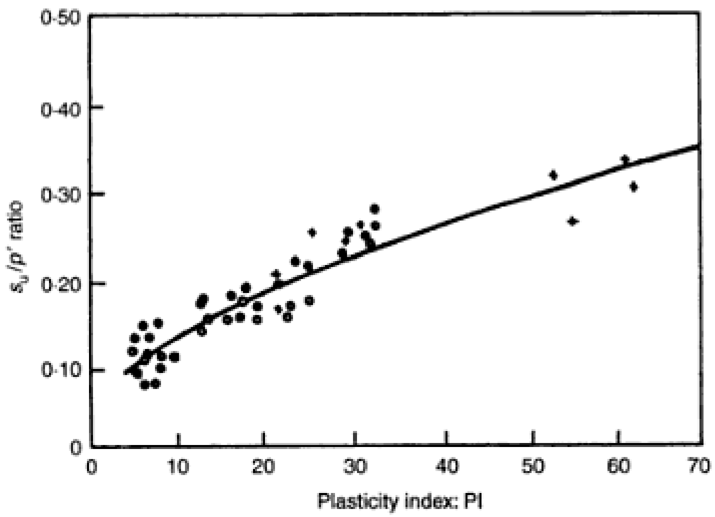


Figure 4.3: Undrained shear strength from field vane tests on inorganic soft clays and silts (after Tavenas and Lerueil, 1987)

4.2.1.3 Relationship between Unconfined compressive strength and N value

A number of empirical correlations between SPT – N value and unconfined compressive strength for different kinds of soils proposed by various researchers are given in the Appendix G, Table A- 4.1. However, the use of the correlation equations is not well defined and according to the Sivrikaya & Togrol (2006) four uncertainties like SPT corrections, statistical meaning of correction, types of soils and test results used in correction have been arisen in use of empirical correlations. Sower (1979) has developed a relationship between SPT – N value and undrained shear strength of different kinds of soils are given in Appendix H, Figure B- 4.1. The relationship among corrected N value (N_{cor}), relative density and drained angle of friction ϕ' for cohesionless soils is presented in Appendix G, Table A- 4.2.

4.2.1.4 Relationship between drained shear strength and Plasticity index of soils

Drained shear strength is used in the slope stability analysis of embankment. The method of determination of the parameter is discussed in chapter seven. If the laboratory equipments are unavailable for carrying out such test, alternative correlations are required to find out this parameter. As the shear strength and plasticity index properties of soil depend on the clay mineral composition of the soil (i.e. increase in mineral content, increase in plasticity index and resulting decrease in shear strength), so a relation may exist between two parameters. However, Gibson (1953) has proposed a relationship between drained angle of friction and plasticity index is indicated in Figure 4.4. The figure shows that both peak and residual angle of internal friction decrease with increase in plasticity index.

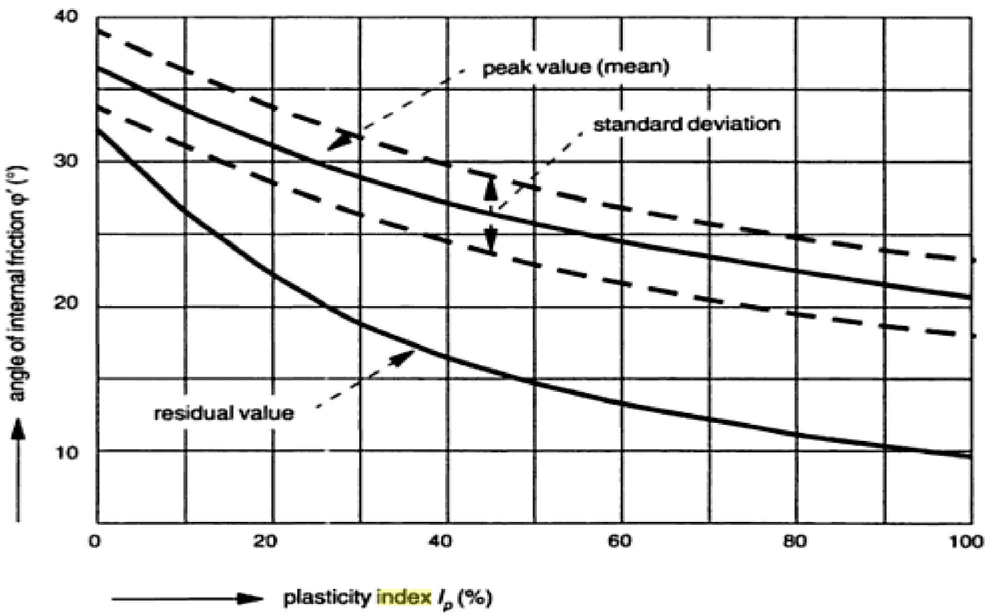


Figure 4.4: Relationship between Angle of Shearing Resistance and Plasticity Index (after Gibson, 1953)

4.2.1.5 Relationship between shearing resistance & unit weight of granular soils

The shearing resistance of granular soils depends on the relative compaction of the soils. Thus the shear strength of granular soils varies with the degree of compaction of the soils (Peck, 1975). A relationship between dry unit weight and the angle of shearing resistance for cohesionless soil established by the US Navy (1982) is presented in Appendix H, Figure B-4.2. The relationship shows that shear strength increases with increase in unit weight and relative compaction. The materials type noted in the figure is related to the unified soil classification system.

4.2.2 Stress – Strain relationship of soil

The stress- strain relationship of soil is one of the special interest for the road engineer who is primarily concern with repeated applications of stresses considerably smaller than those required to cause failure (Glanville, 1973). The estimation of undrained modulus from stress – strain relationship at laboratory may be inaccurate because of sampling disturbance, scaling factors, the effect of stress relief and bedding errors (Ladd, 1969; Raymond et al., 1971). Atkinson (1983) observed that there is a significant difference of undrained modulus value between laboratory and field measurement. The relationship suggested by Wroth (1978) that the modulus determined by field measurement is approximately five times the modulus determined from laboratory measurement. Besides this, the undrained modulus value obtained from UCS test is found one third of the values obtained from settlement observations (Simons, 1971).Thus the difficulties faced in selecting a modulus value from the results of the laboratory tests, it has been suggested that a correlation between insitu modulus (E_u) and the undrained shear strength S_u. The values of S_u are measured on undisturbed soils in the laboratory triaxial test and E_u values are derived from back – analysis of settlement observations on actual structures on a wide variety of soils. Different authors suggested different value of the ratio of E_u/ S_u. Bjerrum (1964) had worked on Fornebu soil and suggested the following relationship:

$$E_u = k\, S_u \tag{4.2.10}$$

Where, k = 250 to 500

Besides this different researchers have suggested the different values of k depending on the types of soils. Among them, Bjerrum (1972) had suggested the value of k = 500 to 1500. Another investigator, Skempton and Henkel (1957) have suggested the value of k = 140. Uddin (1990) had worked on the Dhaka clay and found the value of k for normally loaded state varies between 190 to 210, for overconsolidated clay k values varies between 113 to 198 for OCR values of 2 to 24.

4.2.3 Settlement properties

The compressibility properties of soils have been correlated with index properties of soils by different researchers for many years are discussed in the following subsections.

4.2.3.1 Empirical equations for compression index

A numerous consolidation tests were conducted on soils of different parts of the world. Hence various empirical correlations were developed by several researchers (Serajuddin & Ahmed 1967, Skempton 1944, Terzaghi & Peck, 1967) among compression index (Cc) and liquid limit (w_L), Plasticity index (w_p), initial void ratio (e_o), natural water content (w). These correlations are given in the following Table 4.1. It is mentioned that these correlations are derived for different types of soils are also noted in this table.

Table 4.1: Empirical correlations for determination of Cc suggested by different researcher

Suggested equation(s)	Soil type(s)	Reference	Comments	Equation number
$Cc = 0.0078\,(w_L - 14\%)$ $Cc = 0.44\,(e_o - 0.30)$	undisturbed plastic silts and clay soil samples of different areas of Bangladesh	Serajuddin and Ahmed (1967)	Based on Lab. / field / theoretical data	4.2.11 4.2.12
$Cc = 0.47\,(e_o - 0.46)$	fine grained soils occurring within about 7 (seven) meters from the ground surface of different areas of the Bangladesh	Serajuddin and Ahmed, 1982	Based on Lab. data	4.2.13
$C_c = 0.007(w_L - 10)$	Remoulded clays	Skempton (1944)	-	4.2.14
$C_c = 0.009\,(w_L - 10)$	Inorganic clays of sensitivity up to 4 (low and medium sensitivity) and liquid limit up to 100.	Terzaghi and Peck (1967)	Not applicable where sensitivity is greater than 4	4.2.15
$C_c = 1.15\,(e_o - 0.27)$	All clayey soils	Nishida (1956)	-	4.2.16
$C_c = 0.30(e_o - 0.27)$	Inorganic cohesive soils, silt, silty clay & clay	Hough (1957)	Based on experimental data	4.2.17
$C_c = 0.141\,G_s^{1.2}\left(\frac{1+e_o}{G_s}\right)^{2.38}$	Several natural clays	Rendon – Herreo (1983)	Based on theoretical data	4.2.18
$C_c = 0.2343\left[\frac{w_L(\%)}{100}\right]G_s$	Not mentioned	Nagaraj and Murty (1985)	Based on lab. test data analysis.	4.2.19
$Cc = Gs \times w_P / 200$	remoulded clay	Wood and Wroth (1978)	Based on some published data	4.2.20
$Cc = 0.009w_n + 0.005w_L$	all clays	Koppula (1986)	-	4.2.21

Cont…

Suggested equation(s)	Soil type(s)	Reference	Comments	Equation number
$C_c = 0.37 (e_o + 0.003LL+0.0004w_n - 0.34)$	Different kinds of soils	Azzouz et al. (1976)	Based on statistical analysis of a number of soils	4.2.22
$C_c = 0.48827 (Y_w/ Y_d)^{0.19167}$	a universal relationship	Rendon – Herrero (1980)	-	4.2.23
$C_c = -0.156 + 0.411e_o + 0.00058\ w_L$	-	Al – Khafaji & Andersland (1992)	-	4.2.24
$Cc = 1.15 \times 10^{-2}w_n$	Fr organic soils, peats and organic silt and clay	Azouz et al. (1976)	Based on statistical analysis of a number of soils	4.2.25
$Cc = 0.75(e_o – 0.50)$	Sils of very low plasticity	Azouz et al. (1976)	-	4.2.26 4.2.27
$Cc = 0.01\ w_n$	For Chicago clays			

4.2.3.2 Empirical equations for swelling index

In Bangladesh, there are some marshy lands all over the RMSS resigns in which soils are expansive clay. In the marshy area of Bangladesh and the areas subjected to seasonal flooding; there are soils that contain silt, clay, fine sand and organic matters. Such soils undergo varying amount of swell and shrinkage. Thus in order to ensure the suitable materials used in pavement construction, it is important to ascertain volume stability of the soils. Several investigators have developed correlations to predict the swelling characteristics based on Atterberg limits and placement conditions such as dry density and initial moisture contents. Since Atterberg limit such as liquid limit, plastic limit and the swelling properties of a soil are governed by the types of clay mineral present, so reasonable correlation may exist between these parameters.

Nagaraj and Murty (1985) have proposed the following empirical equation for swell index,

$$\mathbf{C_s} = 0.0463\left[\frac{w_L(\%)}{100}\right]G_s \qquad (4.2.28)$$

In most cases, C_s= 1/5 to 1/10 of C_c (4.2.29)

Vijayvergiya and Ghassahy (1973) have suggested that the swell index, I_s can be estimated using the following relationship

$$I_s = w_n / w_L \qquad (4.2.30)$$

Where, w_n is the natural moisture content (%) and w_L is the liquid limit.

4.2.3.3 Empirical equations for secondary compression index

Secondary compression index is very important for organic and peat soils. Different researchers have suggested different values of secondary compression index for different types of soils. Terzaghi el al. (1996) have reported the ratio of secondary compression index ($C\alpha$) to compression index (Cc) is ranged from 0.01 to 0.07 for all geotechnical materials and also suggested the most common value for inorganic clays and silts is 0.04.

$$\frac{C_\alpha}{C_c} = 0.04 \qquad (4.2.31)$$

Mesri and Godlewski (1977) reported the ratio of $C\alpha$ / Cc is constant for any soils.

$$\frac{C_\alpha}{C_c} = 0.032 \quad \text{Where, } 0.025 < C\alpha < 0.1 \qquad (4.2.32)$$

However, Mesri (1986) has reported the ratio of $C\alpha$ / Cc for peat and organic soils is

$$\frac{C_\alpha}{C_c} = 0.06 \text{ to } 0.07 \qquad (4.2.33)$$

Again, Mesri et al. (1990) suggested the ratio of $C\alpha$ / Cc for sandy clays is as follows

$$\frac{C_\alpha}{C_c} = 0.015 \text{ to } 0.030 \qquad (4.2.34)$$

Again, Mesri and Godlewski (1977) also presented the relationship between $C\alpha$ and Cc for James Bay peat and Mexico City clay is given in Figure 4.5 and 4.6 respectively.

These two figures show different value of the ratio of $C\alpha$ and Cc which occurs due to the different amount of organic matter presence in the soils.

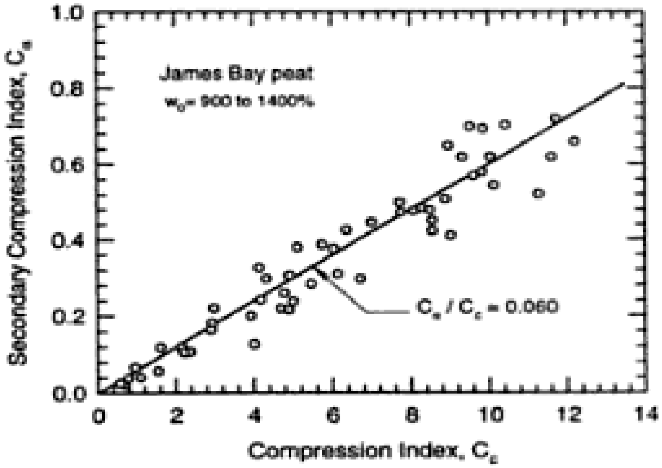


Figure 4.5: Relationship between Cα and Cc for James Bay clay (after Mesri and Godlewski, 1977)

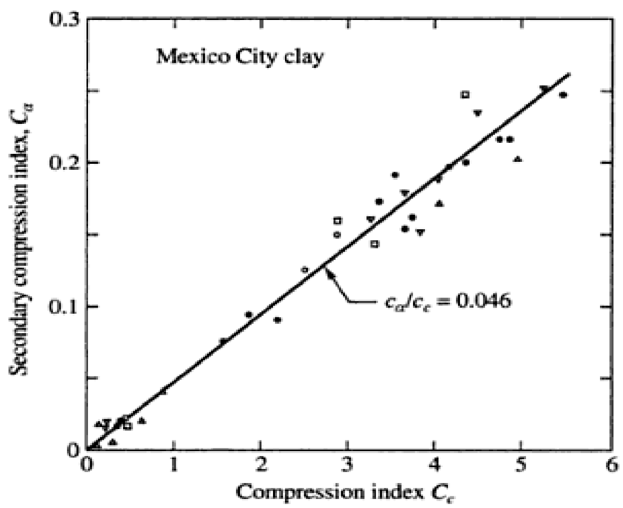


Figure 4.6: Relationship between Cα and Cc for Mexico City clay (after Mesri and Godlewski, 1977)

Ladd (1967) has suggested that for normally consolidated clayey soils, the value of $C\alpha$ is expressed in an equation:

$C\alpha$ = (4 to 6) x C_R (4.2.35)

Where C_R is the virgin compression ratio = Cc / (1+ e_o)

The relationship between secondary compression index and moisture content has established by Mesri (1973) for normally loaded clay deposits and different compressible organic soil is shown in Figure 4.7. On the other hand, same relationship has established in graphical form by U.S.Navy (1988) is also shown in Figure 4.8. From these two figures, same moisture content show different value of C_α which happens due to the difference of soils mineralogy. So, these relationships can not be used for Bangladesh soils. For this reason, it is needed for developing same relationship for Bangladesh soils.

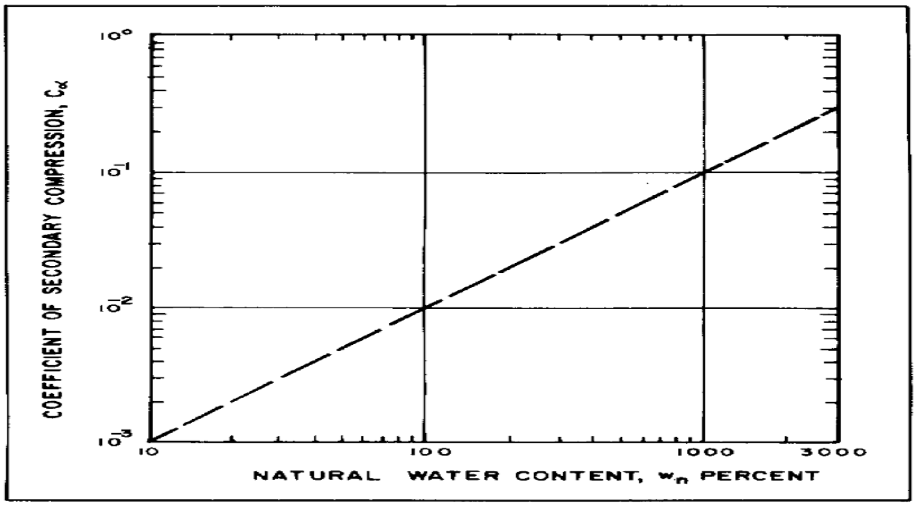


Figure 4.7: Relationship between coefficient of Secondary Compression and Natural moisture content (Mesri, 1973)

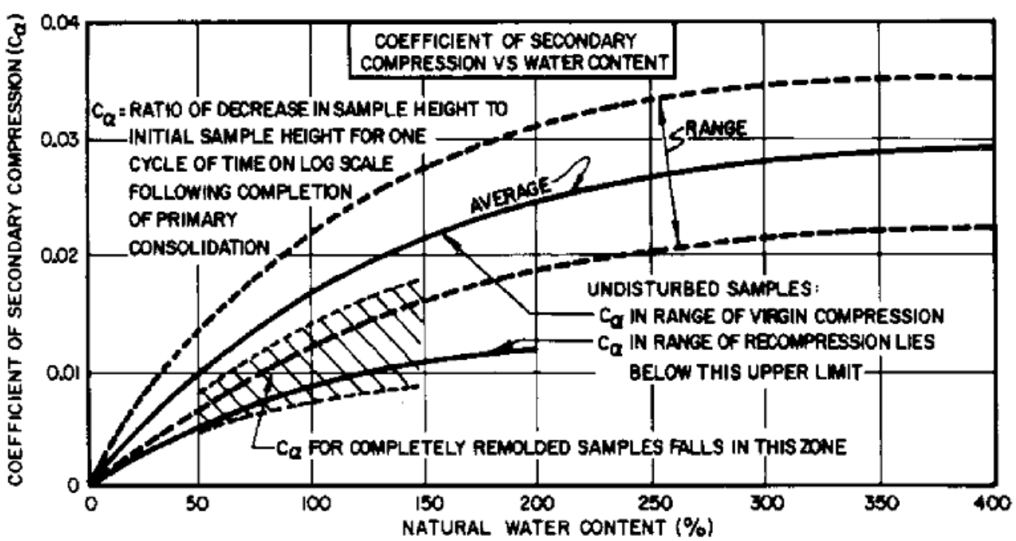


Figure 4.8: Relationship between coefficient of Secondary Compression and Natural moisture content (U.S.NAVY, 1988).

4.2.3.4 Empirical equations for determination of Recompression index

Different researchers have suggested several empirical equations for recompression index of soils relating to their basic soil properties. Among them, Azzouz et al. (1976) have suggested the following empirical equations for determinations of recompression index relating to the soils index parameters for silts of low plasticity:

$C_r = 0.14(e_o + 0.007)$ (4.2.36)

$C_r = 0.003(w_n + 7)$ (4.2.37)

$C_r = 0.001(w_L + 9)$ (4.2.38)

Besides this, Balasubramaniam & Brenner (1981) have proposed the relationship among recompression index and natural water content, liquid limit are as follows

$C_r = 0.00566\, w_n - 0.037$ (4.2.39)

$C_r = 0.00463 w_L - 0.013$ (4.2.40)

Other investigators, Nagaraj & Srinivasa Murthy (1985) have proposed the following equation for Indian clay soils

$C_r = 0.00463\, w_L\, G_s$ (4.2.41)

Moreover, Roscoe et al. (1958) have suggested some typical values of C_r ranges from 0.015 to 0.35 and also suggested the value of C_r are assumed to be 5 – 10% of Cc.

All of the empirical correlations were developed for different types of soils that may not be applicable for Bangladesh soils. Therefore, a necessity is made to develop similar correlations for Bangladesh soils.

4.2.3.5 Equations for coefficient of consolidation

Moh et al. (1989) determined a relationship between Liquid Limit and coefficient of consolidation for normally consolidated range (most of the clayey soils from Taipei) is given in the following equations:

$C_v = 0.033x10^{(-0.025w_L)}$ (4.2.42)

Where Cv is in cm^2/sec and w_L is the liquid limit

Again the relationship between coefficient of consolidation and Liquid Limit for Taipei soil is shown in Figure 4.9 and from the figure observed that correlations for undisturbed normally consolidated soils in Taipei is reasonably good agreement with that suggested by NAVPAC (1982).

The US Navy (1988) has proposed a relationship between coefficient of consolidation and Liquid Limit of soils for undisturbed and remoulded samples is shown in Figure 4.10. Figure shows that the value of coefficient of consolidation is decreasing with the increasing liquid limit of the soil. Again, from the curve observed that the value of Cv for undisturbed samples lies in the range of virgin compression; whereas for remoulded samples, values lies below the mentioned upper limit.

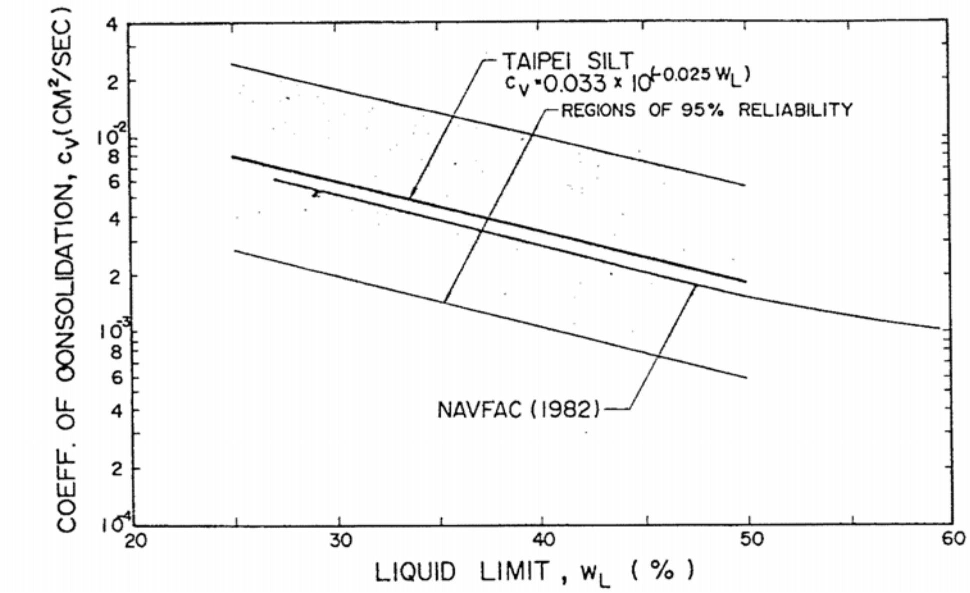


Figure 4.9: Relationship between coefficient of consolidation and liquid limit for Taipei soil (Moh et al. 1989)

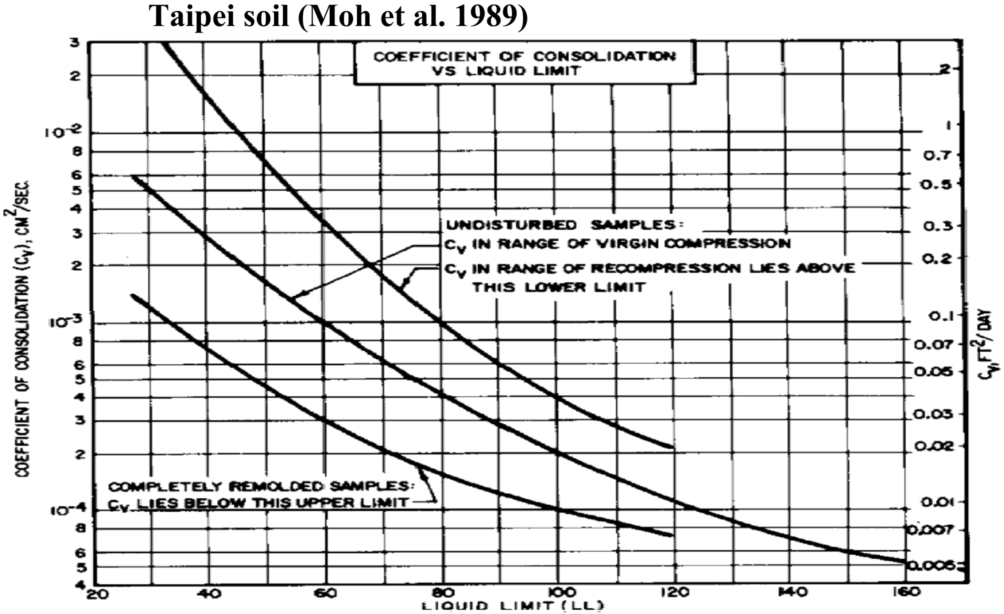


Figure 4.10: Relationship between coefficient of consolidation and liquid limit (U.S.NAVY, 1988).

4.2.3.6 Equations for permeability

Several researchers have proposed different empirical equations of coefficient of permeability for fine grained soils and granular materials that are discussed in the following paragraphs:

Carrier and Beckman (1984) have proposed an empirical equation for estimating the permeability for remoulded clays is given in equation:

$$k = 0.0174\left[e - 0.027\left(w_p - 0.242 I_p\right)/ I_p\right]^{4.29} / \left(1+e\right) \quad (4.2.43)$$

Where k = permeability, m/ sec, e = void ratio, w_p = Plastic limit (%),

I_P = Plasticity index.

The empirical equation proposed by Hazen (1911) a century ago correlates the coefficient of permeability (k) of fine grain and granular materials to their particle size distribution characteristics:

$$K = C\, D^2_{10} \quad (4.2.44)$$

Where k is in m/s and D_{10} is the diameter corresponding to 10% finer in mm and C is the coefficient which is equal to 0.005 and 0.012 for silt & well graded sands and uniform sands respectively.

Taylor (1948) developed a theoretical equation to calculate the permeability of soil relating the soil and permeant (water) properties is as follows

$$k = D^2{}_s . \frac{\gamma}{\mu} \frac{e^3}{(1+e)} . C \quad (4.2.45)$$

Where, k = the coefficient of permeability, Ds = the effective particle diameter,

γ = the unit weight of water, μ = the viscosity of water,

e = the void ratio and C = the shape factor.

Meanwhile, the effective particle diameter Ds is usually taken as the D_{10} and the equation is converted to the Hazen equation is stated above.

$$K = C_1\, D^2_{10} \quad (4.2.46)$$

Where, the constant C_1 replaces $\frac{\gamma}{\mu} \frac{e^3}{(1+e)} . C$

Moreover, the different values of C_1 suggested by different researchers such as Lane and Washburn (1946) reported by Lambe and Whiteman (1979) is varied from 0.01 to 0.42 with and average value of 0.16, whilst Holtz and Kovacs (1981) suggested a range of 0.004 to 0.12 with an average value of 0.01.

Bouchedid and Humphrey (2005) have worked on the permeability of granular subbase materials for Maine Roads. They stated that coefficient of permeability (k) is the function of coefficient of uniformity (C_u) and percentage [11]fines (F) in the granular mixes and suggested a correlation is as follows:

$$\text{Log}(k) = -2.74487 - 0.0939125 \times F - 0.00743402 \times Cu \qquad (4.2.47)$$

Where, k is the coefficient of permeability in cm / sec.

F is the percent fines in percentages

C_u is the coefficient of uniformity (D_{60} / D_{10})

The limitations of the equation are fines content should be 3 % to 14%, the value of Cu should be 10 to 80 and aggregate should be compacted and semi - rounded.

4.3 Subgrade strength at field conditions

4.3.1 Computation of subgrade strength at Field conditions:

The subgrade strength is the most important parameter that is used for design of pavement. This is usually measured by the California Bearing Ratio (CBR). CBR values depend not only on soil type but also on density, moisture content and method of preparation of the samples. Sood & Joshi (1995) have collected data from field investigations, laboratory tests and have developed a correlation by regression analysis for the computation of unsoaked CBR at field conditions. The parameters considered in developing the relationship are field dry density, field moisture content, sand content, fraction retained on 2.36 mm sieve, fraction passing through 75 micron sieve and plasticity index. The correlation is:

$$CBR = -14.004 + \frac{0.345(+2.36mm\#)}{4.55} + \frac{0.141(SC)}{4.05} + \frac{0.154(PI)}{2.38} + \frac{17.247(FDD)}{2.30} - \frac{0.345(FMC)}{2.56} \qquad (4.3.1)$$

$R^2 = 0.732$

Where, SC = Sand content (%), PI = Plasticity index, FDD = Field dry density (gm / cc)

[11] Percent fines (F) means materials passing through mesh size 0.075 mm

FMC = Field moisture content (%), CBR = Unsoaked at field conditions (%).

+2.36# = Fraction retained on 2.36mm # (%).

Moreover, Morin and Todor (1977) have tried to establish a correlation between soaked CBR values at optimum moisture content, maximum dry density, plasticity index and percentage passing for tropical African and South American soils, but no well – defined relationships was found.

However, they found a better correlation when optimum moisture content was taken into account and the relationship is as follows

CBR = 21 – 16 Log (OMC) + 0.07LL (4.3.2)

The above correlation was derived from fine grained soils whose CBR value was less than 9. They also suggested using the correlation for preliminary strength identification of subgrade materials. The above model can be used for the rapid determination of the in situ unsoaked CBR.

Most of the soils of Bangladesh are fine soils containing more than 80 percent of grains passing through sieve no 200 (mesh size 0.075 mm) and a tentative relation has been established between soak CBR, Maximum Dry Density (MDD) and Plasticity Index (RMSS 1994). The relationship is shown in Figure 4.11.

The difference between percentage fines (f = % <0.075mm) and the percentage of clay (c= % <0.002mm) is significant and if the difference is less than 60, the soaked CBR value will be greater than 7%.

Kleyn and Van Heerden have established a relationship between DCP value and CBR as shown in Figure 4.12. Austroad Pavement design guide (2004) has suggested a relationship between insitu CBR value and DCP test results for fine grained subgrade soils is given in Appendix H, Figure B- 4.3.

Chen et al. (1999) have derived a correlation between CBR and DCP value:

$\text{Log CBR} = 2.20 - 0.71*(\text{Log DCP})^{1.5}$ (4.3.3)

Where, DCP is the rate of penetration (mm / blow).

Besides these, Harison (1987) has derived correlations between CBR and DCP value for clay like soils, well graded sand and gravel and a general correlation of acceptable accuracy has been established and represented the correlation by a log – log model than by the inverse model are shown in Table 4.2 .

Table 4.2: Empirical correlations between CBR – DCP value (Harison, 1987)

Materials types	Log – Log Model	Inverse Model	Equation No.
Clay like soils	Log.CBR = 2.56 – 1.16 (log.DCP), where R* = 0.967, Se = 0.09	CBR = $257(DCP)^{-1}$ - 1.2075, Where R = 0.97; Se = 1.96	4.3.4
Sand S – W	Log.CBR = 3.03 – 1.51 (log.DCP), where R* = 0.92, Se = 0.064	CBR = $513(DCP)^{-1}$ - 16.7, Where R = 0.91; Se = 6.84	4.3.5
Gravel G – W	Log.CBR = 2.55 – 0.96 (log.DCP), where R* = 0.96, Se = 0.04	CBR = $333(DCP)^{-1}$ – 7.80, Where R = 0.97; Se = 4.8	4.3.6
Combined data	Log.CBR = 2.81 – 1.32 (log.DCP), where R* = 0.98, Se = 0.091.	CBR = $403.4(DCP)^{-1}$ – 5.86, Where R = 0.97; Se = 6.01	4.3.7
Soaked samples	Log.CBR = 2.76 – 1.28 (log.DCP), where R* = 0.97, Se = 0.097	-	4.3.8
Unsoaked samples	Log.CBR = 2.83 – 1.33 (log.DCP), where R* = 0.99, Se = 0.086	-	4.3.9

R* is correlation coefficient

Se is standard error of estimates.

Besides these, Riley et al. (1987) have established a relationship between CBR and DCP value based on field and laboratory tests made on soils in various countries is as follows

$$CBR = 240\ (DCP)^{-1.18} \tag{4.3.10}$$

Lavneh (1987) established a correlation based on undisturbed, remoulded and also sand and granular materials between DCP and CBR is

$$Log.CBR = 2.2 – 0.71\ \log\ (DCP)^{1.5} \tag{4.3.11}$$

A good agreement exists for CBR values in the range of 5 – 80 percent and also expressed that the DCP estimation will be more conservative when CBR values is less than 5 percent (Lavneh, 1987).

Besides these, the US Army Corps of Engineers converted the penetration rate to the CBR value for all types of soil using the following relationship:

$$CBR = 292\ (DCP)^{-1.12} \tag{4.3.12}$$

Webster et al. (1992) has suggested the following empirical correlations for varies types of soil

$$\text{Log (CBR)} = 2.46 - 1.12 \log (\text{DCPI}) \quad (4.3.13)$$

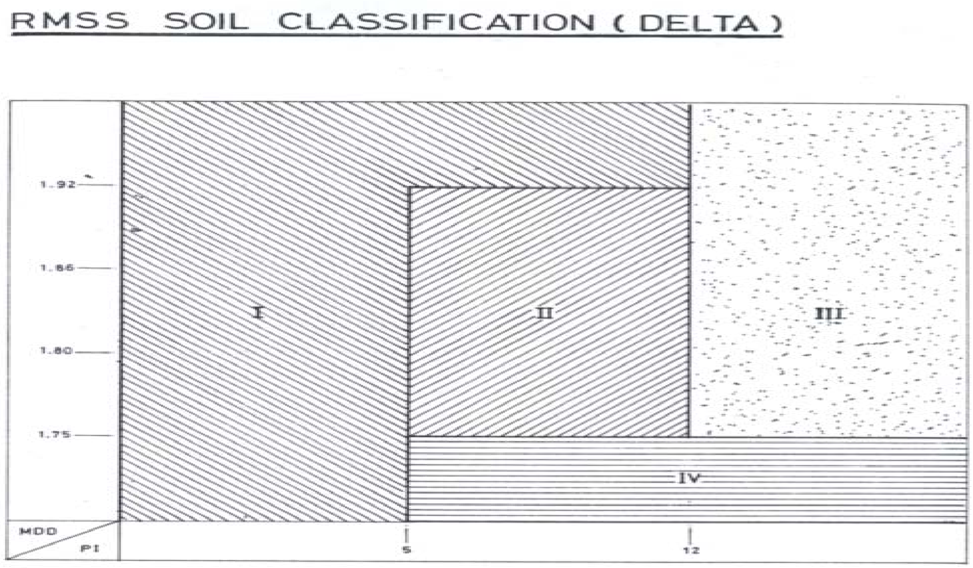
Where, DCPI is the rate of penetration in inches (inch / blow)

Coonse (1999) has also derived a correlation for Piedmont residual soil is as follows

$$\text{Log (CBR)} = 2.53 - 1.14 \text{ Log (DCPI)} \quad (4.3.14)$$

The CBR values found from the Kleyn & Van Hearden graph and Harison's inverse model are almost similar but these values are higher than those obtained from Riley et al.(1987) equation that shows more conservative value of CBR. However in Bangladesh, Kleyn & Van Hearden graph is widely used to determine the CBR value from DCP value that is suggested by Standard Test Procedure of RHD. The disadvantages of the Kleyn & Van Hearden graph are that CBR value cannot be estimated when DCP value is less than 3 (three) and it gives more CBR value than [12]Riley et al. equation. Considering these disadvantages, it is suggested to use Peterson & Miller's equation to estimate CBR value from DCP value and hence it is used in this research.

[12]Riley et al. equation was derived for the soils of various countries of the world based on field and laboratory test.



LEGEND

CLASS	DELTA	PI	MDD	REMARKS
I	< 60			CBR ≥ 7
II	≥ 60	$< 5-12$	1.75-1.92	$3 <$ CBR < 7
III	≥ 60	≥ 12	≥ 1.75	SUITABLE FOR LIME TREATMENT
IV	≥ 60	≥ 5	< 1.75	TO BE AVOIDED

Figure 4.11: Subgrade soil classification (RMSS, 1994).

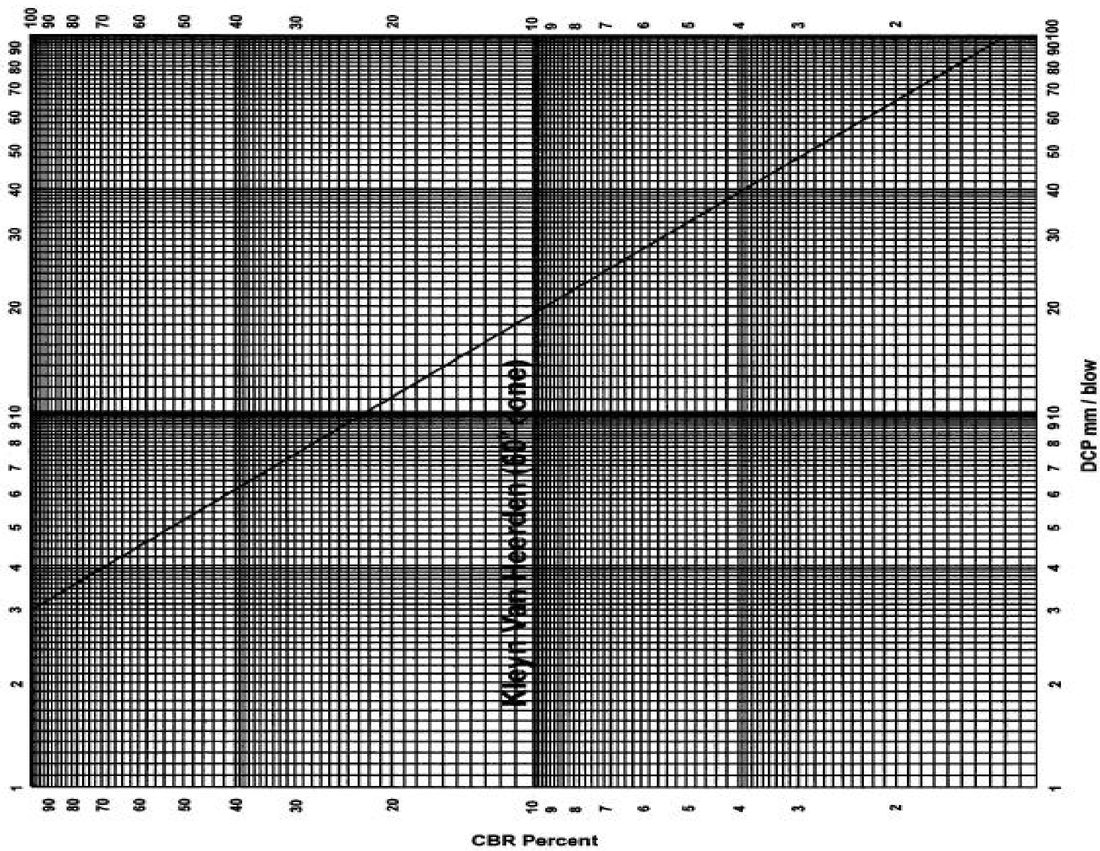


Figure 4.12: Kleyn & Van Hearden chart for DCP – CBR Relationship.

4.4 Computation of subgrade Stiffness at Field conditions:

Various empirical correlations are available to determine the resilient modulus of subgrade soils. These correlations are discussed in the following subsections:

4.4.1 California Bearing Ratio (CBR) models:

The following empirical relationship between Resilient Modulus (M_R) and CBR value of subgrade soils (Powell et al., 1984; Brown el al. 1990) are used to estimate the M_R value:

M_R (MPa) = 10 x CBR for CBR < 5 % (4.4.1)

M_R (MPa) = 17.6 x $CBR^{0.64}$ for CBR ≥ 5 % (4.4.2)

4.4.2 Soil properties models:

In Mechanistic – empirical (M-E) design, the resilient modulus is estimated by using the following generalised k_1-k_2-k_3 constitutive model (Mohammed et al., 2006):

$$M_{Ropt} = k_1 . p_a . \left(\frac{\theta}{p_a}\right)^{k_2} . \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{4.4.3}$$

Where, M_{Ropt} = resilient modulus at optimum moisture content.

K_1, k_2 and k_3 = regression parameters, P_a is the atmospheric pressure, θ is the bulk stress

τ_{oct} = octahedral shear stress.

But the mechanistic – empirical pavement design guide has considered the seasonal variation in properties of unbound materials by adjustment of the resilient modulus is given as follows:

$$M_R = 10^{a + \frac{b-a}{1+\exp[\beta + k_s.(s-s_0)]}} M_{Ropt} \tag{4.4.4}$$

The guide recommended the values of above parameters for the fine- grained subgrade materials are as follows:

a = -0.5934, b = 0.4, β=-0.3944, K_s = 6.1324

So with this known parameter and the variables like the degrees of saturation are evaluated in laboratory, the resilient modulus at any degrees of saturation can be determined.

4.4.3 Other models:

Empirical correlation between resilient modulus and DCP value for subgrade soils has been proposed by Powel (1987) as noted below:

$$M_r = 338\,(DCP)^{-0.39} \tag{4.4.5}$$

Limitations of the equation (4.4.5) are 10mm / blow < DCP < 60 mm/blow and CBR values: 30 > CBR > 5

It is noted that whenever the subgrade modulus determined from the equation (4.4.5) is to be used in the AASHTO Design Guide equations, it needs to be multiplied by 0.33 to make it compatible with the laboratory subgrade modulus used to develop the AASHTO equations.

However, Chen and Bilyeu (1999) have proposed the same equation (4.4.5) for FWD back calculated Resilient Modulus (MPa) and DCP value (mm / blows).

Beside this Pen (1990) has proposed another relationship between subgrade elastic modulus (Es) in MPa and DCP value (mm / blows) is as follows

$$\text{Log (Es)} = 3.652 - 1.17 \text{ Log (DCP)} \quad (4.4.6)$$

Chai and Roslie (1998) have developed a correlation between resilient modulus and DCP value for subgrade soil in the following form

$$E \text{ (MPa)} = 17.6 \, (269 / \text{DCP})^{0.64} \quad (4.4.7)$$

Again they have developed a relationship between the backcalculated modulus and the DCP value in the following form

$$E \text{ (back)} = 2224 \, (\text{DCP})^{-0.996} \quad (4.4.8)$$

Where, E (back) = Backcalculated subgrade modulus (MPa).

Jianzhou et al. (1999) found that there was a strong relationship between DCPI and the FWD- backcalculated moduli in the following form

$$E \text{ (back)} = 338 \, (\text{DCPI})^{-0.39} \quad (4.4.9)$$

George and Uddin (2000) developed relationships between resilient modulus and DCP for fine grained soils is as follows

$$M_R = 532.1(\text{DCP})^{-0.492} \quad (4.4.10)$$

They also developed a correlation for coarse – grained soils in the following form

$$M_R = 235.3(\text{DCP})^{-0.475} \quad (4.4.11)$$

The California Bearing Ratio models (Powel et al., 1984 and Brown el at.1990) are frequently used in Bangladesh for resilient modulus estimation as the equations are simple and does not need cyclic triaxial test. Besides this, for quick determination of resilient modulus from DCP test, the above mentioned equations can be used for making comparison that determined from this study.

4.4.4 Relationship between Resilient Modulus and Unconfined Compressive Strength for subgrade soils:

Empirical correlations for resilient modulus have been proposed by a number of researchers (Heuklelom and Klomp1962; Fredlund et al.1977; Jones and Witczak1977; Thompson and Robnett 1979; The Asphalt Institute 1982; Powel et al. 1984; Brown1990; Drumm et al. 1990). According to Lee (1997), data for most of the correlations show significant scatter and the correlations are reasonable only within a certain range of variables and many of the correlations are not considered the stress level. But Lee (1997) has developed a correlation between resilient modulus and unconfined compressive strength for Washington, South Bend and Bloomington clayey subgrade soils is as follows

$$M_R = 695.4\ (S_{u1.0\%}) - 5.93\ (S_{u1.0\%})^2 \qquad (4.4.12)$$

Where, M_R = Resilient Modulus at maximum axial stress of 41.4 kPa and confining stress of 20.7 kPa (3 psi); $S_{u1.0\%}$ = Stress causing 1% strain in conventional unconfined compressive strength test (in psi). The same relationship is also presented in Appendix-H, Figure B- 4.4.

The above mentioned correlation is applicable for different types of clayey soils and the relationship is unique regardless of moisture content and compaction efforts as suggested by Lee (1997).

4.5 Correlations for strength and stiffness of unbound base and subbase materials

In Bangladesh, road subbase and base are generally consisted of unbound granular layers. In most of the developing countries, CBR test is most widely used for pavement design. But due to the lack of equipment for conducting the insitu CBR test in the field, DCP test is done to find out the field CBR value. Different empirical correlations are available for estimating the CBR values are discussed in the following paragraphs:

Ese et al. (1995) have derived empirical correlations between CBR value and DCP value for aggregate base course is as follows

$$\text{Log (CBR)} = 2.44 - 1.07\ \text{Log (DCPI)} \qquad (4.5.1)$$

Again, NCDOT (Pavement, 1998) has also established a correlation between DCP value and CBR value for Aggregate base course and cohesive is as follows

$$\text{Log (CBR)} = 2.60 - 1.07\ \text{Log (DCPI)} \qquad (4.5.2)$$

The design of pavement using CBR method is empirical one. But world is moving from empirical method to analytical method that consider the stress – strain relationship of materials. The stress – strain relationship of unbound granular materials is characterised by the resilient modulus which is used as the input parameter for the design of analytical pavement design. The resilient modulus of unbound materials depends on the overall stress to which they are exposed, increasing with increasing stress.

A large number of investigations were conducted to determine the stress – strain response of the unbound granular materials. Some of these investigations and correlations are described as follows:

4.5.1 Rada and Witczak model

Rada and Witczak (1981) have done a comprehensive evaluation of laboratory Resilient Modulus for granular materials and investigated the accurate correlations between M_r and CBR.

$$M_r = k_1 \ \theta^{k_2} \tag{4.5.3}$$

Where, k_1 and k_2 are the materials constant, θ is the bulk stress.

The value of k1 and k2 for different aggregate class have determined and presented in Table 4.3. From the table it is seen that the value of k2 decreased with increasing the value of k1. Another researchers, Flintsch et al. (2005) have determined the k1 and k2 value for unbound subbase layer is shown in Table 4.4 and from the table it is observed that the value of k2 decreased with increasing the value of k1.

Table 4.3: Summary of the k1 and k2 value for different class of aggregates (Rada & Witczak)

Aggregate class	No. of data points	K1 parameter	K2 parameter
Silty sands	8	1620	0.62
Sand gravel	37	4480	0.53
Sand – Aggregate blends	78	4350	0.59
Crushed stone	115	7210	0.45
Lime rock	13	14030	0.40
Slag	20	24250	0.37
All data	271	9240	0.52

Note: Applicable to Mr= k1 θ^{k2} where, Mr and θ are in psi.

Finally Rada & Witczak (1980) have developed an empirical equation relating to M_r, θ and CBR.

$$M_r = (490 \log \theta - 243)\ \text{CBR} \tag{4.5.4}$$

Where, θ = 10 psi used for most highway subbase design.

θ = 20 – 40 psi used for most highway base design

Besides this, Monismith (1992) has been established a relationship between resilient modulus and sum of the principal stress that is shown in Figure 4.13.

Table 4.4: The value of k1and k2 constants from k - θ Model for granular subbase layers (Flintsch et al. 2005)

Relative Compaction (%, ASTM D1557)	Dry density (kN / m^3)	K1 value	K2 value	R^2 value
96	23.6	4898	0.656	0.991
94	23	7568	0.59	0.979
96	23.6	3846	0.703	0.984
97	23.8	3357	0.583	0.967
92	22.5	9016	0.558	0.969
95	23.3	15346	0.489	0.947
97	23.7	7362	0.609	0.993
97	23.7	8222	0.589	0.986
93	22.8	6427	0.623	0.981
91	22.8	6998	0.600	0.977

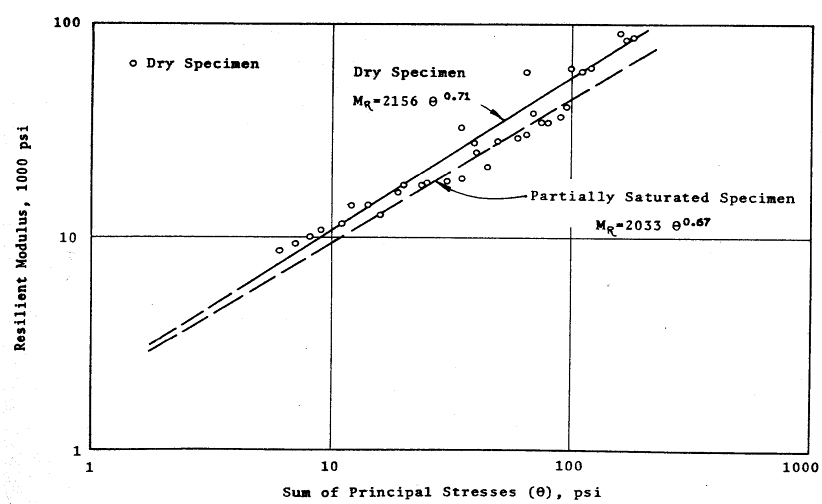


Figure 4.13: Resilient modulus of unbound Granular base (k- θ model) Vs some Principal stresses (Monismith, 1992)

There is also a relationship between subgrade modulus and granular layers (subbase and base layer) composite modulus and from that relationship modulus of the granular layers can be determined. The relationship is as follows:

$E_2 = E_3 \times 0.2 \times h^{0.45}$ (4.5.5)

Where E_2= Composite modulus of subbase and base (MPa)

E_3= Modulus of subgrade (MPa), h = Thickness of the granular layers (mm)

As E_3 is known, modulus of the granular layers can be determined using the above equation.

O'Flaherty (1991) stated that input data for granular base and subbase used in the standard design curves are as follows:

$E_n = 3\ E_{n+1}$ for $E_{n+1} \leq 50$ MPa (4.5.6)

$E_n = 150$MPa for $E_{n+1} > 50$ MPa (4.5.7)

Where E_{n+1} is the modulus of the underlying layer and the upper limit for the thickness of the compacted layer is 225mm and the each layer is compacted separately.

The resilient modulus of a granular layer can be calculated using the Dormon & Metcalf's (1965) equation

$$E_g = \left(\frac{h_g}{35.75mm} \right)^{0.45} xE_s \qquad (4.5.8)$$

Where E_g = Modulus of granular layer, E_s = Modulus of subgrade, h_g = thickness of the granular layer (mm)

From the equation (4.5.8), it is observed that the elastic modulus of granular layer is only depended on the thickness of the layer and stiffness of the support but is independent of the properties of granular layer.

Rada and Witczak model (1980) gives separate values of resilient modulus for subbase and base layers. As in Bangladesh, the subbase and base layers are constructed with separate materials, for this the resilient moduli of the two layers are different.

4.5.2 Relationship between strength properties of aggregates

Aggregates strength is measured by Aggregates crushed value (ACV) and Ten percent fines value (TFV). Weinert (1980) has developed a correlation between ACV and TFV is shown in Figure 4.14. This relationship was derived for crushed granular stone aggregates.

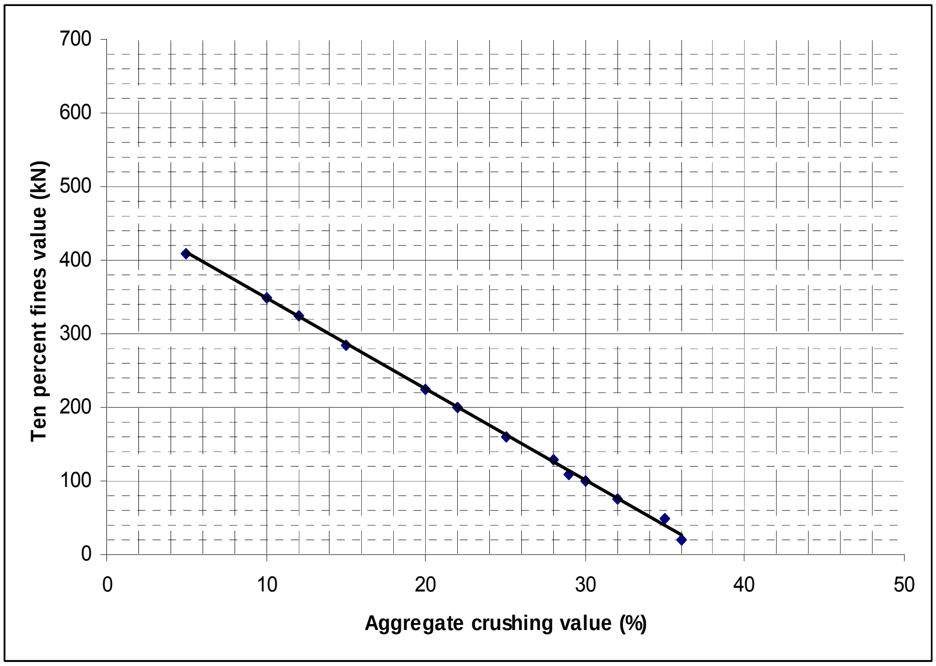


Figure 4.14: The linear relationship of aggregate crushing values and ten percent fines values (Weinert 1980)

4.5.3 Others model

Various correlations were suggested by different researchers (Elia et al. 2006, Nazzel et al. 2007) for estimation of resilient modulus. Elia et al (2006) have derived an empirical correlation for unbound granular materials and relating the regression constants with materials properties as well as compaction characteristics. On the hand, Flintsch et al. (2005) have correlated the resilient modulus with degrees of saturation and relative compaction. Nazzal et al. (2007) have conducted light falling weight Deflectometer (LFWD), falling weight Deflectometer (FWD) and Dynamic Cone Penetrometer (DCP) tests on different pavement materials included base, subbase and subgrade materials at several highway sections in Louisiana and found good correlation between them. The correlations are as follows

$$E_{LFWD} = \frac{5301.54}{8.31 + (DCP)^{1.44}} \tag{4.5.9}$$

Where, E_{LFWD} is the elastic modulus found from LFWD test, DCP is the DCP value in mm / blow and the equation has R^2 value 0.87.

Again they suggested a relationship between resilient modulus found from FWD test and the elastic modulus found from LFWD test is as follows

$$M_{FWD} = 0.964E_{LFWD} \quad (4.5.10)$$

Where, R^2 value is 0.94.

However, Fleming (2000) was proposed a similar equation based on the results of several field test conducted on different subgrade soils is as follows

$$M_{FWD} = 1.031E_{LFWD} \quad (4.5.11)$$

Most of the granular base materials were best represented by the following equation (Mahoney, J.P. and Jackson, N.C. 1990).

$$E_{BS} = 8500\theta^{0.375} \quad (4.5.12)$$

Where, E_{BS} is the resilient modulus of the coarse grained materials and soils (psi) and θ is the bulk stress (psi).

4.6 Correlations for Bituminous layer

The pavement surface deflection is measured by applying the static or dynamic load to the pavement surface and measuring the corresponding pavement deflection. The area of pavement deflection under and near the load application is collectively known as the deflection basin. Deflection measurement can be used in back calculation methods to determine pavement structural layer stiffness and the subgrade resilient modulus. Thus many characteristics of a flexible pavement can be determined by measuring its deflection in response to load.

There are three categories of non-destructive deflection testing equipment such as static deflections; steady state deflection and impact load deflection are the Benkleman beam, Dynaflect & Road Rater and Falling weight Deflectometer (FWD) respectively.

The correlations between Benkleman Beam and Falling Weight Deflectometer are developed for specific sets of conditions that may not be present for those using the correlation. The relationship is

BB = 1.33269 + 0.93748 (FWD).

(4.5.13)

Where, BB = Benkleman Beam deflection (inches x 10^-3)

FWD = FWD centre of load deflection (inches x 10^-3)

For the bituminous layer, the design parameter is the dynamic modulus of bituminous mixes and Poisson's ratio. The Witczak prediction equation (reported by Loulizi et al. 2006) for determination of dynamic modulus is as follows:

$$LogE^* = 3.750063 + 0.02932\rho_{200} - 0.001767(\rho_{200})^2 - 0.058097Va - 0.802208\left(\frac{V_{beff}}{V_{beff}+V_a}\right)\left[3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}\right] \div 1 + e^{-0.603313-0.313351Log(f)-0.393532Log(\eta)} \quad (4.5.14)$$

Where, E* = dynamic modulus (psi), P200 = % passing no 200 sieve

P4 = cumulative % retained on No 4 sieve, P34 = cumulative % retained on No ¾ sieve, P38 = cumulative % retained on No 3/8 sieve, f = frequency (Hz), V_{ef}=effective bituminous content (% of volume), Va = air void content, η = bitumen viscosity (10^6 poise). Besides this, using penetration index (shown in Appendix H, Figure B-4.5) and temperature difference, stiffness of the bituminous materials can be determined by Van der Poel, (1954) chart (shown in Appendix H, Figure B-4.6). Now using this stiffness modulus of binder, volume of binder and volume of mineral aggregate; the stiffness modulus of bituminous mix can be determined with the help of Shell chart (1978) that is presented in Appendix H, Figure B-4.7.

4.7 Summary

In Bangladesh, there is a lack of necessary field and laboratory equipment for performing the appropriate test for determination of engineering properties of soils and materials. For this reason, empirical equations are inevitable for estimating of the design input parameters or making comparison for the test results undertaken by this study. Various empirical equations are available for the above mentioned tasks. These equations are derived for different types of soils and materials from different parts of the world. So, considering the properties of soils and granular materials found in Bangladesh; the most suitable empirical equations are used for estimation of design input parameters. It is mentioned that some of correlations relating to the engineering properties of soils and granular materials are reported in Appendix – B as these correlations are not used in this works but may be used in future RHD road and embankment design project for Bangladesh.

Chapter 5: Review of Field and Laboratory Testing Method

5.1 Introduction

This chapter contains the available test methods for determining of the properties of natural soils, granular materials (both gravel and bricks) and bituminous materials used in pavement and embankment construction in Bangladesh. It also deals with the advantage, disadvantage and limitation of the different testing methods. It is possible to use only a limited number of tests although many test methods have been used in this study, emphasis has been given to conduct index type of tests for purposes of developing correlations. It should be appreciated at output that may of the tests conducted and repeated herein are not routinely conducted. The requirement of any design test is that it should be carried out with equipment that is either readily available and can be operated effectively by testing personnel of normal quality. Besides this the tests should have considerable reproducibility in order to minimize the considerable amount of testing inevitable on the varying soils normally encountered on a road pavement.

5.2 Method of Testing

Properties of soils can be determined either insitu or exsitu (laboratory). Insitu techniques are used to measure the properties of materials without measuring the techniques such as geophysics. Local tests (e.g. plate bearing, dynamic cone penetration test, and static cone penetration test) and permeability determinations may be undertaken by insitu test. Laboratory test requires recovery of materials through a range of ground investigation methods. Recovered specimens are than tested in a laboratory. Laboratory tests are further divided into two groups: these that are used to characterize a material to enable its classification and these that are used to ascertain engineering properties, may be used in design.

5.3 Insitu Testing for Soils & Granular Materials

The strength characteristics of cohesive soils can be determined using either laboratory or in situ tests. The soil samples recovered from the field are inevitably subjected to a certain degree of disturbance during sampling, handling and transportation. Besides this laboratory testing on small intact samples on soils with some fissures or layering can be misleading. Some are impossible to sample without significant disturbance. Meanwhile, insitu tests can provide more accurate and reliable measure of properties of soils. According to Clayton and Matthews (1995), field testing is desirable for very soft or sensitive clays, sands and gravels and stony types of soils to determine their engineering properties. On the other hand, the actual stress – strain behaviour of the materials can be found from the field test. Some insitu tests for soils and granular materials are discussed in the following Table 5.1:

Table 5.1: Insitu tests for soils and granular materials

Name of the Tests	**Test Standards**	**Advantage, Disadvantage and Limitations of the test**
Field Vane shear test	BS 1377: part 9:1990	The vane shear test is one of the most important techniques for determining insitu strength of soils. Robust, simple standardized test that is easy to carry out. It is particularly useful for determining the undrained shear strength of soft to medium cohesive soils. The field vane shear test is most useful where considerable variation in the undrained shear strength can be found with depth (Ladd et al.1980). Hence it is used to establish the change pattern of undrained shear strength with depth. But the test is normally restricted to fairly uniform, cohesive, fully saturated soils and is used mainly clay having undrained shear strength up to about 100 kPa (BS 5930:1999). The limitation of the test is that results are not accurate for stiff clay or soils contain significant amount of coarse silt or sand content. Correction of results may be required for those soils otherwise results may be misleading.

Cont…

Name of the Tests	Test Standards	Advantage, Disadvantage and Limitations of the test
Cone penetration test (CPT)	BS 1377: part 9:1990	This insitu test is used to determine the strength property of soft soil. It can be carried out quickly and large number of tests can be done in a day. As there is no need for a borehole, so disturbance affect associated with boreholes can be avoided. The advantages of CPT are rapid data acquisition, continuous record of resistance in the stratum of interest and results are accurate and repeatable. Different measurements are made which enhance interpretation. The major disadvantages of the test are it is not applicable for hard soils and interpretation of soil type producing the cone resistance requires either considerable experience or recovery of samples for correlation testing.
Standard Penetration test (SPT)	BS 1377: part 9:1990	The main advantage of this test is simple, inexpensive and feasible to carry out wide range of materials. Besides this, it also gives the shear strength of subsoil containing significant amount of silts, sand and gravel where it is not possible to collect soil samples. It also gives the idea about compactness or looseness of the soil deposits. The disadvantages of the test are not applicable for gravely soils. The result is affected by borehole disturbance such as piping, base heave and stress relief. Many correlations are required for interpretation and design.

Cont…

Name of the Tests	Test Standards	Advantage, Disadvantage and Limitations of the test
Dynamic Cone penetrometer test	STP[13] of RHD	This DCP test is the solution for rapid and economical estimation of strength (CBR value) of pavement layers consists of granular materials and subgrade soils. The DCP has the ability to verify both the level and uniformity of compaction, which makes it an excellent tool for quality control of pavement construction. Besides this, it can also be used to determine the tested layer thickness (Chen et al., 2001). Harrison (1986) & Livneh et al. (1989) also found that there is a strong correlation between CBR and DCP penetration. The disadvantage of this test is the difficulties experienced in penetrating the granular materials during the test.
Insitu unit weight measurement	BS 1377: part 9:1990/ STP of RHD	For assurance of quality control in the field, insitu density test is widely used for the construction of granular pavement layers. Various methods such as Core – Cutter method, Sand – replacement method, volumenometer method, Rubber – Balloon method and nuclear density method are available for measuring the in situ density. Sand replacement and the core cutter methods are used frequently due to their availability. These two methods are the destructive test. While nuclear density method is the non destructive test used now a day for measuring the density of both granular materials and subgrade soils.

[13] STP means Standard Test Procedure for Roads and Highways Department of Bangladesh

5.4 Insitu testing of Bituminous Layer (s):

The properties of bituminous materials can be determined by both field and laboratory test methods and using either destructive or non destructive techniques. The Falling Weight Deflectometer (FWD) test, Benkleman beam test and Dynaflect and the Road Rater are the three non – destructive tests usually done on pavement layers to evaluate their properties. With FWD test, surface deflections are measured at different locations of deflection bowl. These deflections are used for determining the elastic modulus of bituminous layers and the underlying pavement layers. On the other hand, maximum deflection of the pavement can be measured by Benkleman Beam test. The dynamic deflection of a pavement produced by an oscillating load is measured by steady state deflection equipment known as Dynaflect and the Road Rater. Only Benkleman Beam was used in this study as per discussed stated in chapter two.

5.5 Laboratory Testing for Soils & Granular Materials

Most of the engineering properties of soil and granular materials are determined by laboratory testing. The laboratory tests are usually conducted on selected samples extracted from the field. Index properties, strength and settlement properties of soils are determined in laboratory while the strength, resilient modulus and gradation properties of granular materials are also determined in laboratory. The standard test methods, their advantages, disadvantages and limitations are discussed in Table 5.2:

Table 5.2: Laboratory tests for soils and granular materials

Name of the Tests	Test standard	Purpose, advantage, disadvantage and limitations of the tests
Atterberg limit test	BS 1377-2:1990/ ASTM D4318- 84	Atterberg limits are used to classify the soils. This test is applicable for cohesive soils. The liquid limit and plastic limit tests only be performed on that part of soils which passes through no. 40 sieve (0.425 mm). Therefore for many soils, a significant part of the soil specimen (i.e., particles larger than 0.425 mm) are excluded during testing; hence causes problem when classify the soils (Robert,

		2000). The Atterberg limit give no indication of particle fabric or residual bonds between particles which may have been developed in the natural soils but are destroyed in preparing the specimen for the determination of limits (Lambe & Whitman, 1979).
Organic content test	BS 1377-3:1990 / ASTM D2974 - 87	There are three methods for determination of organic content of soils such as Loss on ignition, Oxidation with hydrogen peroxide and Dichromate oxidation. Both of the oxidation methods are used to oxidize the organic matters present in the soils mass. According to Alexander & Byers (1932), oxidation is incomplete and results obtained therefore too low. On the other hand, organic matter is destroyed by ignition which results in quantitative oxidation but break down inorganic constituents as well. So, the loss of ignition method is however suitable for determination of organic content of soils (Norman, 2007).
Particle Size Distribution test	BS 1377-2:1990	The Particle Size distribution (PSD) test is one of the most important tests that are used in almost all soil classification systems for both coarse and fine grained soils. Range of particle size present in a soil can be ascertain by undertaking grading analysis using sieve down to about 0.06 mm. Between 0.06 mm to 0.002 mm size, a hydrometer is used. The test procedure of hydrometer is however approximate because many fine soil particles are plate like shape rather than sphere (Robert, 2006). Both Sieve analysis and hydrometer analysis give the continuous grading of the particles presence in the materials.

Cont…

Name of the Tests	Test standard	Purpose, advantage, disadvantage and limitations of the tests
Vane shear test	BS 1377-7:1990	It is the most widely used versatile devices for quick investigation of undrained shear strength and sensitivity of soft deposits of clay (Carlson 1948, Cadling and Odenstad 1950). The small size of the laboratory vane makes the equipment unsuitable for testing samples with fissuring for fabric and therefore it is not very frequently used. According to Leussink & Wenz (1967), the vane shear results for organic deposits are unreliable due to the distortions caused by the fibrous character of soils. Bjerrum (1972) demonstrated that it gives the over estimated shear strength for high plasticity clay and therefore proposed an empirical correction factor discussed in chapter four.
Unconfined compressive strength (UCS) test	BS1377 – 7:1990	The UCS test is popular due to its simplicity and quickness. Another advantage of this test is failure surface will tend to develop in the weakest plane of the cohesive soil specimen. So the test is most frequently performed on cohesive saturated soils; because the specimen must be able to retain its plasticity during the application of the vertical pressure. For soft clay, UCS test is not reliable. Its strength depends on the arbitrary standard that the load required to produce 20% strain is the actual shear failure. The limitation of the test is the definition of the Cu which is assumed to be equal to the maximum shear stress.
Quick undrained triaxial test	BS 1377-7:1990	The triaxial shear test is one of the most widely used conventional and reliable methods for determining the undrained shear strength of soils. This test is performed on undisturbed or remoulded soil samples with fully saturated or partially saturated. Depending on the simulation of the

		field conditions; the confining pressure on the sample should be maintained at about 0.5 σ_v, σ_v and 2 σ_v where σ_v is the total vertical insitu stress. For compacted soils the cell pressure should be maintained to the estimated total stresses likely to occur in the field conditions. The size of the specimen has a significant effect on the results of this test (Bishop et al. 1965; Agarwal 1968; Marsland and Randolph 1977). If the soil samples are fully saturated, the angle of internal friction should be zero. But practically the remoulded soil samples are not fully saturated. When the load is applied on the partially saturated soil sample, the soil particles exhibit friction among them resulting insignificant value of angle of internal friction have been arisen.
Direct shear box test	BS1377 – 7:1990	The drained shear strength parameters are determined by this test. The test is very simple and easy to carry out in a short duration of time. The shearing resistance offered by a soil is a very important factor in the design of earth slopes for the highway embankments and thus shearing of foundation soil can result in complete pavement disintegration and the loss of embankment through sliding. This test is used to determine residual shear strength parameters for the analysis of pre – existing slope instability (Skempton 1964; Skempton & Petley 1967). This test is most suitable for dry and saturated cohesionless soils but can be done for all soils. In this test failure plane is predetermined horizontal plane, but in actual field condition, this failure plane may not be an actual failure plane. Though it is simple test, the reliability of the result of the test may be questioned because the soil is not allowed to fail along the weakest plane.

Ring Shear test	BS 1377 – 7:1990	The drained residual shear strength parameters can be determined from this test. This test is suited to the relatively rapid determination of residual shear strength and the capacity of testing under different normal stresses to quickly obtain the shear strength envelope. Because of the short drainage path through the thin remoulded cohesive soils specimen. The drained ring shear test is an accurate baseline value for the drained residual strength that can be used to evaluate the accuracy of the direct shear and centrifuge test results. The fast ring shear tests are useful because they provide an improved understanding of the fast residual shear behaviour along discontinuities. The advantage of the test compared to the other devices is that the ring shear apparatus can be used for very long and almost unlimited of shear displacement (Agung et al., 2001). Besides this, the tests have proved to be reliable for simulating close to rainfall induced landslides (Fukuoka et al., 2009).
Drained triaxial test	BS 1377 – 8: 1990	The consolidated drained triaxial (CU) test and consolidated undrained triaxial test(CU) with pore pressure measurement give the drained shear strength parameters such as drained cohesion ($C^{/}$)the and drained angle of internal friction ($\phi^{/}$). Considering the field drainage conditions, one of the above tests can be done. But according to Clayton et al. (1995), the later test is normally preferred because it can be performed more quickly and more economically. These tests give the more field representative values of drained shear strength parameters of soils.
Consolidation test	BS 1377 – 5:1990	The consolidation characteristics of soil are determined by the one dimensional consolidation test suggested by

		Terzaghi. The test results are used to calculate the magnitude and rate of consolidation of soil beneath foundations. But in practice, the flow of water and displacements which takes place during consolidation is nearly always three dimensional. These three dimensional consolidation properties can be measured in Rowe cell apertures. The analysis of three – dimensional effects is extremely complex and is rarely practicable (Davis and Poulos, 1965). Thus for most applications Terzaghi's one – dimensional analysis provides a sound basis for the estimation of the magnitude of settlements, although the rate at which they develop has to be interpreted with caution. The test is performed by means of the oedometer test. The procedure and analysis are restricted to problems involving saturated soil masses, mainly clays, fine silts and other such soils of low permeability.
California Bearing Ratio (CBR) test	ASTM D 1883 - 99	The strength of subgrade soil is measured in terms of California Bearing Ratio (CBR). Considering the worst condition of the subgrade soil, four days soak CBR test is usually determined and used in pavement design of flood prone country like Bangladesh. RMSS (1994) has reported that the ratio of unsoaked CBR to soaked CBR for subgrade soil is equal to as high as 4 (four). Compaction plays an important role on CBR. Both standard and modified compactions were done on soil samples. The total energy applied on the soils (for one litre mould) for standard and modified compaction is 551.81Joules and 2483.16 Joules respectively. So the energy applied to the modified compaction is 4.5 times higher than the standard compaction test. This test is also performed for granular base, sub base and

		improved subgrade layer of the pavement. It is the indicator of the shear strength of the layer and is done by expressing the forces on the plunger for a given penetration as a percentage of standard force. Although this test is the most widely used for assessing pavement materials strength, it has been criticised for its various limitations. Lee & Bindra (1981) and Brown (1981) observed that the CBR test gives an indication on shear strength and resistance to permanent deformation, but does not give the resilient behaviour of materials that is required for pavement design. Another weakness of the test is that it can not be performed if the particle sizes are over 20mm. However, to overcome this difficulties Lee and Bindra (1981) proposed a Modified Bearing Ratio (MBR) test that allows aggregates up to 37.5mm sizes.
Aggregates Strength tests (ACV&TFV).	BS 812-110:1990 & BS 812-111:1990	Many test procedures have been developed to evaluate the strength properties of aggregate. Some of the widely used strength tests for coarse aggregates are Aggregate Impact Value (AIV), Aggregate Crushing Value (ACV), Ten Percent Fines Value (TFV) and Los angles abrasion test. However, considering the availability of the equipment, ACV test and TFV test are conducted in RHD for this research works.
Aggregate shape test (Elongation index and Flakiness index)	BS 812-105.2:1989 and BS 812-105.1:1989	Granular aggregates may be characterised by their shapes. Aggregates angularity and flakiness index test are most frequently used shape tests for granular aggregates. These two tests provide an idea about how much elongated and flaky particles presence in the aggregate mass. The flakiness index test is not applicable to materials passing a 6.30 mm BS test sieve or retained on a 63.0 mm BS test sieve. On the other hand, elongation index test is not

		applicable for materials passing a 6.3mm sieve or retained on a 50 mm test sieve. Both of the testing methods are stated in the chapter seven.
Particle size distribution test for granular materials	BS 1377-1:1990.	Granular materials gradation can be done by sieve analysis. Standard sizes of sieves are used for this test which provides distribution of particle sizes in the aggregate mass. Different standard suggested different aperture sizes of sieve. Many researchers recommended a well- graded distribution of particles i.e. particles of all sizes in a proportion that gives a dense packed mass (Lay 1990). In that cases the larger particles are surrounded by the smaller particles that provide the better support and confinement to the interlocked larger particles (Bartley 1982). Resulting mass gives the highest density and hence highest internal friction as well as resistance to shear failure (Atkins, 1983). Barksdale and Itani (1989) have reported that with increasing density only at low values of normal stress, the resilient modulus increased markedly. Thus the particle size distribution or grading of granular materials seems to have some influence on materials stiffness. So considering the above mentioned factors, particle gradation should be selected in such a way that gives optimum strength and stiffness as well as provide good drainage facilities.
Cyclic triaxial test for soils and granular materials.	AASHTO T 307 -99	Considering the sustainability of the pavement structure, world is moving away from the empirical design of pavement foundations to develop a performance specification approach that facilitates analytical design (Frost et al., 2005). In the past, most of the highway agencies have used CBR to characterize subgrade in the design of pavements and to correlate it with the resilient modulus. CBR, however, is a static property that cannot

		account for the actual response of the subgrade under the dynamic loads of moving vehicles. As discussed earlier pavement materials are subjected to dynamic repeated loading at field conditions. To simulate the field condition, a cyclic triaxial test can be conducted in laboratory by providing different confining pressure, axial stress and cyclic stresses. The test procedure is based on the Strategic Highway Research Programme Protocol P46 in which repeated load triaxial testing is specified for determining the resilient modulus (Elias & Titi, 2006). The test method has recently been upgraded from AASHTO T 294 – 94 and AASHTO T 274. There are certain limitations inherent in using cyclic triaxial test to simulate the stress and strain conditions of a soil element in the field during earthquake. The interaction between specimen, membrane and confining fluid has an influence on cyclic behaviour. Membrane compliance effects cannot be readily accounted for in the test procedure or in interpretation of the test results. Changes in pore-water pressure can cause changes in membrane penetration in specimens of cohesionless soils. These changes can significantly influence the test results. The test is conducted under undrained condition to simulate essentially undrained field conditions.

5.6 Laboratory Tests for Bituminous Materials

The bituminous layer of pavement consists of specified graded stone aggregates and bitumen as binder material. The bituminous materials are collected from the field by core cutter machine. The stability and flow of the bituminous core samples were tested in accordance with the ASTM D 1559. Besides these, bitumen was extracted by centrifuge extraction bowls equipment at laboratory following the standard of ASTM D2172 - 05 and the aggregates obtained from this test may be used for sieve analysis. The properties of bitumen like specific gravity, penetration, softening point, fire point & flash point, solubility, ductility, viscosity and loss on heating have been determined in laboratory. These properties are determined by the specific gravity test, penetration test, ring and ball test, fire point & flash point test, solubility test, ductility, viscosity test and loss on heating test following the standard of ASTM D2726- 05a, ASTM D -5, ASTM D36- 06, ASTM D -92, ASTM D2042, ASTM D113, ASTM D2170 and ASTM D 6-95 (2006) respectively.

5.7 Summary

The properties of soils and granular materials used in embankment and pavement design have been identified. These properties can be assessed by different testing methods. Each method has some limitations, advantages and disadvantages that are reported in this chapter. Some of the inevitable testing equipments are also unavailable in Bangladesh. Considering the availability, advantages and disadvantages; tests were carried out for determination of the design input parameters. But in some cases, simple test have not always provided the design input parameters; appropriate correlations were used which were derived from similar soils and materials likely to be found in Bangladesh. The appropriate tests were conducted based on the site investigation reports discussed in the following chapter six.

Chapter 6: Review of Site Investigation method

6.1 Introduction

Methodologies used in undertaking a site investigation together with its design and planning. A good site investigation comprises of a number of stages in terms of activities and possible repeat visits. All site investigation should be undertaken to address specific needs which may include determination of properties of soils or ground water condition or others. The aims of site investigation are to determine all the information relevant to site usage, including meteorological, hydrological and environmental information (BS 5930:1999). To satisfy the aims, appropriate site investigation techniques, its planning and design are discussed in this chapter.

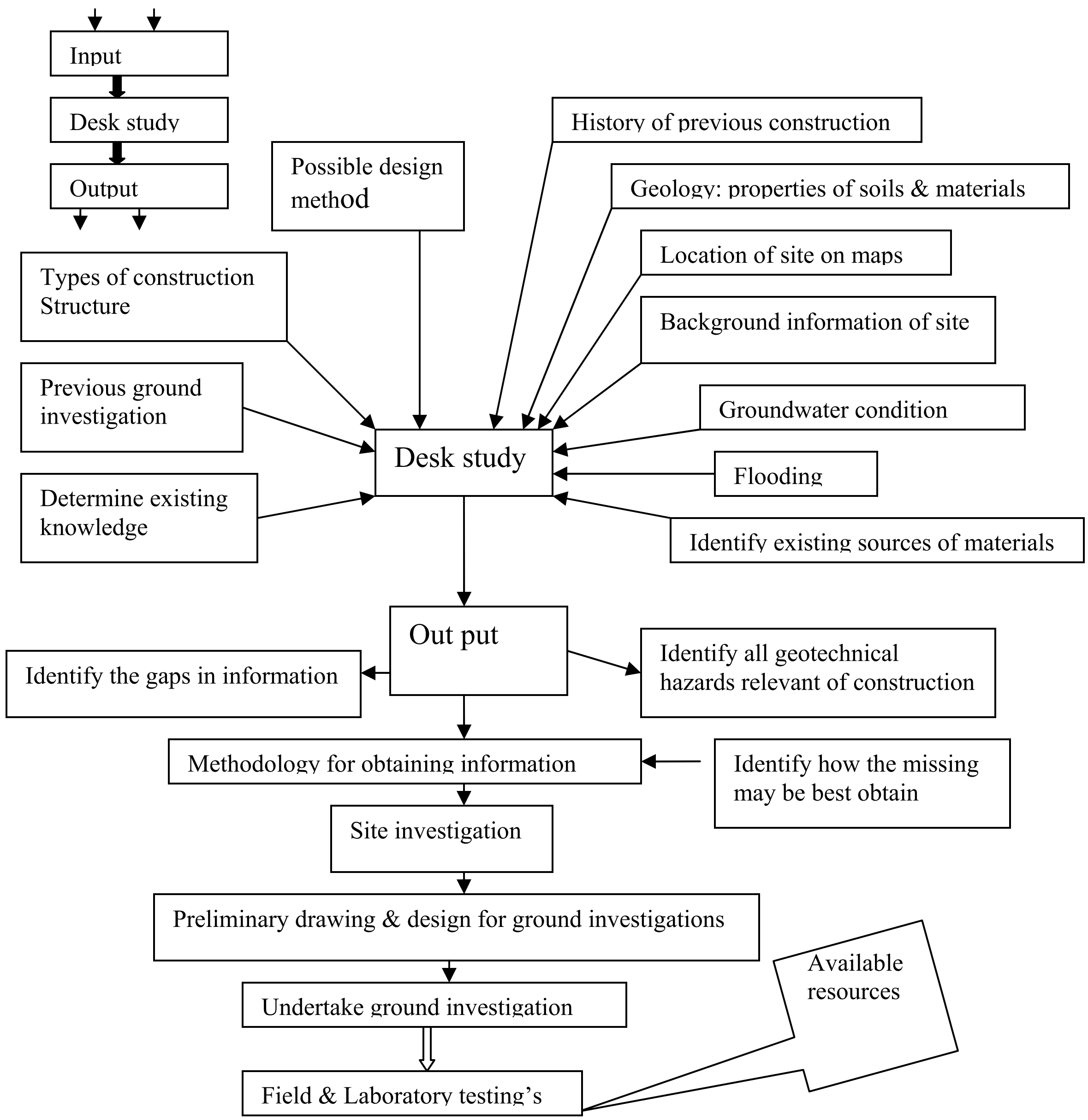
6.2 Process of Site Investigations:

Site investigation is a complex process which is vital for success of any construction project. Effective site investigation can lead to a project in a systematic way using techniques that are relevant, reliable and cost effective (Clayton et al.1995). Any geotechnical investigation should be optimized by considering relative accuracy, relative cost, availability and relevance to the problem. These should be assessed for each way of determining the design input parameters (Clayton et al. 1995). The normal site investigation work usually takes in the form of trial pits dug at various points to expose the soils at foundation level and the foundation structure together with deep trial pits or boring to investigate the full depth of soil affected by bearing pressure followed by testing and monitoring to determine relevant properties (Tomlinson, 2001). Site investigation is necessary for any road built anywhere depending on what is known about the materials and both design and construction methodologies. As per BS 5930:1999, investigation is necessary for assessment of suitability and quantity of available soils e.g. borrow pit for embankment construction and location of suitable disposal site for excess spoil. In addition to this, it also helps to match most suitable plant and equipment available with the materials. For embankment construction, prior knowledge of ground strata in the alignment of road may influence the design such as deeper cut is necessary where the soils are unsuitable and vice versa. So, the methods for road and embankment exploration are done by means of trial pits & trenches and boreholes respectively.

According to Murthy (2003), any soil exploration consists of three parts such as desk study, execution and data interpretation. All these parts are equally importance for success of any exploration programme. The details of these phases are discussed in the following subsections:

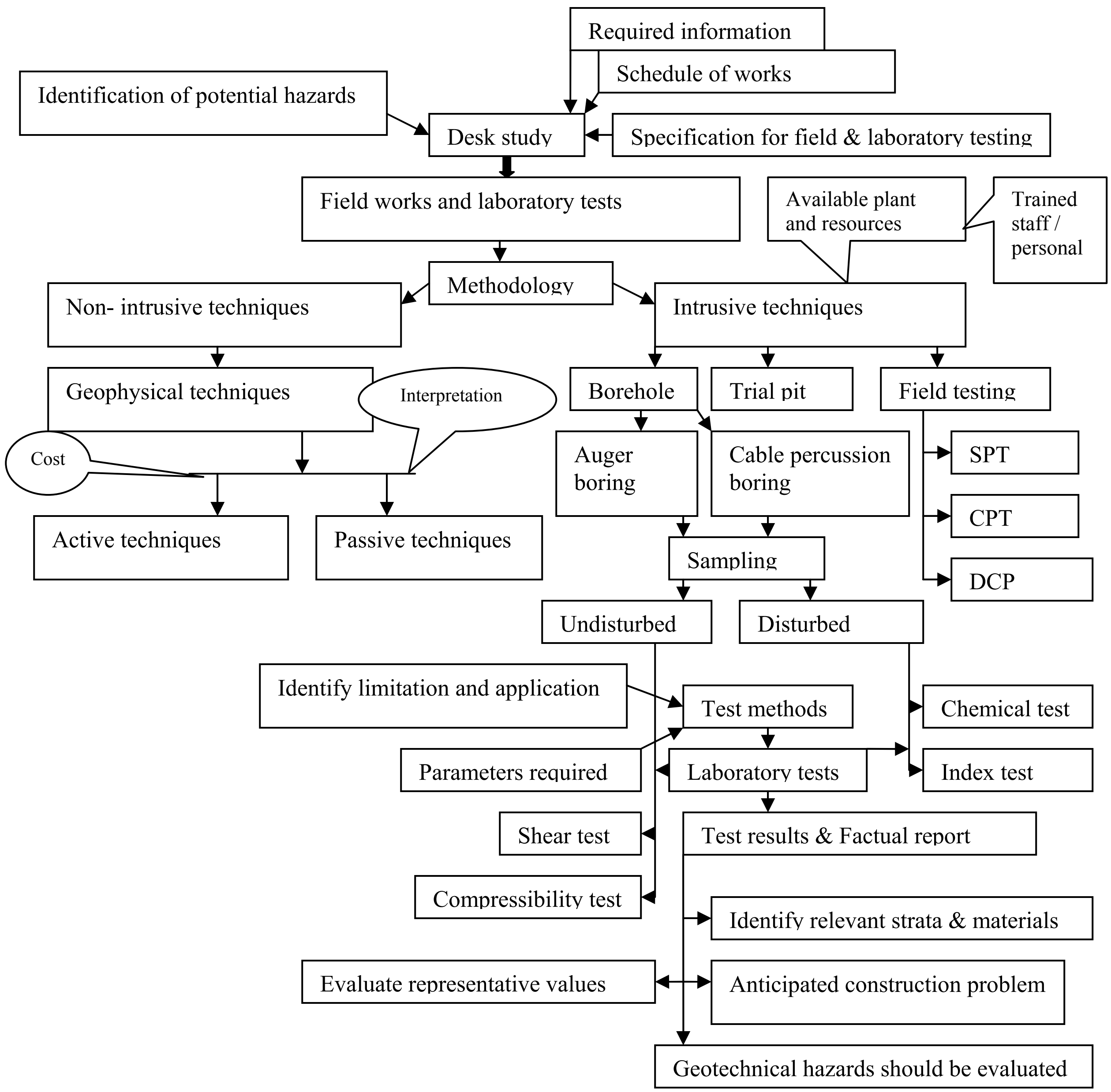
6.2.1 Phase 1: Desk Study

Desk study is important for making a programme of exploration. This phase considered the size, types & importance of the project and whether the investigation is preliminary or details (Murthy, 2003).The details are given in flow chart (Clayton et al.1995):



6.2.2 Phase II: Field works and Laboratory tests

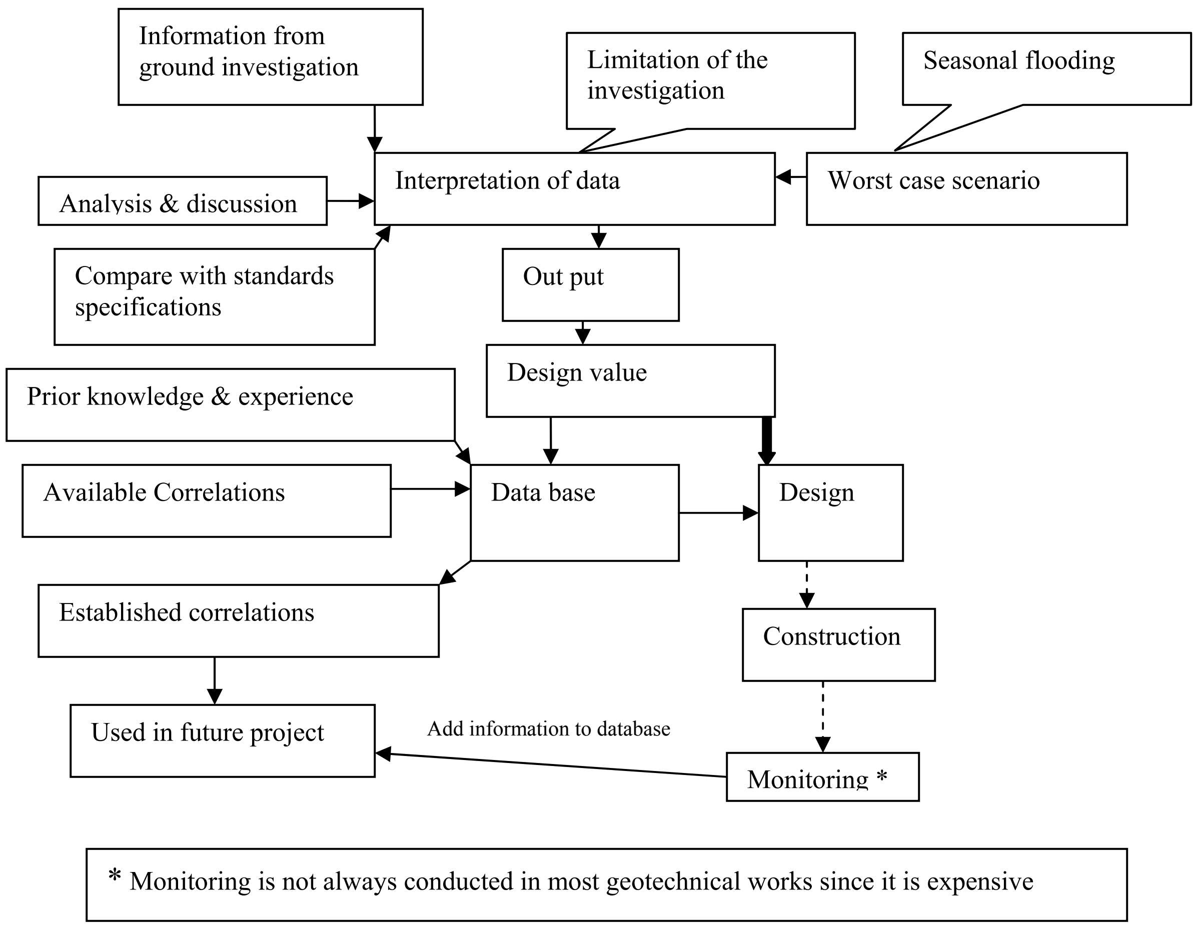
This phase is about undertaken the physical works both investigation and testing in the field together with monitoring. The activities of this phase are given in a flow chart (BS5930:1999):



Remarks: Hazards includes occurrence of water in unfavourable conditions, organic soils, presence of contamination- their concentration & extent.

6.2.3 Phase III: Interpretation

The interpretations of the results are given in the flow chart:



6.3 Borehole / trial pit layout and frequency

The frequency of boreholes and its layout depend on the types of the project and geological conditions of the site. According to Clayton, Matthews & Simons (1995), most of the project will fall into three categories such as isolated small structures, compact projects and extended projects which considered motorways as well as highways projects. The frequency of boring for road project built on embankment must be determined on the basis of the uniformity or site geology and its expected soil variability. Hvorslev (1949) recommended the boreholes spacing for highway project ranges from 30 to 60 meter to 160 meter for changeable soils and 300 meter for uniform soils (Road Research Laboratory, 1954). The depth of boring suggested by Clayton et al. (1995) is 5 to 10 meters at every 150 m frequency along the proposed road line for a motorway in the UK. They also stated that depth of trial pits and boreholes should be at least 5 meter below ground level and at least to the height of the embankment for road and embankment project respectively. For stability and settlement concern, the depth of exploration below the bottom of embankment should be penetrated all soft soils (at least equal to the height of the embankment) and should be equal to the embankment width respectively (Clayton et al.1995). As per BS593:1999, the depth of exploration should be sufficient to check possible shear failure through the foundation strata and to assess the likely amount of any settlement due to compressible strata.

Moreover, as per BS 5930:1999, the depth of exploration for roads and airfields should be sufficient to determine the strength of subgrade which may be 2 m to 3 m below the formation level. According to the RHD trail pit manual (2005), the pit size is about 750 mm x 750 mm and depth depends on the thickness of different layers.

6.4 Standard Specifications

The site investigations have been conducted by following the various code of practice (BS 5930:1999 & section 1 to 9 of BS 1377:1990). The details descriptions of explorations and insitu testing methods are used in British practice are given in BS5930: Site Investigations. Trial pits for pavement layer was done in accordance with the RHD DCP & Test Pit Survey Manual (2005). The subgrade stress – strain properties measured by resilient modulus was conducted in accordance with the AASHTO T 307. The similar test method is followed for determination of resilient modulus of granular materials of pavement layers. All bituminous materials tests were done in accordance with the ASTM Standard. Besides these, Overseas Road Note 31and RHD technical specifications (2005) were followed for pavement granular materials.

6.5 Summary

Site investigation is an important phase for successful implementation of any project. It is aimed to provide necessary information about the properties of available soils & materials, environmental and hydrological condition of the sites. Site investigation has a true distinct phase comprises of a desk study, field and laboratory testing and interpretation. This may be a single stage or a multi stage where often the desk ably same information is gathered. Based on the findings a further more in- depth investigation is undertaken. The field work includes both non-intrusive and intrusive investigation and field as well as laboratory testings. Clearly the key to the site investigation is that it should provide most, if not all the information necessary to design the required structure that fulfills it both functional and performance requirements. This is very reasonable dependent and particular attention should be provided to the information is gathered and interpreted.

Based on the discussion of this chapter, subsoil and pavement materials explorations were done on one road embankment and four roads pavement by following the above specifications described in the next chapter 7.

Chapter 7: Description of Testing Programme and Procedures

7.1 Introduction

The appropriate field and laboratory testing methods are selected based on some criteria described in chapter six of methodology. The design input parameters for the design of both embankment and pavement were determined by performing field and laboratory tests on three categories of roads built on embankment; all of which are under jurisdiction of the Roads and Highways Department (RHD). Two national highways, one regional highway and one district road were chosen for test roads. It included a section of road on an embankment which had failed in the part and same area were showing sign of distress road embankment been chosen for test embankment. The site investigations have been conducted as per required specification mentioned in chapter two of site investigation. The basis of the specifications were that all tests were conducted to the required standard in approved laboratory, pits , boreholes etc were made based on limited time available. Furthermore all testing had to be completed in the specified time.

7.2 Embankment Subsoil Testing Programme

The Tongi – Ashulia – EPZ Road (N302) embankment was selected for test road embankment because about one kilometre of road embankment has been failed. Soil samples were collected from four boreholes (BH) at failure location of embankment. The locations are at chainage 6+000 (south), 6+200 (north), 6+480 (south) and 6+600 (north) named as BH- 01, BH – 02, BH – 03 and BH – 04 respectively. The details of the boreholes locations and the position of boreholes are given in Figure 7.1 and Figure 7.2 respectively. Field work for soil exploration was design on the basis of availability of the equipment which included wash boring equipment and hand auger boring equipment BH – 01 and BH – 02 to BH -04 were made during July,2008 and December, 2008 respectively. BH – 1 was made on top of embankment and soil samples are collected from every 3 (three) meter intervals.

Bore holes number two to four (BH – 02 to 04) were made on toe of embankment where soil samples were collected at every 0.5 (half) meter intervals with hand auger upto the depth of 5.5 meters. BH- 2, 3 & 4 were located at 10 meter from toe of embankment area which is not affected by surcharge load. The collection of soil samples with hand auger is also shown is Plate 7.1.

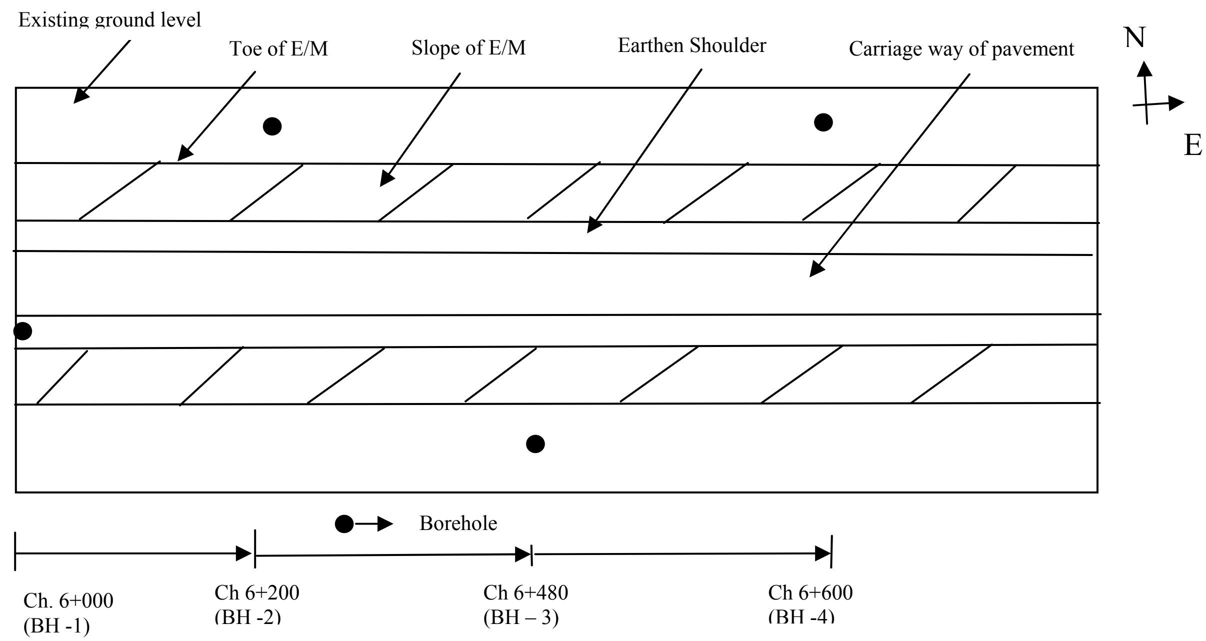


Figure 7.1: Location of borehole (BH) points in Tongi – Ashulia – EPZ road.

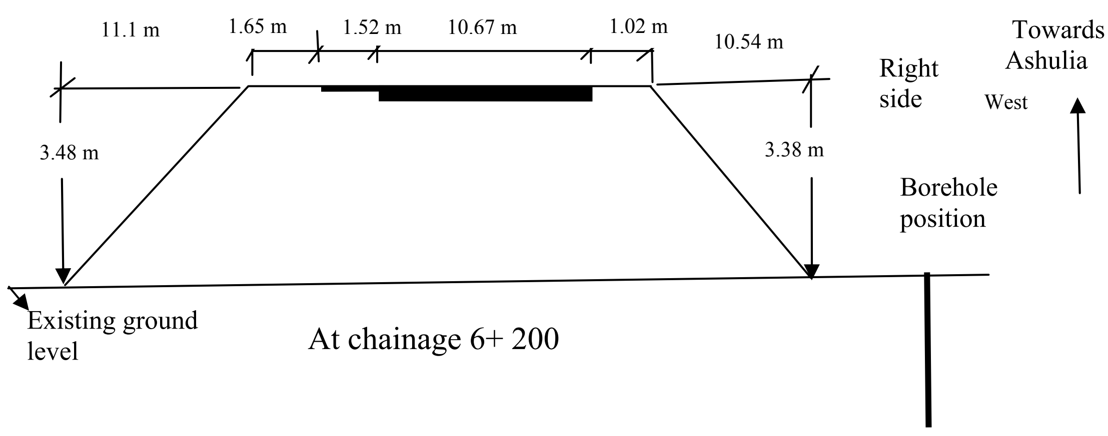


Figure 7.2: Position of bore hole at toe of embankment (x –section) of Tongi – Ashulia – EPZ Road.

Plate 7.1: Boring is in Progress by Hand Auger at Tongi – Ashulia – EPZ Road embankment.

7.3 Testing Methods for Soils

The field and laboratory tests were done on selected samples from road pavements as well as embankment soils for evaluating their properties. The former test gives the more representative parameters of materials and soils in actual field conditions. However, in some cases, field testing may not be suitable for materials and soils due to some constraints that are discussed in the previous chapter. In which case, laboratory testing was conducted on soil samples for determination of soil index properties, shear strength properties and settlement properties of soils. The numbering and labeling of samples have clearly written on the sample bag (air tight polythene) and on a label to be put inside the sample bag with permanent marker using the convention stated in Appendix - A. The details of the laboratory testing were discussed in the following subsections:

7.3.1 Atterberg limit test

The liquid limit and plastic limit test were done on collected soils samples from four boreholes. The liquid limit test can be done by two methods such as Casagrande method and Cone penetrometer method (BS 1377: part 2:1990:4.3). In this project, the liquid limit test was done as per ASTM standard ASTM D 4318 – 95a which is the Casagrande's method similar to the test method performed in RHD Standard Test Procedure (STP) – 3.2. While in plastic limit determination, the test was done as per RHD specification Standard Test Procedure- 3.2.4 which is similar to ASTM standard.

7.3.2 Particle size distribution test

Both sieving and sedimentation procedures enable for determination of particle size distribution of a soil from granular particles down to the clay size (0.002mm). Hence, combined sieve and hydrometer analysis were done for the soil samples collected from different depths and different bore holes. For coarse – grained soils, the particle size is determined by passing soil samples either by wet or dry shaken through a series of sieves of descending size. While the fraction of soil passing through 0.063mm (fine grained cohesive soils) sieve, particle size is determined by the hydrometer test which involves the measurement of the specific gravity of a soil – water suspension at fixed time intervals and the sizes are determined from the settling velocity and times recorded using Stoke's law which gives the relationship between the velocity of the spherical particle and its diameter while settling within a solution. Both of the sieve analysis and hydrometer analysis are done as per British Standard (BS 1377 – 2: 1990, section – 9).

The particle size distribution test has done on soil samples collected from bore holes 2, 3 and 4. For each bore hole samples collected from 1, 2, 3, 4 and 5 m depth were tested.

7.3.3 Organic content test

Organic content is determined as per ASTM: D 2974 – 87. The standard soil sample to be oven dried at 105^0 C for 24 hours initially. The weight loss can be used to determine moisture content. Than fifty grams of dried soil is placed in porcelain dish and burned in a muffle furnace at 440^0 C for at least 24 hours.

The organic content (OC) is determined using the following formula:

$$\%OC = \frac{w_{105} - w_{440}}{w_{105}} X100$$

Where,

OC is the organic content in percent.

W_{105} is the weight of 105^0 C dry soil samples.

W_{440} is the weight of 440^0 C dry soil samples.

The test was done on the collected 14 numbers of samples selected by visual observation of organic soils from boreholes 2, 3 and 4. Among them, 5 samples from borehole 2 and 4 each and 4 samples from borehole 3 were tested following the above mentioned procedure.

7.3.4 Unconfined compressive strength

The main purpose of the unconfined compression test is to determine the strength of soils without any confining pressure. The determination was undertaken in accordance with the British standard BS1924 Part 1: 1990. Every soil samples collected from bore hole was tested. The test set up is shown in Plate 7.2.

7.3.5 Laboratory Vane Shear test

The test was done in accordance with the clause 3 of British Standard BS1377: Part 7: 1990. The vane shear test is a method of obtaining the undrained shear strength of cohesive soils using hand held device with four vanes. The instrument comprises a torque head with a direct reading scale that is turned by hand and the non- return pointer indicates the shear strength. The test was done for all soil samples collected from borehole 2, 3 & 4 by placing the sample in a tube is shown in Plate 7.3.

7.3.6 Undrained Compressive Strength determined using a hand held Penetrometer

The undrained strength was determined by using a hand held penetrometer as shown in Plate 7.4. The penetrometer comprises of plunger attached to a calibrated spring loaded plunger. Resistance to penetration measured at fixed plunger penetration is read off the scale at side of the spring hours to give measure of strength. The reading of the penetrometer ranges from 0 to 4.5 tsf (431 kPa) that were calibrated several thousand unconfined compression tests of silty and clay like soils (penetrometer manufactured by Soiltest Inc.). To minimize errors, several readings were taken near each other and discarded those reading that may vary significantly from the majority and average reading has been taken.

This test was done on each collected samples from BH – 2, 3 and 4 for quick determination of unconfined compressive strength.

Plate 7.2: The Unconfined Compressive Strength equipment and tested soil samples during operation of the test

Plate 7.3: Laboratory Hand Vane shear test for undisturbed soil samples collected from different depths.

Plate 7.4: Use of Hand Penetrometer for measuring the undrained strength

7.3.7 Unconsolidated Undrained (UU) Triaxial Compression Test

The test was done in laboratory in accordance with the BS 1377 – 7:1990 for nine remoulded soil samples that were collected from borehole number 2, 3 and 4. The three samples from each bore hole i.e. nine specimens were tested. Four of the samples were organic and rest of the samples were inorganic in nature. The following specimens were tested and their identification numbers are as follows: BH-02/C6200/D - 0.5, BH-02/C6200/D -4.0, BH-02/C6200/D -5.0, BH-03/C6480/D - 2.5, BH-03/C6480/D - 4.0, BH-03/C6480/D -5.0, BH-04/C6600/D -1.5, BH-04/C6600/D -2.0, BH-04/C6600/D -3.0. The soil samples of 38 mm diameter and 63.5mm height were prepared by Dietert apparatus by trimming 10 blows in each end. The slenderness ratio of the samples is less than the standard test sample (height: diameter = 2:1), so the test gives slightly higher results than the standard test. The moisture content was maintained similar to the field moisture condition but for organic soil samples this could not be maintained because compaction was done by Dietert apparatus for the remoulded specimen. The test was done at a cell pressure of 25 kPa, 50 kPa and 100 kPa for each of the specimen in accordance with article 4.5.4.1.3 of chapter four. Because the samples are collected at maximum depth of 5.5 meter from the ground surface and maximum overburden pressure was found almost 47 kPa. A picture of triaxial test apparatus is shown in Plate 7.5.

7.3.8 Shear Box test

The test was performed in accordance with British Standard BS 1377 – 7: 1990 to determine the consolidated drained shear strength of soil sample. The test was done on nine remoulded soil samples; namely BH-02/C6200/D - 0.5, BH-02/C6200/D -4.0, BH-02/C6200/D -5.0, BH-03/C6480/D - 2.5, BH-03/C6480/D - 4.0, BH-03/C6480/D -5.0, BH-04/C6600/D -1.5, BH-04/C6600/D -2.0, BH-04/C6600/D -3.0. The moisture content of the remoulded specimens was maintained as field one but this for organic soils could not be controlled due to the flow of soil particles. The shear box apparatus is shown in plate 7.6. Besides this, the same test has been conducted on fine to medium sand used in subgrade layer.

Plate 7.5: A picture of triaxial test apparatus for determination of quick undrained shear strength of soils

Plate 7.6: A Picture of Shear box apparatus used in performing direct shear box test

Plate 7.7: A picture of ring shear apparatus for determination of residual shear strength of remoulded soil

7.3.9 Ring Shear test:

The ring shear test is suited to the relatively quick determination of residual shear strength of remoulded soils. This test was done in accordance with section – 6 of BS 1377 – 7: 1990 on nine soil samples of which direct shear box test were performed; to determine the residual shear strength. The moisture content of the soil samples were the same as used in direct shear box test. The similar normal load applied to the direct shear box test was applied to ring shear test for determination of corresponding residual shear stresses value. The test set up is shown in Plate 7.7.

7.3.10 Consolidation test:

This test was performed in accordance with the British Standard BS1377 – 5:1990. The test was carried out on nine remoulded soil samples namely as BH-02/C6200/D - 0.5, BH-02/C6200/D -4.0, BH-02/C6200/D -5.0, BH-03/C6480/D - 2.5, BH-03/C6480/D - 4.0, BH-03/C6480/D -5.0, BH-04/C6600/D -1.5, BH-04/C6600/D -2.0, BH-04/C6600/D - 3.0. The initial pressure is depended on type of the soil, than a sequence of pressure is applied to the specimen and each being double the previous one. The magnitude of the final incremental loading depends on the actual loading expected in the field. An experimental set up for doing the consolidation with conventional oedometer is shown in Plate 7.8. At the end of the test dry weight of the specimen was determined.

The void ratio at the end of each increment period can be calculated from the dial gauge reading. From the test result, relationship among void ratio, pressure and time were plotted and hence the consolidation characteristics were determined.

Plate 7.8: A picture of oedometer for doing the consolidation test

7.4 Selections of existing roads for evaluation

Selections of roads and test sections and reasons for selecting them are described in the following sections.

The roads and test sections were selected considering the following criteria:

1. At least one road from each categories
2. Importance of the road (considering traffic volume)
3. Road pavement built on embankment
4. Embankment of significant height that is subjected to seasonal flooding.
5. Suitable for destructive and non destructive field testing.
6. Safety of operatives and minimising disruption to traffic.
7. Proximity to the Dhaka city

Considering the above mentioned criteria the test sections were chosen. The exact locations are given in Table 7.1 and shown in a map in Figure 7.3. Each section is also described below in details.

Table 7.1: The name, location and length of test section

SL. No	Road categories	Road Name and Number	Location of test section	Length of section
1	National Highway	Joydevpur – Tangail – Jamalpur Road (N4)	Ch.2 + 450 to 3+450 GPS coordinates of start section, N23^o- 59'- 23", E90^o- 21' – 28.4"	1000 M
2	National Highway	Tongi – Ashulia – EPZ Road (N302)	Ch. 6 + 00 to 6 + 800	800 M
3	Regional Highway	Tongi – Kaligong – Ghorashal-Panchdona Road (R301)	Ch. 12 + 900 to 13 + 200 GPS coordinates of start section N23^o- 56'- 09.3", E90^o- 30' – 32.9"	300 M
4	Zilla Road	Masterbari – Mirzapur Road (Z3024)	Ch. 4 + 750 to Ch.5 + 050 GPS coordinates of start section N24^o- 05'- 01.4", E90^o-21'– 1.7"	300 M

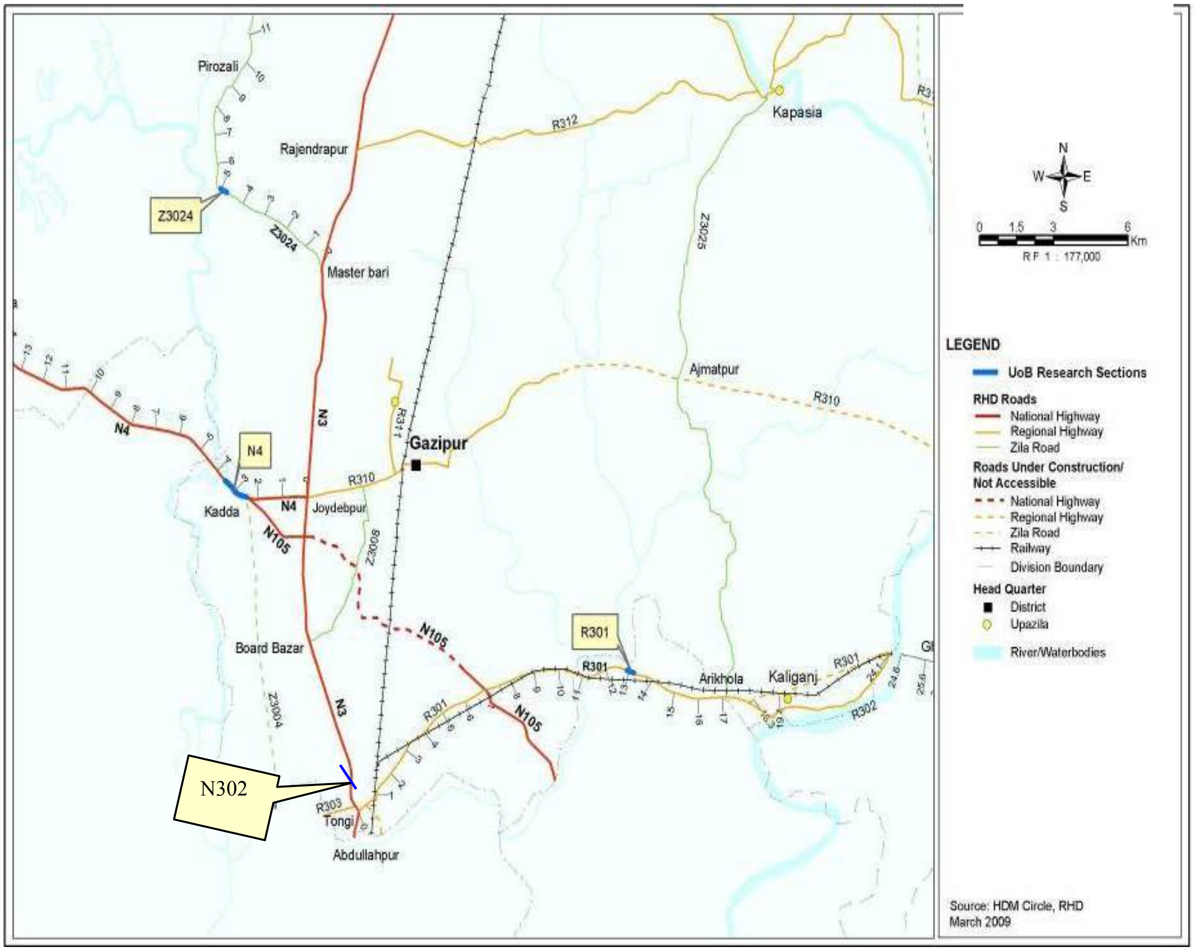


Figure 7.3: Location of the selected roads as well as test sections are shown in RHD map (GIS location map)

7.4.1 Joydevpur – Tangail – Jamalpur Road (N4):

This road is a national highway comprising 7.3m wide two-lane road with 1.2m wide hard shoulders and 0.95m wide earthen shoulders. The sections to be surveyed comprises four sub - sections of 20m in length and one sub - section of 200m long all within a 1,000 meter length, as shown in Figure 7.4. In each 20 meter subsection, two Benkelman beam in each lane (one in a wheel path and another in between wheel path), one trial pit and one core along with DCP test adjacent to the trial pits were done.

For the 200 meter subsection, two Benkelman beam in each lane, trial pits at 100 meter intervals staggered on either side of the road with a core from bituminous layers and coring with DCP testing through the core holes at 20 meter centre's between the trial pits were done and shown in Figure 7.5.

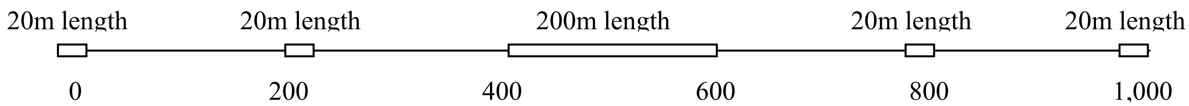


Figure 7.4: The length of test sections divided into different subsections for details investigations.

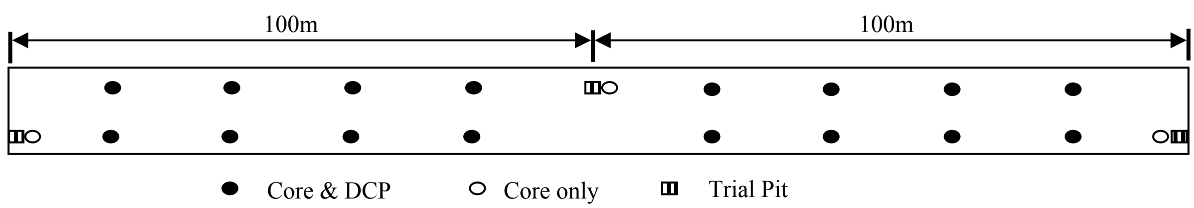


Figure 7.5 : The details of test testing and 200 m section for N4 as well as R301.

7.4.2 Tongi – Ashulia – EPZ (N302) Road:

This is a national highway comprising 10.67 m carriage way, 1.52 m hard shoulder and 2.67 m earthen shoulder. The road is passing through marshy land and its height of embankment is almost 3.5 m. Only coring at bituminous layer and DCP of that core locations have been done on that road. The locations of coring as well as DCP test points are shown in Figure 7.6.

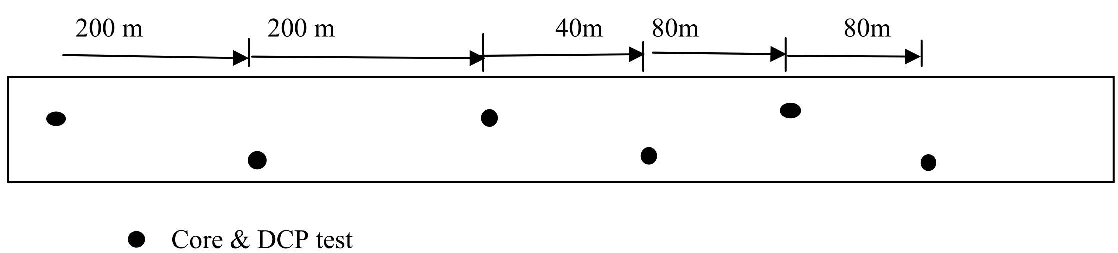


Figure 7.6: The test point location of Tongi – Ashulia – EPZ Road.

7.4.3 Tongi – Kaligong –Ghorashal- Panchdona Road (R301)

This road is a regional highway comprising 5.5m wide with earthen shoulders. The test section to be surveyed comprises one 200m long sub - section as already shown in Figure 7.5. For the 200 metre sub - section, three Benkelman beam in each lane, trial pits at 100 metre intervals staggered on either side of the road with a core from bituminous layers and coring with DCP testing through the core holes at 20 meter centre's between the trial pits have been done.

7.4.4 Master Bari – Mirzapur – Pirujali – Nuhashpalli – Mawna Road (Z3024)

This road is a single lane rural road around 3.8 m wide with 3m earth shoulders which is known as Zilla road as per RHD classifications. The test section to be surveyed comprises one section 200m long as already shown in Figure 7.5. For this 200 meter section, two Benkelman beam test, trial pits at 100 meter intervals with a core from bituminous layers and coring with DCP testing through the core holes at 20 meter centers between the trial pits were done and shown in Figure 7.7.

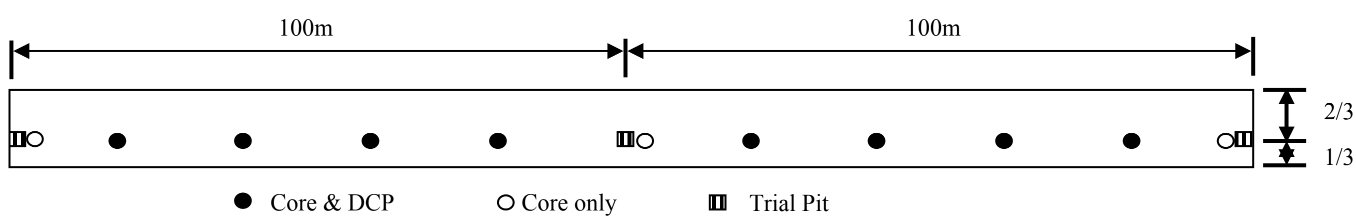


Figure 7.7: The details of the test sections of 200 m long for Zilla road (Z3024) .

7.5 Pavement Testing Programme

Tests have been done on the above mentioned three categories of roads. The field tests were carried out on the above mentioned roads. Soils and road materials were tested at Bangladesh University of Engineering & Technology (BUET) and Dhaka University of Engineering & Technology (DUET). The details of the field testing programme are described in the following subsections:

7.5.1 Location of the Trial Pit Points & Pit log

The location of the trial pit points of three roads of Joydevpur – Tangail – Jamalpur (N4) road, Tong – Kaliganj – Ghorashal – Panchdona (R301) road and Master Bari – Mirzapur – Pirujali – Nuhaspalli – Mawna (Z3024) road are given in the following Table 7.2.

The pits are excavated manually in accordance with the RHD's trial pit survey manual (2005). The size of the pit was 0.75 m x 0.75 m and depth was ranged from 0.6 m to 1.32

m. The excavation was started from top of the bituminous surface and dug downwards at least 300 mm into subgrade layer. A typical layout of trial pit is shown in Figure 7.8.

Table 7.2: Location of the trial pit points

Road Name & Number	Pit point reference No	Trial Pit No	Trial pit location			Final depth of trial pits (m)
			Chainage (km)	Distance from road edge (m)	Lane (left / right)	
Joydevpur – Tangail – Jamalpur (N4) Road	N4/2+500/L	TP1	2+500	0.00	L	1.035
	N4/2+700/R	TP2	2+700	0.00	R	1.15
	N4/2+900/L	TP3	2+900	0.00	L	1.30
	N4/3+000/R	TP4	3+000	0.00	R	1.18
	N4/3+100/L	TP5	3+100	0.00	L	1.18
	N4/3+300/R	TP6	3+300	0.00	R	1.32
	N4/3+500/L	TP7	3+500	0.00	L	1.02
Tong – Kaliganj – Ghorashal – Panchdona (R301) Road	R301/12+955/R	TP1	12+955	0.00	R	0.75
	R301/13+050/L	TP2	13+050	0.00	L	0.62
	R301/13+150/R	TP3	13+150	0.00	R	0.62
Master Bari – Mirzapur – Pirujali – Mawna - (Z3024)Road	Z3024/4+820/R	TP1	4+820	0.50	R	0.60
	Z3024/4+900/R	TP2	4+900	0.40	R	0.60
	Z3024/5+000/L	TP3	5+000	0.40	L	0.60

During the start of excavation of pavement from top layer, bituminous layer were removed carefully to avoid disturbance of the layer below and thicknesses is recorded. If there was no easy separation of layers then the sample was taken for the full thickness of the bituminous layers and sent to the laboratory for separation. After removal of the bituminous layers a DCP test have been done in one corner of the pit, penetrating to at least 300mm into the sub-grade. The field density tests were carried out by sand replacement method in an undisturbed area of at least 150mm away from the DCP hole after complete removal of each granular layer as well as sub-grade layer. The location of the trial pit, trail pit log and field testing points for three roads are discussed below:

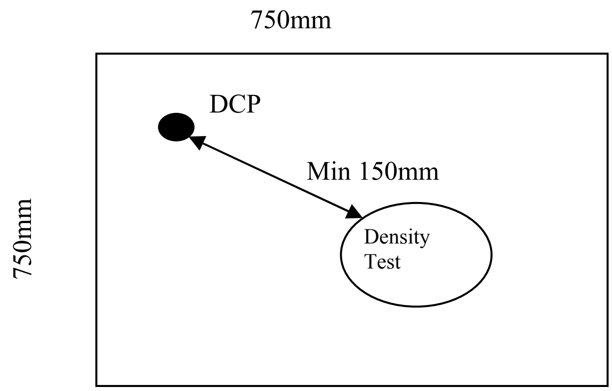


Figure 7.8: Layout of Trail Pit

(i). Joydevpur – Tangail – Jamalpur (N4) Road : Seven trial pits were dug manually at the location noted in Table 7.2. The descriptions of the strata encountered are given in trial pit log sheets are shown in Appendix Table 7.1 to 7.2. From all of the trial pits it was observed that, the thickness of the bituminous layer is about 130 mm, followed by aggregate base type I and II and their thickness vary from 120 to 190 mm and 130 to 170 mm respectively. The thickness of the subbase and improved subgrade layer vary from 215 to 270 mm and 290 to 590 mm respectively. The different materials encountered will be described on the log sheet in accordance with the guidance notes for material classification.

ii. Tong – Kaliganj – Ghorashal – Panchdona (R301) Road: In this road, three trial pits were dug manually at 100 meter interval. Locations of the pits are already given in Table 7.2. Descriptions of the materials encountered are given in trial pit log sheet shown in Appendix Table 7.3. Materials used in this road are different from those used in national highway (N4). The thickness of the bituminous layer was found 55 mm, followed by road base made of water bound macadam with brick aggregates. The thickness of the base layer ranged from 80 mm to 120 mm. Besides this, a non conventional layer was found named scarified and recompacted existing old bituminous layer followed by subbase and improved subgrade layer. The materials found at subbase and improved subgrade layer were brick aggregate & sand mixture and medium sand respectively.

iii.Master Bari – Mirzapur – Pirujali – Nuhaspalli – Mawna (Z3024) Road: Three trial pits were made on this road at the location already shown in Table 7.2. Initially it was proposed to construct the trial pit at 100 meter intervals from chainage 4+800. But during coring, bituminous core broke up when lifting. For this reason, this point has been shifted 20 m ahead at new chainage 4+820 where coring is done successfully. The description of strata encountered is given in Appendix Table 7.4. As it is a Zilla road, the thickness of the bituminous layer is found much thinner than regional & national highway. The thickness of that layer ranged from 20 to 25 mm, followed by road base made of water bound macadam (WBM) with brick aggregate. After that layer, brick flat soling was found and in some chainages two layers of brick flat soling was observed.

7.5.2 Sampling from Trial Pits

Samples were collected from trial pit points of each road for details laboratory testing. The samplings were started from top of the pavement layers towards downward upto the subgrade layer. The pits were drug manually. Plate 7.9 shows the excavated trial pits at R301 road and Plate 7.10 shows an old bituminous layer that was found below the improved subgrade layer of N4 road (chainage 3 + 500) which means new pavement was constructed on the old pavement without scarifying that old pavement layer. For identification of samples collected from sites are properly numbered and labelled. The procedure for numbering and labeling of samples are given in Appendix- B.

The amount of granular materials for laboratory tests will be collected in accordance with the requirements given in the RHD Standard Test Procedures (May 2001) and RHD DCP & Test Pit Survey Manual (2005). The amount of materials collected from bituminous layer, granular layer, improved subgrade and subgrade layers are about 15 kg, 10 – 45 kg, 2 kg and 25 kg respectively. Depending on the numbers of test, granular material may vary from 10 to 45 kg.

7.5.3 Reinstatement of the Trial Pits and Coring Points

After collection of samples from each layer of pavement, the trial pits were backfilled by the aggregates stabilised with 5% cement. Materials were mixed thoroughly with water prior to placement in the hole in layers. Each layer was hand compacted with rammers as shown in Plate 7.11. The bituminous layer was then reinstated to their full depth using a mini roller (four tonne) is also shown in Plate 7.12. Before placing the bituminous materials in holes, all

Plate 7.9: Digging of trail pits manually

Plate 7.10: Trail Pit at N4 road (Old bituminous layer below ISG)

Plate 7.11: Reinstatement of granular layer of trial pits using aggregate + 5% cement.

Plate 7.12: Reinstatement of bituminous layer is in progress by bituminous materials in pit at N4 road.

faces of the holes have been painted with bituminous tack coat. The finished surface were sealed using a 7mm seal coat that extends at least 150mm beyond the limits of the excavation. Coring points was backfilled by bituminous materials with proper compaction. Before placing the materials in holes, bituminous tack coat was applied on all faces of the holes.

7.5.4 Core Cutting

Specimens were covered using core cutter machine. Bituminous core were cut to ascertain the actual thicknesses of the layers as well as to provide access to lower granular layers for doing the DCP test. Cores were of 100 mm diameter.

Thirty cores were cut from the Joydevpur – Tangail – Jamalpur Road from chainage 2+450 to chainage 3+550 that are shown in Plates 7.13 (a) and 7.13 (b). Among them seven were done in trail pit locations in both sides and rest of them were done outside the trial pit locations which is 20 meter interval (both sides) from Ch 2+900 to Ch 3+100. Cores were collected from that locations and kept in a wooden boxes shown in Plates 7.14 (a) & 7.14 (b). Besides this, seven cores were cut during July/2008 from Tongi – Ashulia – EPZ (N302) Road at chainage 6+000, Ch 6+200, Ch 6+400, Ch 6+440, Ch 6+ 520, Ch 6+ 600 and Ch 6+800. The cores from N302 road were cut only for determination of the thickness of the bituminous layer.

In Regional highway like Tongi – Kaligonj – Ghorashal – Panchdona (R301) Road, twenty two cores were cut from chainage 12+900 to chainage 13+200. Among them three in the trial pit locations and rest of them are 20 meter interval (both sides) between chainage 12+950 to 13+150.

Eleven cores were cut from bituminous layer of the pavement from chainage 4+800 to chainage 5+000 at an interval of 20 meters as only one side of the Zilla road, Master Bari – Mirzapur – Pirujali – Nuhashpalli – Mawna (Z 3024). As it is single lane road and the pavement width is only 3.66 meter, cores were cut from alternate sides of the pavement.

Plate 7.13 (a): Coring of Bituminous layer has in progress at N4 road

Plate 7.13 (b): Coring of Bituminous layer has in progress at N4 road

Plate 7.14 (a): Some coring samples of bituminous layer that will be sent to testing laboratory.

Plate 7.14(b): Some coring samples of bituminous layer that will be sent to testing laboratory

7.5.5 Testing Programme for Dynamic Cone Penetrometer (DCP)Test:

The dynamic cone penetrometer test was done on the Joydevpur – Tangail – Jamalpur Road from chainage 2+450 to chainage 3+550 at all trial pit locations as well as coring point locations. Thirty DCP tests were conducted of which seven were at trial pit locations and rest of them were in the core locations. At core locations, test was conducted after removal of the bituminous core from that point which is shown in Plate 7.15. DCP test at trial pit location is shown in Plate 7.16. Again DCP test were conducted of coring point of Tongi – Ashulia – EPZ Road during July, 2008.

The same test was done on the Tongi – Kaligonj – Ghorashal – Panchdona (R301) Road and Master Bari – Mirzapur – Pirujali – Nuhashpalli – Mawna (Z 3024) at trial pit locations and coring point locations of the above mentioned chainages.

DCP test was done at toe and top (along earthen shoulder) of embankment at different chainages to estimate the CBR value of those soils. DCP test is shown in Plate 7.17 (a) and 7.17(b). Besides this, soil samples were collected from those DCP test positions with new innovative soil sampler described in Appendix – C for determination of moisture content of the soil samples. The picture of the DCP sampler is shown in Appendix Figure 7.1. The moisture susceptibility of the top and toe soils were the properties of soils used to estimate.

7.5.6 Field Testing Programme for In Situ Density Test

The test was done in accordance with the BS 1377-9:1990. It was carried out on base, subbase, ISG and subgrade layer after excavation of bituminous layer of the pavement.

Insitu density of upper layers in all three roads was done at trail pit locations and 150 mm away from DCP test point as shown in Figure 7.8. Seven insitu densities by sand replacement method were done at Joydevpur – Tangail – Jamalpur Road in all pavement layers as well as subgrade layer except bituminous layer. The same test was done on Tongi – Kaligonj – Ghorashal – Panchdona (R301) Road and Master Bari – Mirzapur – Pirujali – Nuhashpalli – Mawna (Z 3024) Roads in all layers as well as subgrade layer except bituminous layers at a quantity of three numbers each. The Plate 7.18 and Plate 7.19 show the testing process of in situ density by sand replacement method for base layer and subgrade layer respectively.

Plate 7.15: DCP test at core location N4 road.

Plate 7.16: DCP test at pit location from top of granular base layer of N4 road.

Plate 7.17 (a): DCP test done at toe of Embankment of Tongi – Ashulia – EPZ Road

Plate 7.17 (b): DCP test done at toe of Embankment of Tongi – Ashulia – EPZ Road

Plate 7.18: In situ density by sand replacement method of base layer of pavement at R301

Plate 7.19: Insitu density by sand replacement method for subgrade soils of Road No. Z3024

7.5.7 Testing Programme for Benkleman Beam Test

The test was conducted in accordance with the AASHTO test T – 256 to measure the surface deflection of the flexible pavement under moving wheel loads. It is the non-destructive test of the pavement. Pavement surface deflection is an important pavement evaluation method and measurement was done as the vertical deflected distance of surface due to the application of the load.

Fourty five Benkleman beam tests were conducted on Joydevpur – Tangail – Jamalpur Road (N4) in fifteen location from chainage of 2+500 to Ch 3+500. In each location, three tests were conducted in which two and one number in left and right carriage way in a staggered manner. At each trail pit points two Benkleman Beam tests were conducted. However, 200 meters sections from chainage 2+900 to chainage 3+100, was scrutinised in which this test was done at an interval of 20 meters. In rest portion of the above mentioned test sections, the test was done at an interval of 200 meters.

This test was done along the Tongi – Ashulia – EPZ (N302) Road. The locations of this test are at Ch 6+000, Ch 6+200, Ch 6+400, Ch 6+440, Ch 6+520, Ch 6+600, and Ch 6+800 in which one test was done in each location at alternate carriage way.

Twenty two tests were done along Tongi – Kaligonj – Ghorashal – Panchdona (R301) Road at the location of Ch 12+950 to Ch 13+150 at an interval of 20 meters. In each location, test have been done in both carriage way.

However, this test was also done in single lane carriage way Zilla road named Master Bari – Mirzapur – Pirujali – Nuhashpalli – Mawna (Z 3024). Eleven tests were conducted between Ch 4+800 to Ch 5 + 000. The frequency of the test is 20 meters interval.

7.6 Laboratory Testing Programme for Pavement Materials

7.6.1 Cyclic Triaxial Test

The test was done for determination of the resilient modulus of subgrade soils and granular materials in accordance with the AASHTO T307 procedure. The test procedure is based on the Strategic Highway Research Programme Protocol P46 in which repeated load triaxial testing is specified for determine the resilient modulus (Elias & Titi, 2006). The test was done first time for Bangladesh subgrade soils as well as sand which is used as improved subgrade layer (ISG) of pavement. The test was done on 100mm diameter soil samples with height 200 mm and compacted to BS standard compaction (2.5 kg Proctor) which represents the Bangladesh condition. The granular materials passing through 20 mm sieve were used for preparation of samples. A 100mm diameter and 200 mm height sample was prepared using vibrating rammer. The moisture content of the materials was 11.75%. The sample was then soaked at four days. After four days soaking, the samples were allowed one day for drainage of water from the samples to avoid undesirable pore pressure. There are different test conditions are available in AASHTO T307 for subgrade soils, base & subbase materials and user defined conditions. The test was done as per recommended test conditions for subgrade soils specified by AASHTO T307. Different confining, deviator and cyclic stresses are applied ranging from 41.4 kPa to 13.8 kpa, 27.6 kPa to 68.9 kPa and 12.4 kPa to 62.0 kPa respectively. The samples were conditioned by application of 500 cyclic stress level of 24.8 kPa together with a confining stress level of 41.4 kPa. This conditioning step removes irregularities on the top and bottom surfaces of the test specimen. The initial stage of permanent deformation of the test specimen takes place under the conditioning stress (Mohammad, Herath, Rasoulian & Zhongjie, 2006). A cyclic load applied with a frequency of 1 Hz (stage: 0.1 s loading period and 0.9 s relaxation period). The test was conducted using three confining pressure (41.4, 27.6 & 13.8 kPa) and five deviator stress level (12.4, 24.8, 37.3, 49.8 & 62.8 kPa) along with five cyclic stress level ((1.4, 2.8, 4.1, 5.5 & 6.9 kPa) for each confining pressure. For each combination of confining and cyclic stress, 100 cycles of loads were applied and the load and displacement were recorded for the last five cycles. It is to be mentioned here that the confining pressure to the subgrade soil samples in the triaxial cell is given by air. Deflections were measured with two axial linear variable differential transducers (LVDT). There are fifteen numbers of sequence were prescribed to perform this test where if the permanent strain is reached 5% in any stage of loading, the test will automatically stopped and it will give the resilient modulus upto the previous cycle of loading. The testing apparatus is shown in plate 7.20.

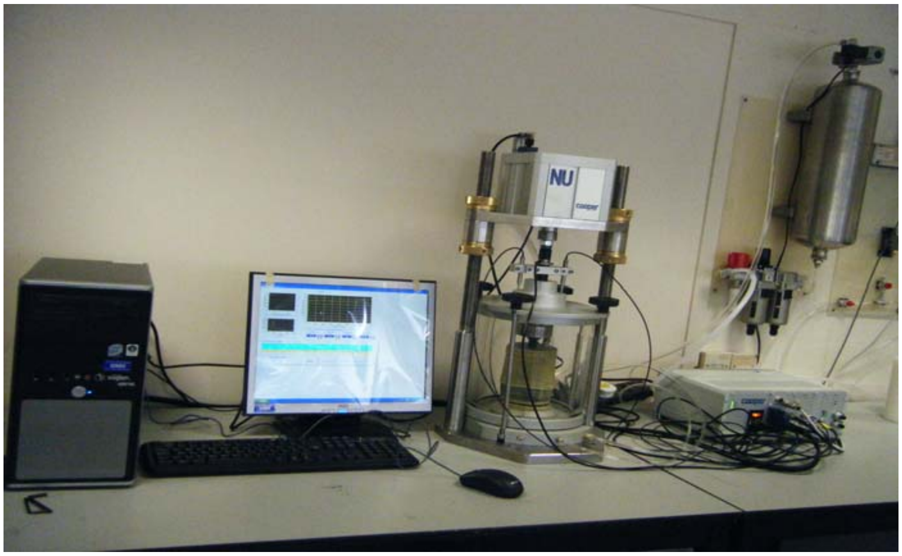


Plate 7.20: Cyclic triaxial test is going on Subgrade soil sample

7.6.2 California Bearing Ratio (CBR) Test.

This test was conducted on subgrade soils and granular materials of pavement. The test is appropriate for the materials that passing through 20 mm sieve. The test was carried out in accordance with the section 4.5 of BS 1924-2: 1990. As a flood prone country considering the worst condition of the subgrade and other pavement layers, Soak CBR (4 days) are usually determined and this result is used in pavement design of Bangladesh. Four days soak CBR is recommended as per ASTM specification section -7, D 1883 – 05, for this reason four days soak CBR is done in Bangladesh.

The samples of cohesionless soils are compacted at moisture contents equal to or greater than the optimum moisture content, they should be left sealed for 24 hours before being tested so that excess pore water pressures induced during compaction are dissipated (recommended by the RN 31).

The soaked CBR test was carried out on base material sample numbers N4/2+700/R/G/RB1(upper base), 4/2+700/R/G/RB2 (lower base), N4/3+100/L/G/RB1 (upper base) and N4/3+100/L/G/RB2 from N4 road and R301/12+955/R/G/SG (upper), R301/12+955/R/G/SG (lower) and R301/13+050/L/G/SG from R301 road and Z3024/4+820/R/G/SG, Z3024/5+000/L/G/SG from Z3024 road. On the other hand, the same test was carried out in the subbase material sample numbers N4/2+700/R/G/ISG and N4/3+100/L/G/SB of N4 road, R301/12+955/R/G/SG and R301/13+050/L/G/SG of R301 road.

The soaked CBR test of improved subgrade layer (sand blanket layer) was carried out on the following samples: N4/2+700/R/G/ISG and N4/3+100/L/G/SG of N4 road; R301/13+050/L/G/SG of R301 road.

The soaked CBR test on subgrade soils was carried out on the following soil samples: N4/2+700/R/G/ISG & N4/3+100/L/G/SG of N4 road, R301/12+955/R/G/SG and R301/13+050/L/G/SG of R301 road, Z3024/4+820/R/G/SG and Z3024/5+000/L/G/SG of Z3024 road.

7.6.3 Aggregates Strength Test:

The mechanical strengths of aggregates were measured by aggregates crushing value (ACV) and Ten percent fines value (TFV). The ACV and TFV tests were done in accordance with the BS 812- 110:1990 and BS 812 – 111:1990. The materials used in the above mentioned tests were passing through 14 mm and retained on 10 mm sieve. These tests were done for aggregates of bituminous materials, granular base and subbase materials collected from trial pit – 2 & 3 of N4 road. The same tests were carried out on base and subbase materials collected from trail pit – 1 & 2 of R301 and trail pit 1 &3 of Z3024 roads.

7.6.4 Shape Tests for Aggregates:

Both elongation index and flakiness index were done on coarse aggregates used in pavement layers. These tests are important because when elongated and flaky materials are used in the construction of pavement, may cause the pavement failure due to the preferred orientation that the aggregates take under repeated loading as well as vibration (British standard, 1989). For this reason, these tests are important to select the materials used in pavement layers. The elongation index test was done as per BS 812 – 105.2:1990 and the sieves used in this test are 50mm to 6.3mm. On the other hand, the flakiness index test was done as per BS 812 – 105.1:1989 and the test sieves were used in this test was 63mm to 6.3 mm. These tests were carried out on aggregates of bituminous materials collected from trail pit – 2 & 3 of N4 road and trial pit -2 of R301 road.

7.6.5 Particle Size Distribution for Soils & Granular Materials

The test was conducted BS 1377 – 1:1990. The test gives the distribution of particle sizes in the aggregate mass. Seven tests were conducted on base; subbase and ISG layer materials of N4 road. The same test was conducted on base; subbase and ISG layer materials of R301 & Z3024 at quantity of three tests for each. The test was also done on recovered aggregates collected from trail pits – 2 &3 of N4 road and trail pit – 2 of R301 road. In addition to these the same test was done on subgrade soils of three roads.

7.6.6 Marshall Stability and flow:

The test was conducted in accordance with the ASTM D 1559. Bituminous samples of 100 mm diameter were collected from the field by coring. The height of the sample should be in between 25mm to 75mm for applying the calibration chart for proving ring factor. As the height of the specimen has influence on the stability, so a correction factor for stability was applied for the height more or less than 63.5 mm. The stability is expressed in kN. The flow value is the reading on the deformation gauge at the point of maximum loading is expressed in mm.

The test was done on bituminous cores collected from seven trail pit locations of N4 road, three trial pit locations of R301 road and three trial pit locations of Z3024 road.

7.6.7 Bitumen extraction test:

This test was done in accordance with the ASTM D2172 – 05. Four samples collected from trial pit locations of N4, one samples of R301 and one samples of Z3024 roads were tested for extraction of bitumen.

7.6.8 Bitumen Properties tests:

The bitumen properties test such as penetration, viscosity, ductility, loss on heating and solubility have been conducted on recovered bitumen extracted from the samples of N4/2+700/R/B/WC&BC, N4/3+100/L/B/WC&BC from N4 road and R301/13+050/L/G /SG from R301 road. These tests are done as per ASTM standard in the testing laboratory of Bangladesh University of Engineering & Technology, Dhaka following the ASTM D 5, ASTM D 2170, ASTM D113, ASTM D 36-06, ASTM D6-95 (2006) & ASTM D 2042 for Penetration test, Viscosity test, Ductility, Softening point, Loss on heating & Solubility test respectively.

7.7 Conclusion

The details of the field testing programme for both embankment and pavement were described in this chapter. It is mentioned that only limited number of boreholes were made on one embankment test section due to the time constraint and lack of resources. The laboratory tests were conducted on limited number of soils and granular materials based on the standard noted in this chapter. So, on the basis of the testing programme, both field and laboratory tests were conducted and the results, analysis and discussions are made in the following chapter eight, nine & ten.

Chapter 8: Test Results and Discussions for Embankment Soils

8.1 General

This chapter contains the results and analysis of field and laboratory testing for embankment soils. All necessary field investigation like boring, collection of soil samples from different depths, strata encountered and laboratory testing of collected samples were done on road embankment of Tongi – Ashulia - EPZ (N302) that was selected as a test case for evaluation of design. This investigation provided input parameters for the test case study which included assessment of existing design and proposal to improvement in design. The test case design study work was undertaken by Bhattacharyya (2009), design of embankment for Bangladesh, based on the test results of this study. The details of the test results for embankment soils are discussed in the following subsections.

8.2 Borehole and Soil Strata

Borehole of BH- 01 and BH – 02 to 04 have been made on top and toe of embankment by using wash boring and hand auger boring method in different time periods respectively. From BH – 01, both filling and foundation soils properties were evaluated whereas information from BH – 02 to 04 was used to assess engineering properties of the foundation soils of the embankment. The positions of the boreholes for the section of the Tongi – Ashulia – EPZ road were already shown in figure 7.1 of chapter seven.

The strata encountered for different boreholes at different depths were shown in Appendix G, Table A- 8.1. Due to the inconsistency of soil samples collection method (wash boring), BH-01 was not included in that table. The disadvantages of the wash boring is that the natural moisture content may be changed and some fine particles of soil samples may be washed out with water during boring was progress. As a result, particle size distribution of soil samples may not truly reflect the soils. But from the soil strata of BH – 01 observed that organic clay upto 3 m depth followed by grey clay at a depth of 6 m and below that, soil was found brownish clay upto depth of 12m. Meanwhile soils strata from BH – 02 to 04, observed that strata contains brownish clay, gray clay to black organic or pit soils.

8.3 Classifications of Soil - Index properties

All soil sample collected from various boreholes were tested as discussed in Chapter seven to ascertain their engineering characteristics. Soil samples are subjected to many routine tests like Atterberg limit test, natural moisture content determination, particle size distribution test, organic content test and specific gravity test.

Atterberg limit determinations of specimens from borehole number, BH- 01 to 04 are presented in Table 8.1. Besides these, moisture content, unit weight, specific gravity of the soil samples are also presented in this Table.

The Liquid Limit and Plasticity Index of the soil samples are plotted on Casagrande's plasticity chart shown in Figure 8.1; results confirmed that soil samples are silts of low to intermediate plasticity. Results (Figure 8.1) also show that organic soils have lower plasticity index compared to the inorganic ones. Based on these results along with index properties, soils are classified by unified soils classification system (USCS) are reported in Appendix G, Table 8.1.

Liquid Limit and Plasticity Index values of the soil samples ranged from 29% to 68% and 5% to 25% respectively. These soils can be probably classified as low to medium plasticity soils. Again, these low to medium plasticity indicates that the soils of test road embankment have low shear strength.

The strength of soil depends on the moisture content and its mineralogy. A term liquidity index[14] (I_L) can be used to relate strength to both natural moisture content and index properties. Hence, plots of liquidity index vs undrained shear strength for both inorganic and organic soils are shown in Figure 8.2 (a) and 8.2 (b) respectively. Results of specimens from BH -01 were not considered because moisture content of the soil samples were not truly reflected the actual field condition as these were collected by the wash boring method. Results show that when natural moisture content is equal to Liquid Limit, the undrained shear strength value was almost 20 kPa. This is slightly higher value found by Skempton and Northey's (1952) as shown in Figure 4.2 of chapter Four. Meanwhile, same relationship for organic soil is different as it shows almost flat straight line relationship. Not enough data is available at low I_L values to make a prove comparison with Skempton's results. Since bulk of his results are for this range I_L = 0 at

[14] Liquidity index = $(w - w_p)/(w_l - w_p)$

Plastic Limit and I_L = 1 at Liquid Limit. So the comparison for organic soils is not suited valid due to the insufficient data. On the other hand, it is seen that liquidity index for organic soils is much higher than the inorganic one because organic soil contains higher amount of natural moisture content due to the presence of organic matter in the soil minerals.

Table 8.1: Index Properties of soils collected from different depth of BH – 1 to 4

BH No	Depth from E.G.L	Moisture content (%)	Liquid limit (w_L), %	Plastic limit (w_P),%	Plastici-ty Index (I_P),%	Unit weight, Y (kN/ m^3)	Dry unit weight, Yd (kN/ m^3)	Liquidi-ty Index, I_L	Specific gravity, Gs	Initial void ratio, e_o	Organic content (%)
1	3	63.72	51.95	31.02	20.93	12.2	7.45	1.56	2.56	2.37	-
	6	37.59	35.25	21.69	13.56	17.23	12.52	1.17	2.64	1.07	-
	9	36.65	44.43	30.58	13.85	17.96	13.14	0.44	2.68	1.00	-
2	0.5	28.24	46.00	25.00	21.00	19.97	15.57	0.15	2.59	0.63	7.54
	1	29.01	38.00	25.00	13.00	18.37	14.24	0.31	2.61	0.80	-
	1.5	29.87	39.50	24.30	15.20	19.90	15.32	0.37	2.58	0.65	-
	2	33.86	50.00	29.20	20.80	17.44	13.03	0.22	2.64	0.99	-
	2.5	90.74	57.00	36.00	21.00	13.60	7.13	2.61	2.42	2.33	15.21
	3	49.82	42.00	24.00	18.00	17.40	11.61	1.43	2.55	1.15	-
	3.5	143.85	54.00	38.00	16.00	13.21	5.42	6.62	2.35	3.26	18.04
	4	232.39	33.00	21.00	12.00	11.61	3.49	17.62	2.20	5.18	28.80
	4.5	37.32	56.00	35.00	21.00	15.92	11.59	0.11	2.57	1.17	-
	5	30.39	42.60	19.20	23.40	19.57	15.01	0.48	2.64	0.73	3.89
	5.5	26.51	40.00	20.00	20.00	19.54	15.45	0.33	2.63	0.67	-
3	0.5	40.26	29.10	23.67	5.43	18.39	13.11	3.06	2.58	0.93	-
	1	64.43	57.00	37.17	19.83	15.74	9.57	1.37	2.62	1.69	-
	1.5	35.45	47.00	26.04	20.96	17.92	13.23	0.45	2.70	1.00	-
	2	51.96	49.40	28.97	20.43	16.77	11.04	1.13	2.60	1.31	-
	2.5	60.09	58.50	37.21	21.29	15.77	9.85	1.07	2.18	1.17	14.11
	3	42.43	49.00	30.18	18.82	17.36	12.19	0.65	2.64	1.12	-
	3.5	39.86	46.00	28.52	17.48	18.13	12.96	0.65	2.64	1.00	-
	4	54.82	43.00	26.02	16.98	17.00	10.98	1.70	2.55	1.28	4.19
	4.5	181.42	51.40	30.80	20.60	12.44	4.42	7.31	2.38	4.28	23.39
	5	229.34	58.90	38.60	20.30	12.45	3.78	9.40	2.21	4.74	28.72
	5.5	160.36	51.00	34.87	16.13	12.10	4.65	7.78	2.31	3.88	-
4	0.5	43.84	38.60	16.80	21.80	13.54	9.41	1.24	2.58	1.69	-
	1	38.42	39.00	16.76	22.24	19.45	14.05	0.97	2.61	0.82	-
	1.5	33.26	40.80	20.20	20.60	17.54	13.16	0.63	2.70	1.01	2.27
	2	59.50	48.60	27.33	21.27	16.44	10.31	1.51	2.60	1.47	4.24
	2.5	81.32	58.00	33.57	24.43	14.70	8.11	1.95	2.40	1.90	-
	3	260.98	67.80	55.83	11.97	11.42	3.16	17.14	2.18	5.76	30.47
	3.5	44.47	42.40	27.41	14.99	15.84	10.96	1.14	2.59	1.32	-
	4	54.16	45.30	30.55	14.75	16.16	10.48	1.60	2.61	1.44	-
	4.5	169.20	55.38	30.39	24.99	12.71	4.72	5.55	2.31	3.80	22.01
	5	193.59	50.70	27.38	23.32	11.83	4.03	7.13	2.35	4.72	23.83

Note: E.G.L. means existing ground level.

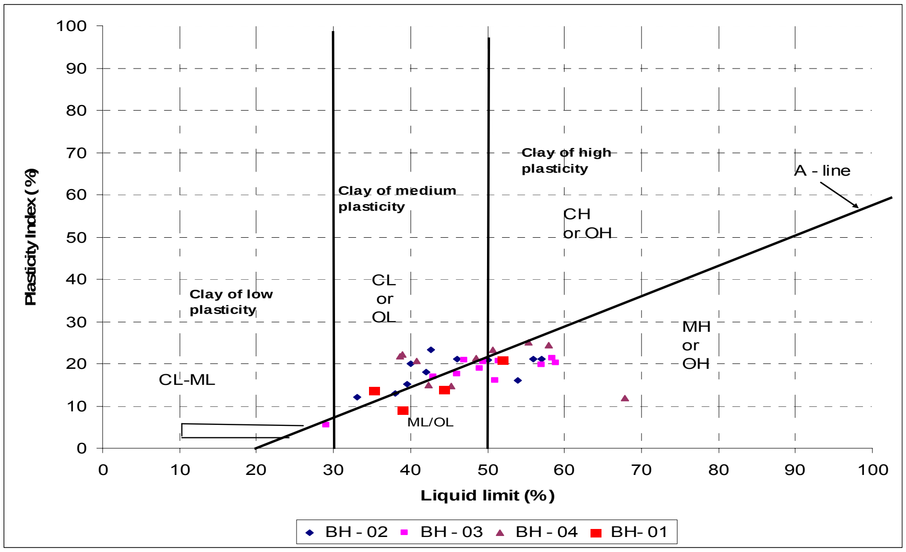


Figure 8.1: Classification of soils from BH – 01 to BH – 04

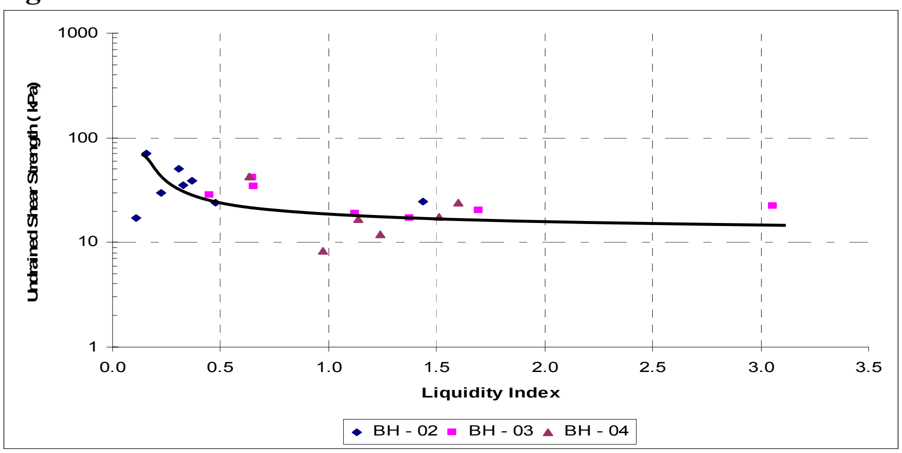


Figure 8.2 (a): Relationship between Undrained Shear Strength vs Liquidity Index for inorganic soil samples.

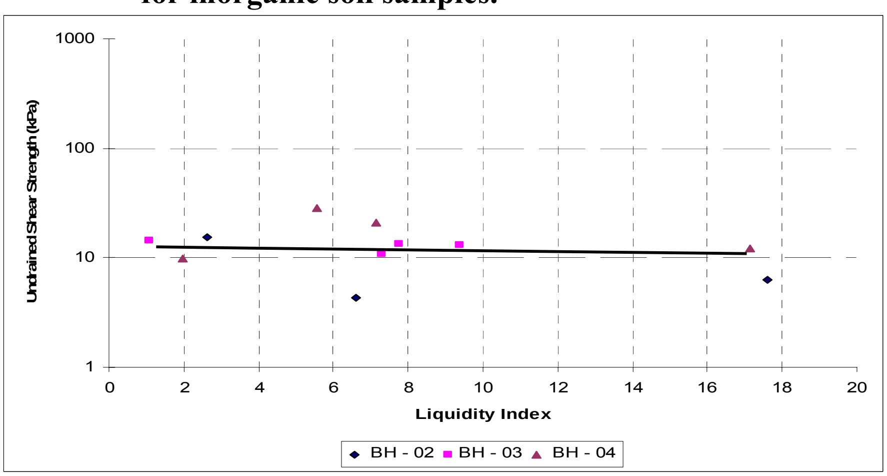


Figure 8.2 (b): Relationship between Undrained Shear Strength vs Liquidity Index for organic soil samples.

Besides this, the variations of moisture content with depth for soil samples of BH – 02 to 04 are shown in Figure 8.3. Results show that at a reduced level (RL) of 5 m, soil samples contain high moisture content such as 230 % to 260%. This layer was observed during ground investigation and was described as very high organic clayey silt. Besides this, another organic peat soil layer was observed in BH – 04 at a RL below 6.5 m. All other soils encountered at tube boreholes were inorganic in nature with natural moisture in the range of 28% to 65%. Moisture content of soil has a very significant effect on its strength. As noted before in data presented in Figures 8.2, its impact can be related to the index properties of the soils.

The relationship between initial void ratio and natural moisture content is shown in Figure 8.4. Results show a clear trend of increase in nature moisture content increase in initial void ratio and magnitude of natural moisture content with void ratio is expressed an equation, $e_o = 0.2314 + 0.0215w$. High initial void ratio results in high moisture content. This trend seems to apply for both organic and inorganic soils in the range of void ratio encountered.

The range of void ratio for inorganic and organic soils varies from 0.5 to 1.7 and 1.9 to 5.9 respectively. The corresponding range of natural moisture content for inorganic and organic soil varies from 25% to 60% and 80% to 260% respectively. It is evident that the void ratio for organic soil is almost four times higher than the inorganic one and the corresponding moisture content for organic soil is also three to four times higher than the inorganic one.

This higher void ratio also resulting the lower the compactness of the soils as well as lower the strength of the soils. Therefore, both high void ratio and moisture content can lead to increase settlement. This matter is discussed further in the following sections.

A plot of dry unit weight vs initial void ratio is shown in Figure 8.5 indicates that dry unit weight of the soil samples decreases with increase in void ratio. The relationship can be expressed in an equation by dry unit weight of soils (kN/m^3) = 12.623 – 5.686 ln(initial void ratio). Because if the soil skeleton has high voids that are filled by air as well as water, resulting contains lower amount of soil mineral particles.

From the Figure observed that the void ratio ranged from 0.5 to 5.8 and the corresponding dry unit weight ranged from 3.2 kN /m^3 to 15.8 kN / m^3. Dry unit weight of the soil sample plays vital role on strength and stability of the structure. Higher density soils have higher strength and stability.

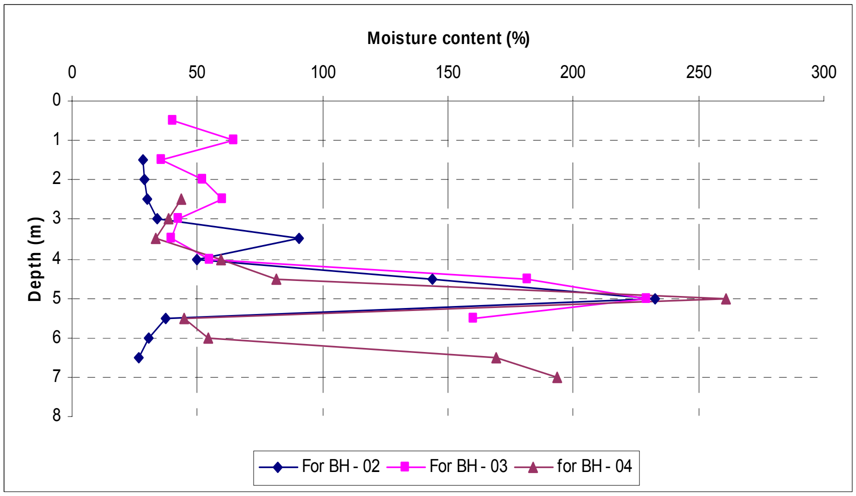


Figure 8.3: Variation of Moisture Content with Depth for soil samples of three holes.

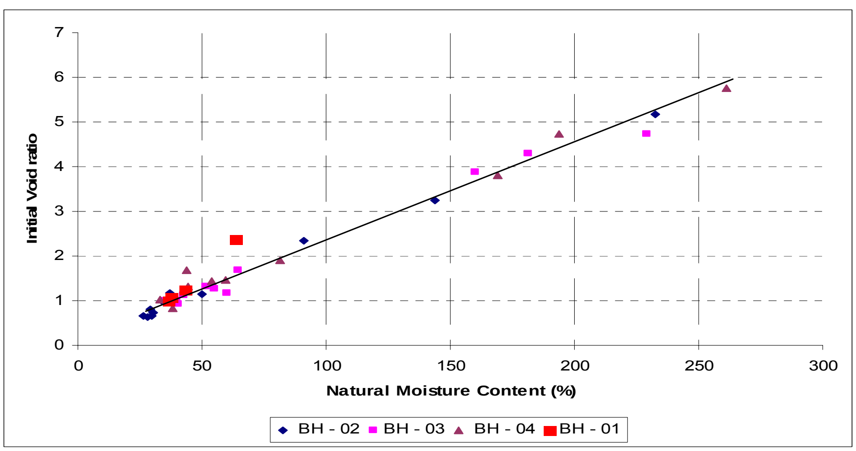


Figure 8.4: A Plot of Initial Void Ratio vs Natural Moisture Content.

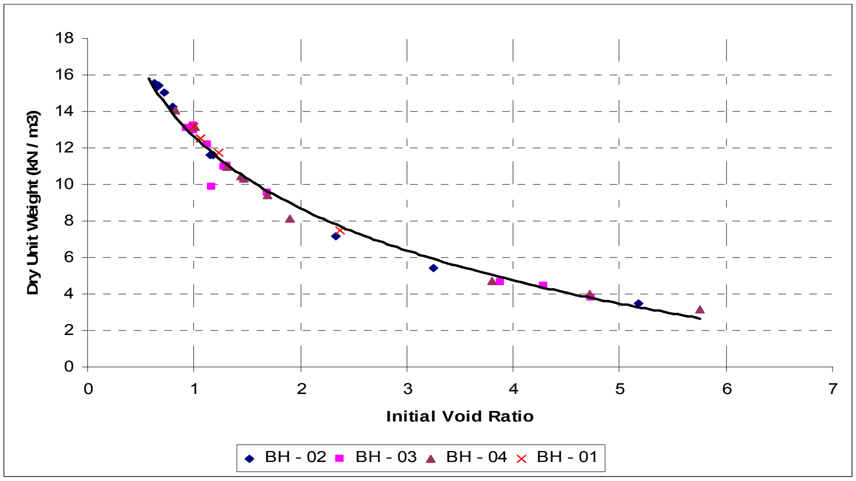


Figure 8.5: A Plot of Dry Unit Weight vs Initial Void ratio for soil samples.

Specific gravity of the soil samples collected from four boreholes is different for organic and inorganic soils. The specific gravity for inorganic and organic soil samples ranged from 2.55 to 2.7 and 2.18 to 2.42 respectively (Table 8.1). These values are compared with the usual range of specific gravity suggested by Bowles (1997) for inorganic and organic clay noted in article 3.3.1.2. The reason of lower than usual value may be attributed to the presence of some organic matter. However, these values for inorganic soils are almost similar to the values suggested by Ferthous (2007) for Bangladesh soils noted in above mentioned article.

So, it is obvious that inorganic soils have higher specific gravity than the organic one. The reason for lower specific gravity for organic soil is that it contains some organic or volatile matter which is light.

8.4 Organic Content of the soil:

Organic content of the soil samples collected from different boreholes at different depths are also presented in Table 8.1. Results show that the overall organic content ranges from 2.27% to 30.47% indicating that same soil samples have high organic content. Particularly the soils described as organic which have organic content ranging from 14.11% to 30.47%. In accordance with the classification made by BS EN ISO 14688 – 2:2004, soils encounter during the investigation can be described as low organic to high organic soils. Organic soils have very low shear strength and highly compressible that exhibit higher settlement for long time. Both strength and compressibility are of great importance for embankment design.

Variation of organic content with moisture content is shown in Figure 8.6. Results show that natural moisture content increases with increase in organic content. This is expected since soil skeleton containing large amount of organic matter, possess large amount of voids and these voids hold the water. The relationship between two parameters can be a grouped into relatively narrow band with an average relationship that can be represented by organic content (%) = 1.3 + 0.118 x moisture content (%).

This relationship is valid for the above soil samples but the organic content of the soil samples depends on the amount, types of organic matter presence and soils mineralogy.

A relationship between organic content and liquid limit are shown in Figure 8.7. As per soil classification made by BS EN ISO 14688 – 2:2004 based on organic content; three group's i.e. organic content of 2 to 6%, 6% to 20% and 20% to above are identified and are plotted against their respective liquid limit. Hence three relationships are found that are expressed as three equations.

The equation for 2% to 6% organic content is

$w_L = 34.983 + 2.4043 * (O.C)$

For 6 % to 20% organic content

$w_L = 40.979 + 0.9397 * (O.C)$

For 20% to above organic content

$w_L = 41.93 + 0.4172 * (O.C)$

Where, w_L is the liquid limit and O.C is the organic content of the soils.

The above mentioned relationship shows that the liquid limit increases with increase in organic content but the rate of increasing the liquid limit with respect to the organic content is higher for low organic soil and this rate decreasing with increasing the organic content of the soil.

Relationship of liquidity index with organic content of soil samples recovered from the boreholes are shown in Figure 8.8. The limited amount of results suggest an upper and lower bound band, with an average relationship of

Organic Content (%) = 7.53 + 1.54 x I_L (%).

Where, I_L is the liquidity index of the soil.

8.5 Particle Size Distribution of the soil sample:

Particle size distributions were conducted by sieve and hydrometer analysis. Soil samples collected from every one meter interval was tested and particle size distribution is shown in Figure 8.9. Results show that the soil samples contain sand, silt and clay particles. But some soil samples contain some colloidal particles that are suspended on the water. These suspended particles are the organic particles and oil of organic matter that are only seen in organic soil samples. However, constituent particles of the soil samples like sand, silt and clay ranged from 8% to 18%, 29% to 50% and 35% to 49% respectively. The above mentioned constituent indicates that soil samples contain high amount of silt and clay particles which results lower shear strength. From the distribution curves it is seen that particles are well graded in almost all soil samples.

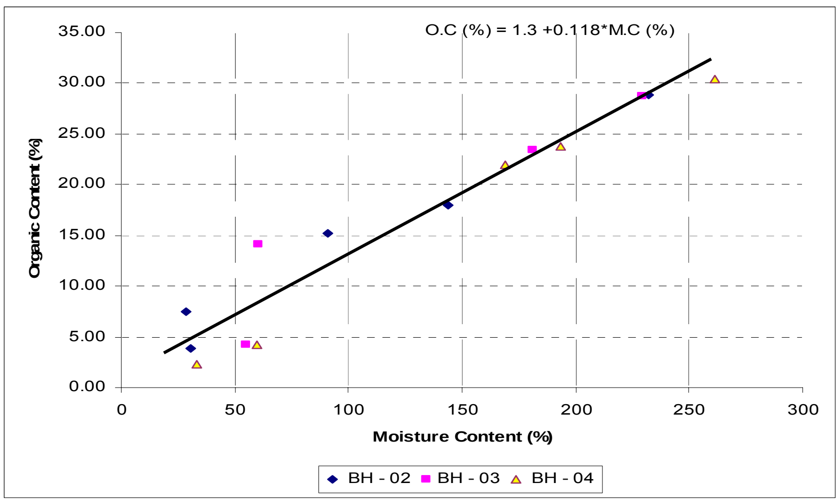


Figure 8.6: Relationship of Organic Content vs Moisture Content for soil samples.

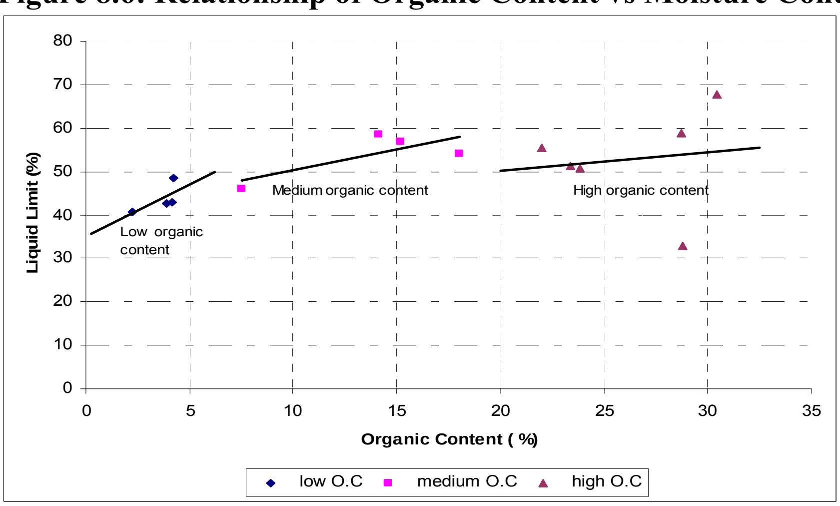


Figure 8.7: Relationship between Liquid Limit and Organic Contents of the soil samples

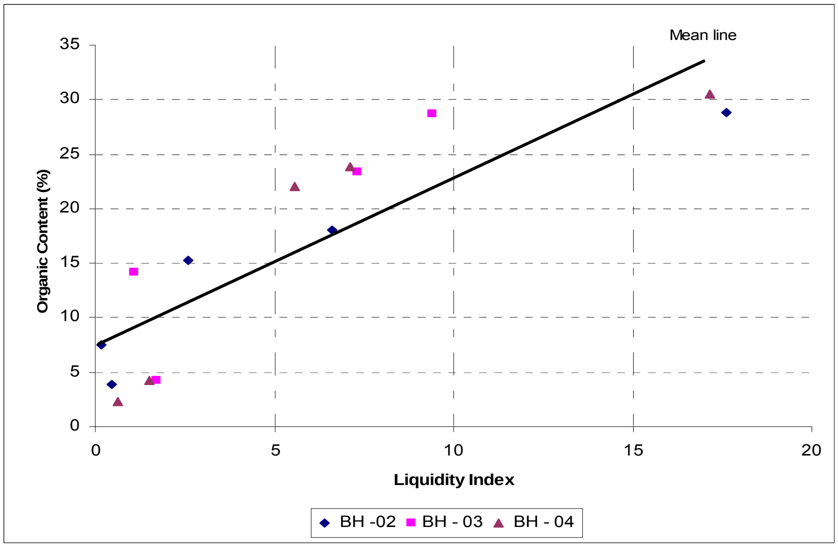


Figure 8.8: Organic content vs Liquidity index relationship of soil samples.

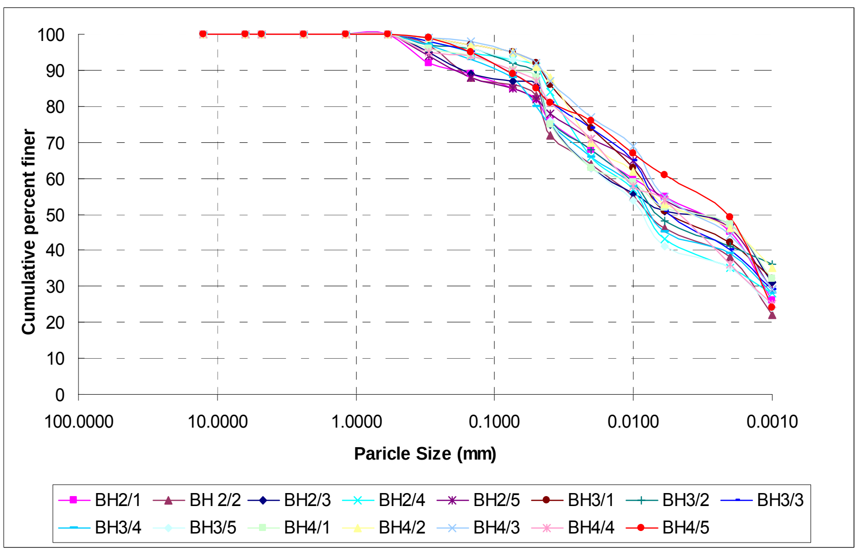


Figure 8.9: Particle Size Distribution curve for soil samples.

8.6 Undrained Shear Strength of Soils

Unconfined compressive strength (UCS), hand vane shear test and hand penetrometer test were conducted on undisturbed soil samples collected from different boreholes from a range of depths. The UCS determinations were made on total 37 undisturbed samples of cohesive soil collected from BH – 01 to BH - 04. Hand vane shear test and hand penetrometer test were done on 32 undisturbed soil samples collected from BH – 02 to BH – 04. All the strength determinations are shown in Table 8. 2.

Variations of undrained shear strength (C_u) with reduced level for soil samples of three boreholes (BH- 02 to BH – 04) are shown in Figure 8.10. It is observed that the undrained shear strength varies from 5 kPa to 40 kPa that indicates the soil samples are vary low to low shear strength. At 4 m to 5 m depth the shear strength of the soil samples is in the range of 5 to 13 kPa that indicates extreme low undrained shear strength. At this depth there is a black organic soil layer. Again, the soil samples collected from BH - 03 at a depth of below five meter also show very low shear strength indicating the organic soil layer. On the other hand, soil sample of BH – 02 at 1.5 m RL shows shear strength of 70 kPa indicating the soils are medium stiff consisting low moisture content. It is possible that upper layer of soil has undergone some drying. The above mentioned undrained shear strength values are within the range suggested by previous researchers such as Ferthous (2007) and Serajuddin (1998) for the depth of 9.5 m (from ground surface) investigated at Bangladesh as noted in article 3.3.2 of chapter three.

A relationship has been found between undrained shear strength obtained from UCS test and hand Vane shear test that is shown in Figure 8.11. Results suggested that the undrained shear strength determined from UCS test ranges from 5 kPa to 70 kPa, whereas the same value found from hand vane shear varies from 7 kPa to 74 kPa. There is thus a good relationship of undrained shear strength determined from hand vane and UCS test. This relationship for the soils examined can be expressed by a linear equation:

$$Cu\ (vane) = 4.662 + 0.968 * Cu\ (UCS).$$

Where, Cu (vane) & Cu (UCS) are the undrained shear strength found from the vane shear test and the UCS test respectively. The unconfined compressive strength is done only in laboratory and it takes more time and effort than hand vane shear test. Whereas, hand vane shear test is very simple, quick and can be done at field. So, by performing the simple and easy hand vane shear test at field and using the above mentioned correlations, undrained shear strength can be determined.

Besides this, another tool for determining the undrained shear test at field is the pocket penetrometer which gives the undrained compressive strength of soils. It is very simple and quick test to estimate the unconfined compressive strength of soils. The resistance to penetration depends on the types of the soils; its mineralogy and moisture content. However, a good linear relationship for undrained shear strength is observed that is found from UCS and hand penetrometer test. The relationship is shown in Figure 8.12. It is also observed that hand penetrometer gives the more shear strength value for same soil sample than UCS test and the relationship is

Cu (Penetrometer) = 7.2 + 0.94 * Cu (UCS).

Where, Cu (penetrometer) is the undrained shear strength found from the penetrometer test and Cu (UCS) is the undrained shear strength found from the UCS test. So by performing the simple test in field, the undrained shear strength can be determined using the above mentioned correlation for the soils examined in this study.

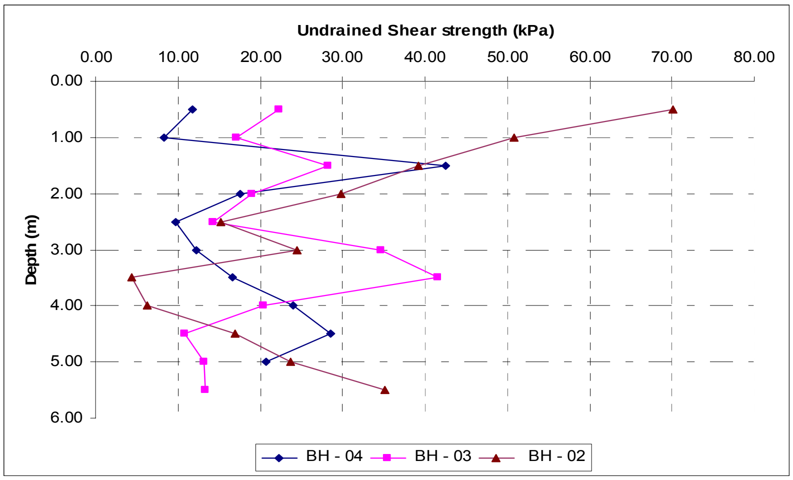


Figure 8.10: Variation of Undrained Shear Strength with Depth

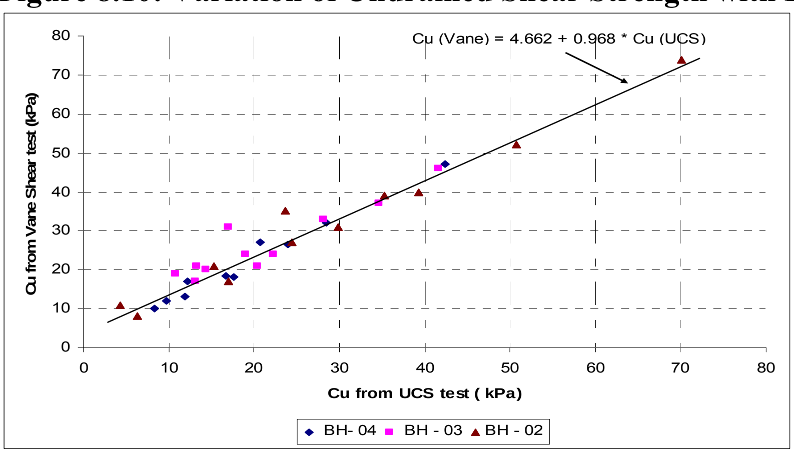


Figure 8.11: A plot of shear strength relationship found from UCS test and hand Vane shear test

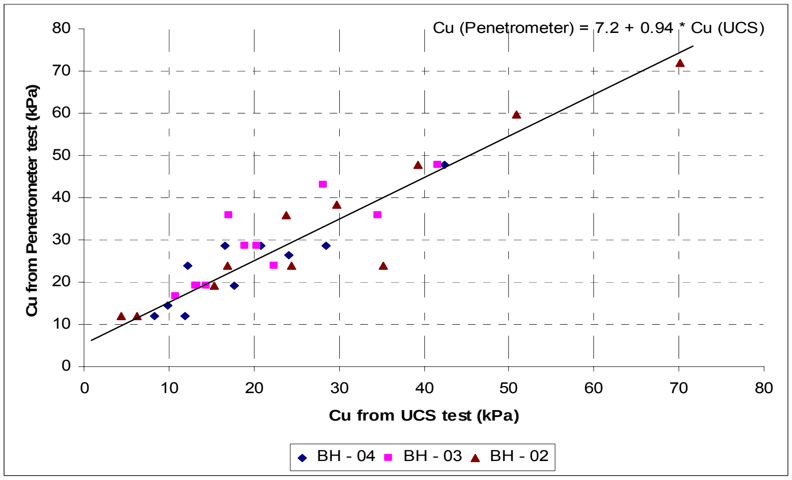


Figure 8.12: Shear strength relationship found from UCS and hand Penetrometer test.

Table 8.2: Undrained, Drained and Residual shear strength parameters for soil samples collected from different depths of BH -1 to 4.

BH No.	Dept-h from E.G.L	Undrained cohesion (Cu) from UCS test, (kPa)	Undrained cohesion (Cu) from Vane shear (kPa)	Undrained cohesion (Cu) from Penetrome-ter, (kPa)	Undrained shear parameter (Triaxial test)			Drained shear parameter (Direct shear box test)		Residual Shear parameters (Ring shear test)	
					C_u (kPa	ϕ (deg	Eu (MPa	C' (kPa	ϕ' (deg	C_r (kPa	ϕ_r (deg
1	3	47.03	-	-	-	-		-	-	-	-
	6	40.7	-	-	-	-		-	-	-	-
	9	44.04	-	-	-	-		-	-	-	-
	12	63.85	-	-	-	-		-	-	-	-
2	0.5	70.12	74	71.82	27	10.3	3.34	4.55	18.2	0.88	4.68
	1	50.80	52	59.75	-	-	-	-	-	-	-
	1.5	39.25	40	47.88	-	-	-	-	-	-	-
	2	29.79	31	38.3	-	-	-	-	-	-	-
	2.5	15.27	21	19.15	-	-	-	-	-	-	-
	3	24.47	27	23.94	-	-	-	-	-	-	-
	3.5	4.34	11	11.97	-	-	-	-	-	-	-
	4	6.327	8	11.97	28	10.4	2.4	6.53	24.4	0.11	6.48
	4.5	16.95	17	23.94		-	-	-	-	-	-
	5	23.72	35	35.91	25	2.3	2.24	4.5	18.1	0.84	6.33

	5.5	35.21	39	23.94	-	-	-	-	-	-	-
3	0.5	22.35	24	23.94	-	-	-	-	-	-	-
	1	17.09	31	35.91	-	-		-	-	-	-
	1.5	28.20	33	43.09	-	-	-	-	-	-	-
	2	18.98	24	28.73	-	-	-	-	-	-	-
	2.5	14.33	20	19.15	23	10.4	3.6	6.52	12.8	0.76	5.72
	3	34.66	37	35.91	-	-	-	-	-	-	-
	3.5	41.60	46	47.88	-	-	-	-	-	-	-
	4	20.38	21	28.73	13	4.7	1.88	4.22	19.0	1.25	5.85
	4.5	10.84	19	16.76	-	-	-	-	-	-	-
	5	13.11	17	19.15	19	6.9	2.5	2.96	16.7	0.63	5.39
	5.5	13.34	21	19.15	-	-	-	-	-	-	-
4	0.5	11.84	13	11.97	-	-		-	-	-	-

Cont…

BH No.	Depth from E.G.L	Undrained cohesion (Cu) from UCS test, (kPa)	Undrained cohesion (Cu) from Vane shear (kPa)	Undrained cohesion (Cu) from Penetrometer, (kPa)	Undrained shear parameter (Triaxial test)			Drained shear parameter (Direct shear box test)		Residual Shear parameters (Ring shear test)	
					C_u (kPa	ϕ (deg	Eu (MPa	C' (kPa	ϕ' (deg	C_r (kPa	ϕ_r (deg
4	1	8.36	10	11.97	-	-	-	-	-	-	-
	1.5	42.43	47	47.88	22	3.4	3.25	4.49	13.9	1.06	5.11
	2	17.64	18	19.15	19	5.9	2.4	4.07	14.6	0.55	7.87
	2.5	9.80	12	14.36							
	3	12.16	17	23.94	28	4.5	2.55	5.88	24.3	2.43	6.9
	3.5	16.64	18.5	28.73	-	-		-	-	-	-
	4	24.05	26.5	26.33	-	-	-	-	-	-	-
	4.5	28.52	32	28.72	-	-		-	-	-	-
	5	20.75	27	28.73	-	-	-	-	-	-	-

Remarks: Triaxial, Direct shear box & residual shear tests were done on remoulded soil samples.

Variation of undrained shear strength with moisture content is shown in Figure 8.13. Results show that the undrained shear strength decreases sharply from 50 kPa to 14 kPa with variation of moisture content from 28% to 50%. So the variation of moisture content from 30% to 50% affects almost 70% reduction of undrained shear strength value and after that there is a little affect on shear strength with change in moisture content. Most of the data upto 50% moisture content related to inorganic soils. The organic soils that have above 50% moisture content shows low shear strength and this strength does not vary significantly with change in moisture content.

A relationship between ratio of undrained cohesion (from UCS test) to effective overburden pressure and liquidity index of the soil samples is shown in Figure 8.14. This relationship is compared with the [15]Bjerrum & Simons (1960) equation noted in 4.2.5 of chapter four is also shown on that figure which shows lower value of Cu/p' than the test results. Results also show that the effective over burden pressure varies from 6.5 kPa to 42 kPa and the ratio of undrained shear strength to effective overburden pressure varies from 0.27 to 9.5. The relationship between this ratio and the liquidity index for the soil samples for this study are expressed as an equation

$Cu / p' = 1.0883 \times (I_L)^{-0.4022}$

Whereas the same relationship for undrained shear strength from vane shear test is found and presented in Figure 8.15 and this can also be expressed in an equation is as follows

$Cu / p' = 1.30 \times (I_L)^{-0.322}$

A plot of undrained shear strength to effective overburden pressure and plasticity index is shown in Figure 8.16. Results show that there is no definite relationship exists between them which are supported by Sridharam & Narasimha (1973) comments noted in article 4.2.1.2 of chapter four. The test results are compared with the equations proposed by Skempton and Henkel (1953) and Bjerrum & Simons (1960) noted in equation number 4.2.3 and 4.2.4 respectively of chapter four is shown on that Figure for making comparison. Results show that most of the data lies in between two equations but no definite relationship is followed.

[15] The equation is valid for I_L is >5%

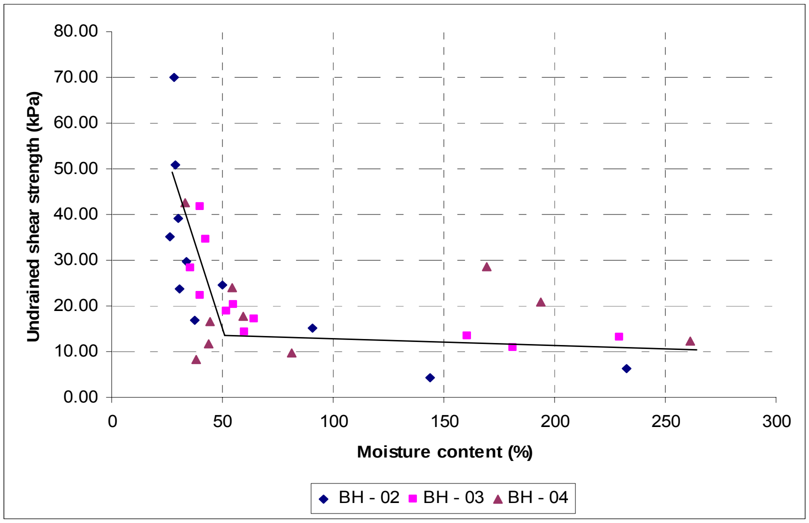


Figure 8.13: Variation of undrained shear strength (from UCS test) with moisture content for soil samples.

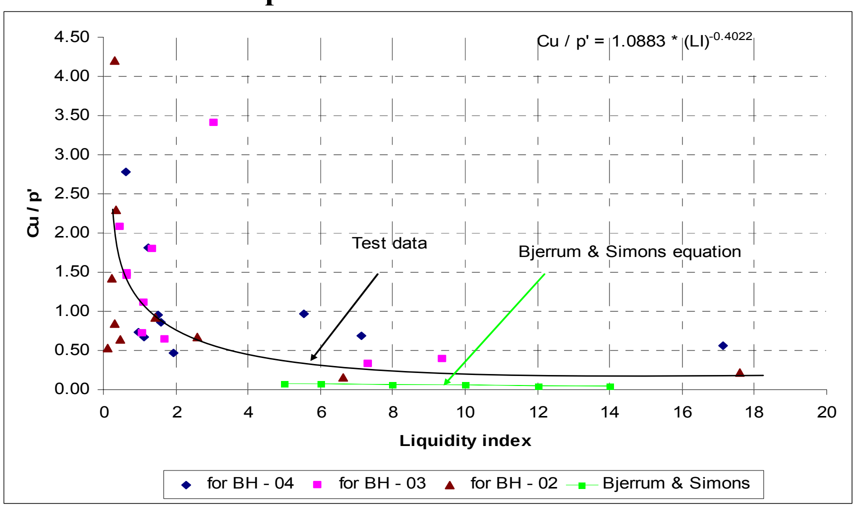


Figure 8.14: Relationship between undrained shear strength (from UCS test) to effective overburden pressure and Liquidity index.

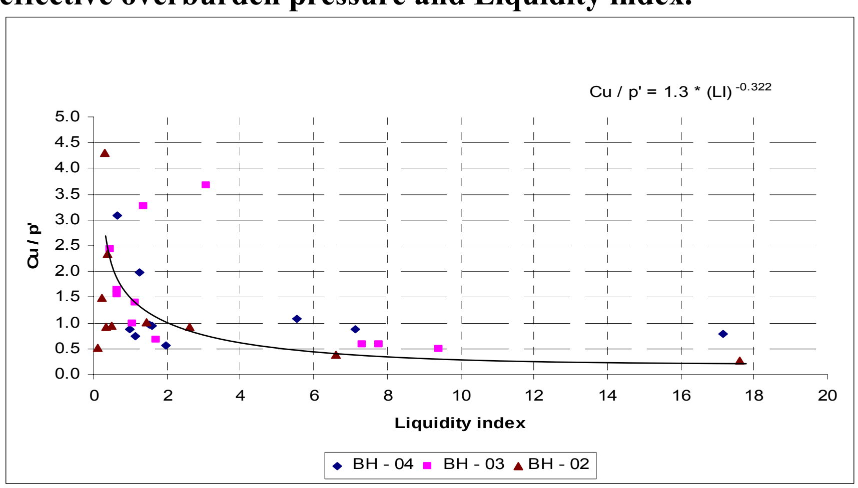


Figure 8.15: Relationship between undrained shear strength (from vane shear test) to effective overburden pressure and Liquidity index.

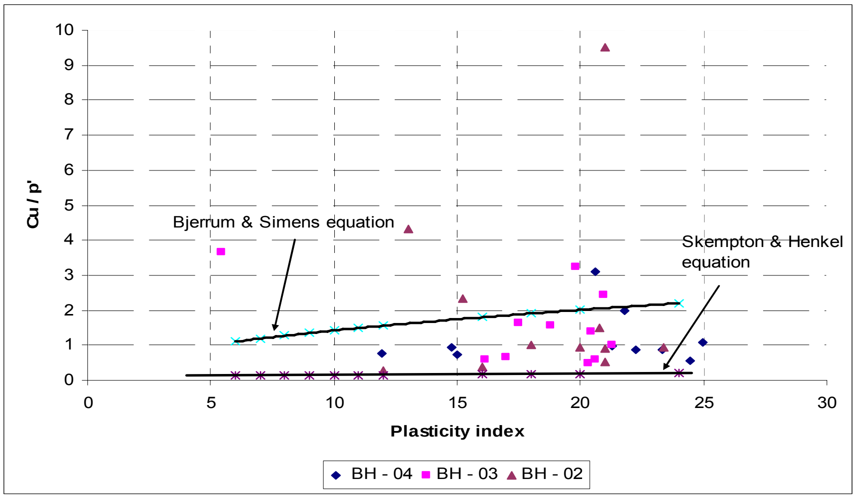


Figure 8.16: Relationship between Undrained Shear Strength to effective overburden pressure and Plasticity index.

8.6.1 Undrained Shear Strength from Triaxial test:

The triaxial test was conducted on nine remoulded soil samples at different moisture content. Among them, five soil samples are inorganic and four are organic in nature collected from boreholes. The test specimens were prepared by dietert apparatus. As far as possible, specimens were made at the field moisture content. Some small deviation did occur due to losses of moisture during sample preparation.

A typical stress – strain curve for undrained triaxial test is shown in Figure 8.17. Results show that failure seems to lie 12% to 15% of strain (presuming the x – axis are in fraction). In other word, axial strain is faster for soft soil samples though it hasn't reached to its ultimate failure stress. This type of behaviour causes trouble for embankment construction on that soft soil since it causes excessive settlement for any structure like highway embankment.

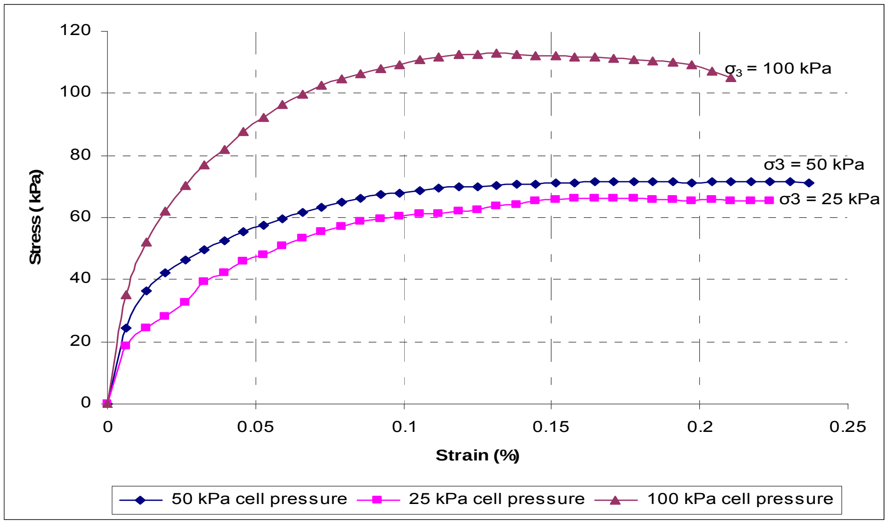


Figure 8.17: A typical plot of deviator stress vs axial strain curve for remoulded soil samples (BH-3/C6480/D -2.5)

The undrained cohesion (C) and angle of internal friction (ϕ) at different moisture content have determined from the Mohr's circle envelope. A typical Mohr's circle envelope was drawn for a soil sample of BH-03/C6480/D – 5 subjected to three different confining pressures is shown in Figure 8.18. The undrained cohesion and angle of internal friction is also shown in Table 8.2. The variation of undrained shear strength with moisture content is shown in Figure 8.19. Results show that there is no single line relation exists between them but it follows a band width represented by an average line. This may occur because both organic and inorganic soil samples are tested and results are plotted together. Organic and inorganic soils have different behaviour and different moisture susceptibility. However, undrained shear strength decreases with increase in moisture content of the soils. The test results show that the undrained shear strength ranges from 13 kPa to 28 kPa over moisture content range of 28% to 60%. The angle of internal friction for the soil samples varies from 2 to 10 degrees.

The relationship between undrained shear strength determined from triaxial test to liquidity index for remoulded inorganic soil samples are shown in Figure 8.20. The value of liquidity index varies from 0.35 to 0.8 and the corresponding undrained shear strength varies from 13 kPa to 27 kPa. When the liquidity index is equal to one, in that case the natural moisture content is equal to liquid limit and the corresponding undrained shear strength is equal to almost 12 kPa which is the same value as found from Tarzahi's chart.

It is mentioned that Terzahi's chart was developed for soil samples collected from different parts of the world which was described in Figure 4.1 of chapter four. A relationship is therefore derived between undrained shear strength and liquidity index expressed in an equation:

$Cu = 11.68 * (I_L)^{-0.8109}$

Where, Cu is the undrained cohesion in kPa and I_L is the liquidity index of the soil sample.

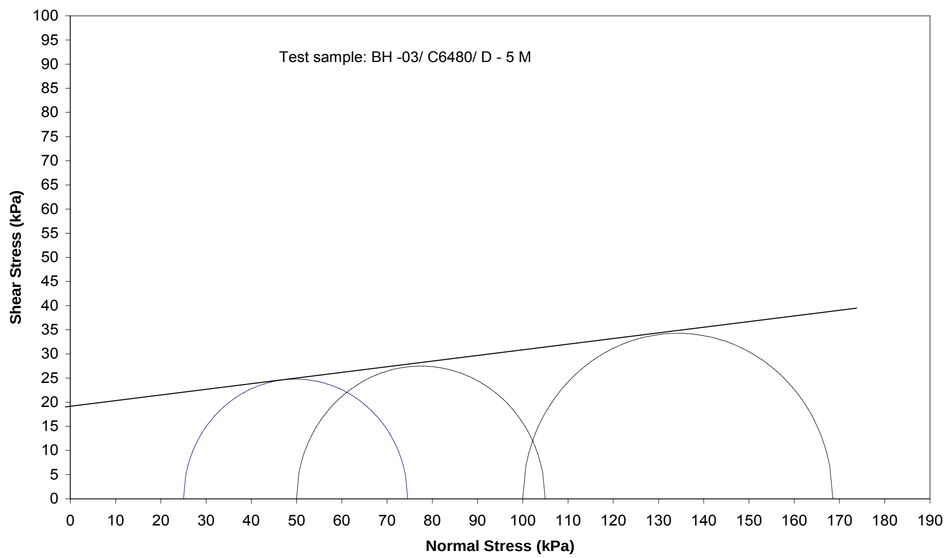


Figure 8.18: A typical relationship between Shear Stress vs Normal Stress for soil sample.

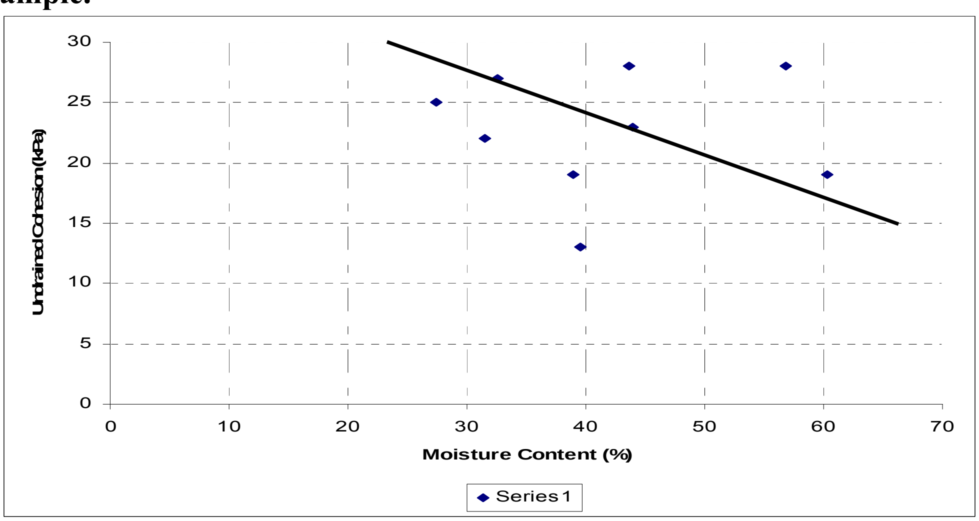


Figure 8.19: A plot of Undrained Cohesion found from triaxial test vs Moisture Content for organic and inorganic remoulded soils

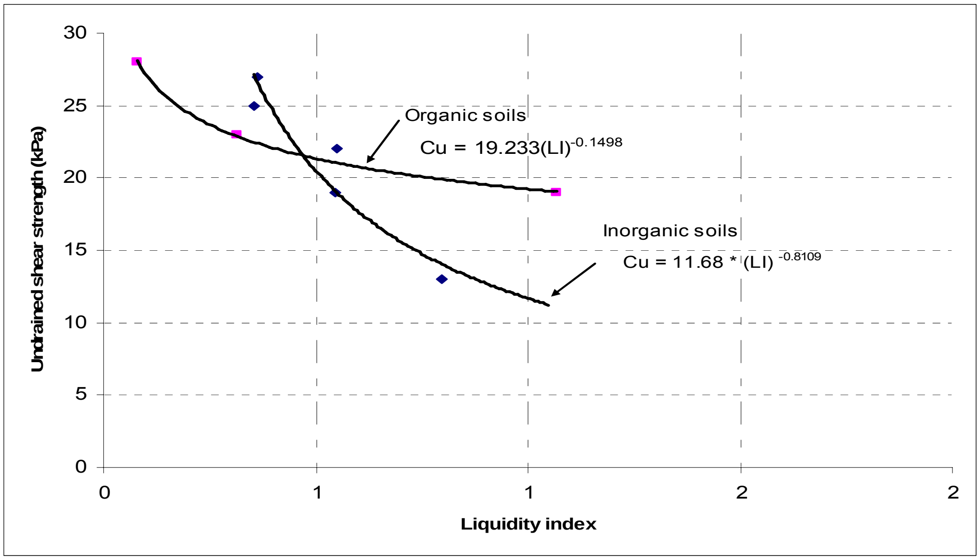


Figure 8.20: Relationship between Undrained Shear Strength (from triaxial test) & Liquidity Index for remoulded soils samples.

On the other hand, the same relationship is plotted for organic remoulded soils are also shown in figure 8.20. Due to the higher moisture content, organic soils have higher liquidity index. Its liquidity index varies from 0.1 to 1.1 and the corresponding undrained shear strength varies from 18 to 28 kPa. The relationship can be expressed in an equation $Cu = 19.233 \times (I_L)^{-0.1498}$

Range of undrained shear strength with undrained modulus for remoulded soil samples is shown in Figure 8.21. This undrained modulus has been determined by triaxial test for 50 kPa cell pressure because the maximum effective overburden pressure for the soil samples collected from different boreholes is about 42 kPa. Results show that the undrained modulus ranges from 1.9 MPa to 3.6 MPa. This undrained modulus value is almost similar to the value determined by Kabir et al. (1997) for Bangladesh clayey soil, noted in article 3.3.3 of chapter three. In addition to that Skempton & Henkel (1957) as well as Uddin (1990) equations noted in article 4.2.2 of chapter four are plotted on the same graph to compare the test results. Results show that the average trend line of the data is almost similar to the Skempton & Henkel (1957) equation. Again the plotted data also show the value of undrained modulus increases with increase in the undrained shear strength.

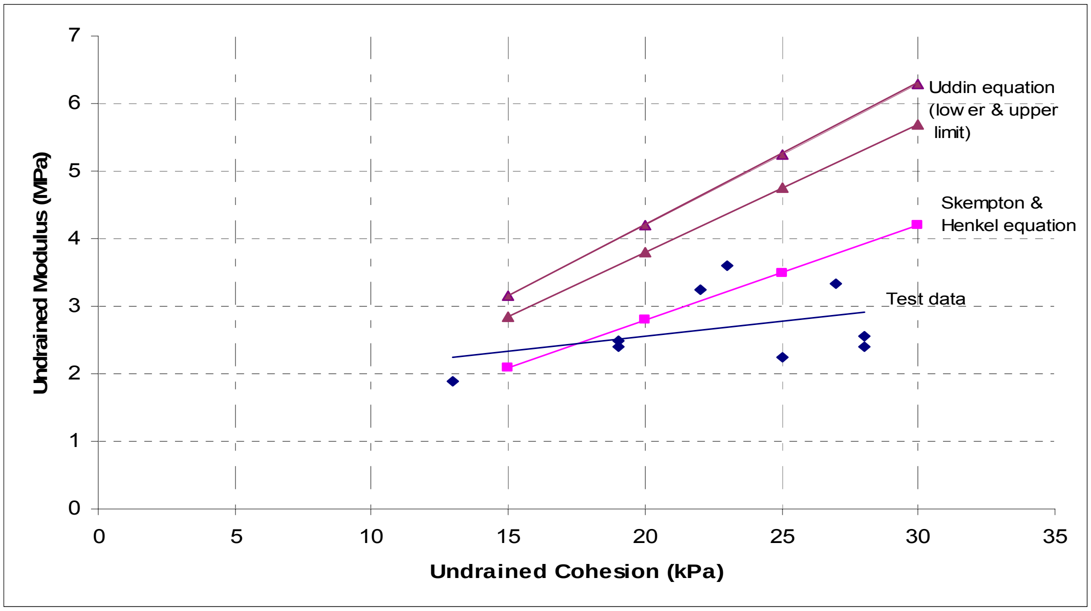


Figure 8.21: A plot of undrained modulus vs undrained shear strength (from triaxial test) for remoulded soil samples

8.6.2 Drained and Residual Shear Strength parameters:

The drained peak and residual shear strength parameters were determined by direct shear box test and ring shear test respectively. The values of the peak and residual shear strength parameters are also given in Table 8.2. Results show that the peak and residual cohesion ranges from 3 kPa to 6.5 kPa and 0.1 kPa to 2.4 kPa respectively. However, the former and later drained angle of friction ranges from 13 to 24 degree and 5 to 8 degree respectively.

The drained shear strength parameters, so obtained, have been compared with the usual range of values of such parameters suggested by Peck et al. (1974) and Murthy (2002) as noted in chapter three. The obtained drained angle of friction is lower than the values suggested by the above mentioned researchers. The reason may be attributed to the presence of organic matter in the soils. Besides this, a typical plot of peak and residual shear stress vs. normal stress is shown in Figure 8.22. Theoretically the residual stress line should pass from the origin, i.e. the residual cohesion should be zero. But practically it is not zero (very insignificant value) because of the elastic behaviour of the cohesive soils.

A relationship between peak and residual angle of friction with plasticity index is shown in Figure 8.23. It is observed that the drained angle of friction decreases with increase in plasticity index and can be expressed in an equation

$\phi^{/}$ (deg) = 148.67 x $(I_P)^{-0.7343}$

Meanwhile, similar relationship between drained angle of friction and plasticity index established by Gibson (1953) were plotted in the same figure to compare the test results. Results show that soils have lower drained peak and residual shear strength than those found from the Gibson's (1953) equation. But changing pattern of angle of friction with plasticity index is almost similar to the Gibson's equation. Both drained shear strength decreases with increase in plasticity.

However, another relationship was observed between residual angles of friction with plasticity index and can be expressed in an equation

$\phi_r^{/}$ = 10.969 x $(I_P)^{-0.2095}$

So, it is evident that the variation of drained residual angle of friction with plasticity index for different soil samples is insignificant.

8.7 Embankment fills soils properties assessed by DCP test

This CBR values were estimated from DCP test at top and toe of embankment. The soil samples were collected with the DCP sampler to determine the moisture content of the soils. A relationship between CBR value and moisture content for both embankment fills and foundation soils of embankment are shown in Figure 8.24. Results shows that fill soils can exhibit less moisture content and high CBR value compared to the foundation soils. Again, observed that some of the embankment fill and foundation soils have almost same moisture content and shows almost same CBR value. So it suggests that both fill and foundation soils of embankment are the same soils that found in embankment foundation.

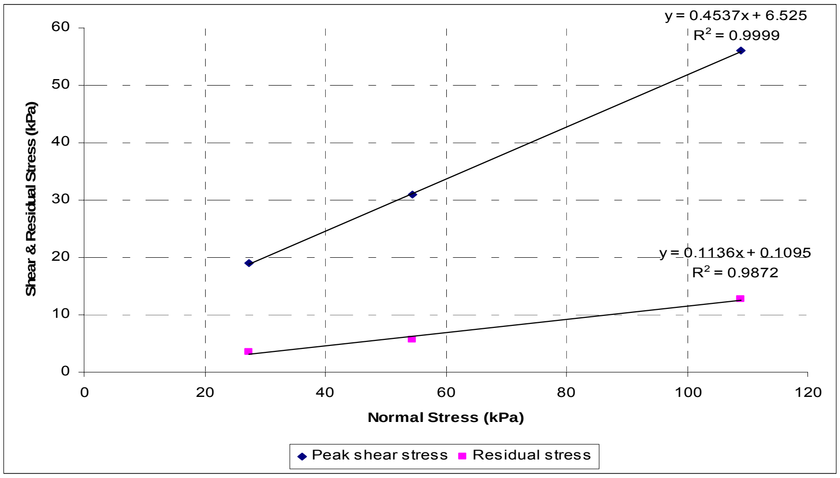


Figure 8.22: A typical plot of Peak and Residual Stress vs Normal Stress for remoulded soil samples of BH – 02/D4

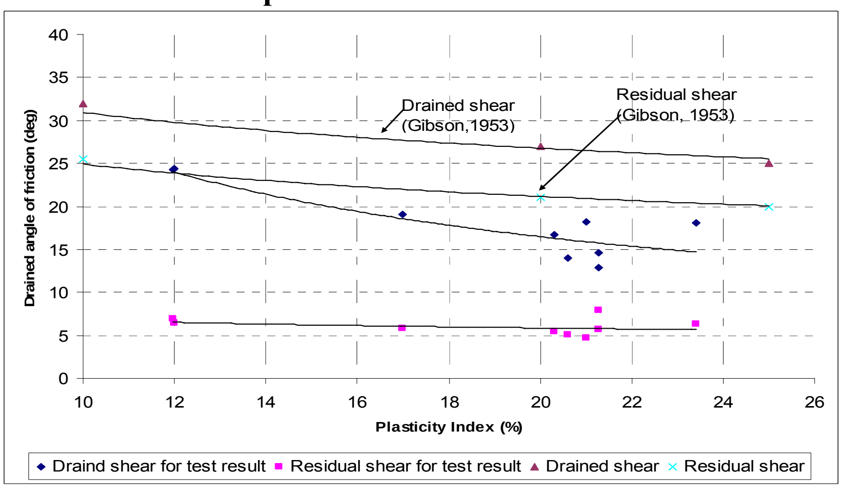


Figure 8.23: A plot of Peak and Residual drained angle of friction vs Plasticity index for remoulded soil samples

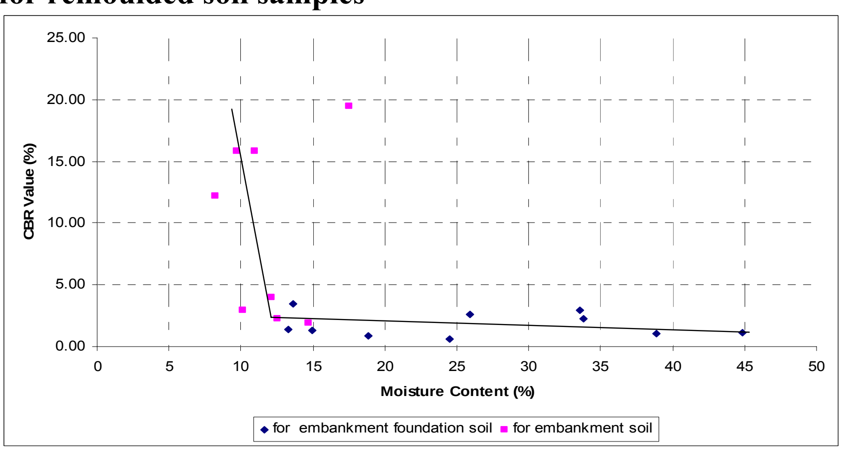


Figure 8.24: Variation of CBR value with moisture content for foundation & fill soils of embankment.

8.8 Consolidation Properties of soils

One dimensional consolidation test were performed to determine the compressibility properties of five remoulded inorganic and four organic soil samples collected from different boreholes and at different depths. The results obtained from these tests are discussed below.

The test results of nine samples collected from three boreholes are given in Appendix G, Table A- 8.2. A typical void ratio and pressure relationship for organic and inorganic soils is shown in Figure 8.25. It is seen from the figure that top three curves are organic soils as these shows higher void ratio as well as higher rate of consolidation. When the pressure is applied to the soil samples, void ratio is decreased due to the release of water from the soil samples. The soil samples is than allowed for swelling by releasing pressure on it and after that it is again subjected to reload to determine the recompression behaviour.

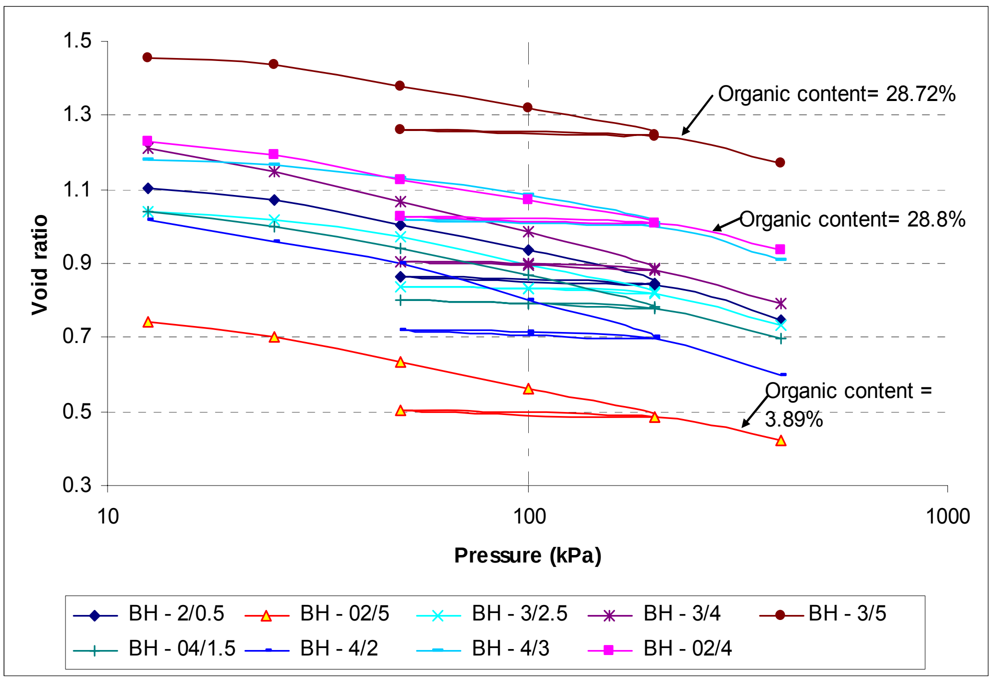


Figure 8.25: Relationship between Void ratio and Pressure for soil sample

8.8.1 The Coefficient of Consolidation

Two methods are used for determination of the coefficient of consolidation (Cv) such as Casagrande's method and Taylor's method. In this research work, Cv value for both organic and inorganic soil was determined by Taylor's method because Casagrande's method does not consider primary consolidation ratio (r) correction (Raymond et al., 1986). However, some Cv value is calculated by both method for made comparison is shown in Appendix G, Table A-8.3. On the other hand, determination of time for 50% and 90% consolidation were calculated using both methods are also shown in Appendix H, Figure B- 8.1.

The variations of coefficient of consolidation with pressure for soil samples collected from three boreholes (BH – 02 to 04) are shown in Figure 8.26. Results show that the value for organic soil is much higher than the inorganic one. This is because the organic soils exhibit higher consolidation with pressure than the inorganic one. For soil samples of BH – 02, it is seen that one sample is organic and rest two are inorganic soil. The value of the coefficient of consolidation for inorganic and organic soils ranges from 0.421 m^2/ yr to 2.25 m^2/ yr and 21.95m^2/yr to 39.629 m^2/ yr respectively. It is seen that organic soils shows initial higher consolidation rate than the inorganic one. Because initially the remoulded soil samples contains higher voids that may fill with air in addition to water and after applying the pressure it shows the higher settlement due to the expelling of air that contributes higher settlement. Whereas the inorganic soil samples shows the coefficient of consolidation does not vary significantly with pressure as like as organic soils.

For soil samples of BH – 03 shows two samples exhibit higher consolidation with pressure than the other one which indicates these two soil samples are organic in nature. The value of the coefficient of consolidation varies for inorganic and organic soils are 0.68 m^2/ yr to 1.702 m^2/ yr and 0.409 m^2/ yr to 17.438 m^2/ yr respectively.

For soil samples of BH – 04 shows one samples exhibit higher rate of consolidation than other two samples indicating this sample is organic in nature and other two samples are inorganic soils. However, the value of the coefficient of consolidation for inorganic and organic soils varies from 0.569 m^2/ yr to 7.267 m^2/ yr and 6.908 m^2/ yr to 46.369 m^2/ yr respectively. It is seen that for inorganic soils the coefficient of consolidation does not vary significantly with pressure whereas for organic soils it varies significantly with pressure. Because remoulded organic soil samples contains higher voids than inorganic one and organic soils are more compressible than inorganic one when applying pressure on it.

Again, it is observed that the rate of consolidation is different for compression and recompression. Its value decreases from compression to recompression for organic soils whereas the value increases from compression to recompression for inorganic soils. This is because organic soils exhibit higher compression and lower swelling than inorganic one. As the swelling is less in organic soils, it takes less water, hence during recompression; the rate of consolidation is comparatively less and vice versa for inorganic soils.

Moreover, the overall variation of the coefficient of consolidation for inorganic and organic soils are 0.421 m^2/ yr to 7.267 m^2/ yr and 0.409 m^2/ yr to 46.369 m^2/ yr respectively. These Cv values are within the range of prescribed Cv values (1.73 to 100.91 m^2/ yr) for Bangladesh soils that suggested by Serajuddin (1998). Besides this, the above mentioned values are within the range of specified values of 0.1 to 12 m^2/ yr suggested by different researchers for different types inorganic soils as noted in chapter three (Table 3.7).

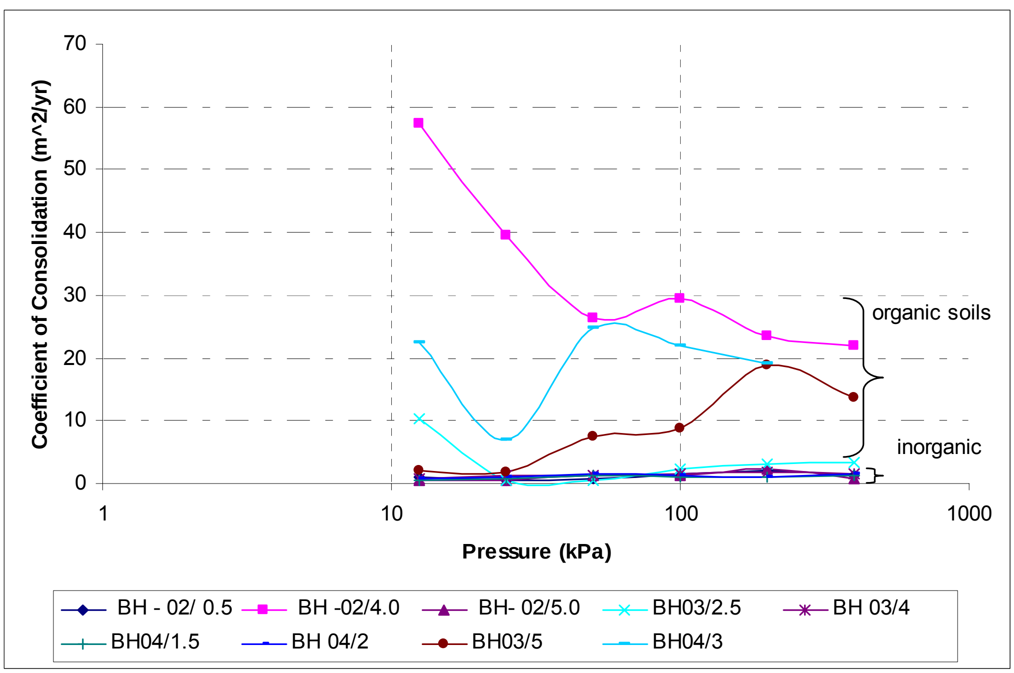


Figure 8.26: A plot of coefficient of consolidation vs pressure for soil samples of BH-02 to 04.

8.8.2 Coefficient of Volume Compressibility

A relationship between coefficient of volume compressibility (Mv) and pressure for soil samples collected from three boreholes (BH – 02 to 04) are shown in Figure 8.27. Both coefficient of volume compressibility and pressure are plotted in log scale and the relationship shows almost straight line. From the relationship it is seen that coefficient of volume compressibility decreases with increase in pressure for both organic and inorganic soils. Initially the value of coefficient of volume compressibility is high and it is decreased with increase in pressure. Besides this, its value is slightly lower in organic soils than inorganic ones. The value ranges from 0.26 m^2/MN to 3.05 m^2/MN, 0.22 m^2/MN to 3.78 m^2/MN and 0.18 m^2/MN to 1.93 m^2/MN for three soil samples of BH - 2 due to the change in pressure 12.5 kPa to 400 kPa.

The same relationship was observed for three soil samples of BH – 03 where two samples are organic and one is inorganic in nature. The value ranges from 0.26 m^2/MN to 2.59 m^2/MN , 0.18 m^2/MN to 4.41 m^2/MN and 0.26 m^2/MN to 4.83 m^2/MN due to the change in pressure 12.5 kPa to 400 kPa. Again, similar relationships were observed in soil samples of BH – 04 where organic soil samples shows lower coefficient of volume compressibility than the inorganic one. Its value ranges 0.298 m^2/MN to 3.55 m^2/MN, 0.30 m^2/MN to 5.46 m^2/MN and 0.24 m^2/MN to 2.72 m^2/MN due to the change in pressure from 12.5 kPa to 400 kPa. However, the initial value of coefficient of volume compressibility shows much higher for all soil samples as the remoulded soil samples have higher void ratio consisting of air with water. It is also seen that for same pressure change, organic soils exhibit higher change in coefficient of volume compressibility than the inorganic ones as the organic soils contains more voids than inorganic ones. After first application of load, the air in addition to water (partial) is expelling from the soils hence the coefficient of volume compressibility is higher. Considering the above mentioned factors, coefficient of volume compressibility ranges from 0.18 m^2/MN to about 2 m^2/MN which are within the limiting values specified for soft and organic clay by G. Barres, (2001), as noted in chapter three (Table 3.8) of this thesis.

The coefficient of volume compressibility depends on the void ratio of the soil samples. Hence the variation of the coefficient of volume compressibility with void ratio is shown in Figure 8.28 where the coefficient of volume compressibility is in log scale. Results show that initially void ratio was high and it is decreased as the pressure is increased resulting coefficient of volume compressibility sharply decreased for both organic and inorganic soil samples. It is also seen from the graph; the organic soils have high void ratio and its variation with coefficient of volume compressibility is similar to that of the inorganic one.

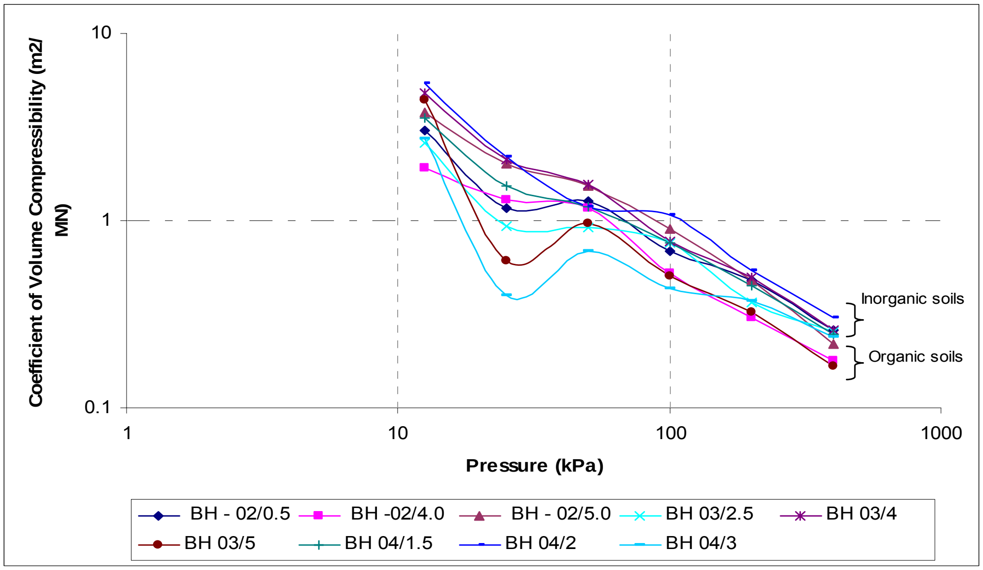


Figure 8.27: A plot of coefficient of volume compressibility vs pressure for soil samples of BH – 02

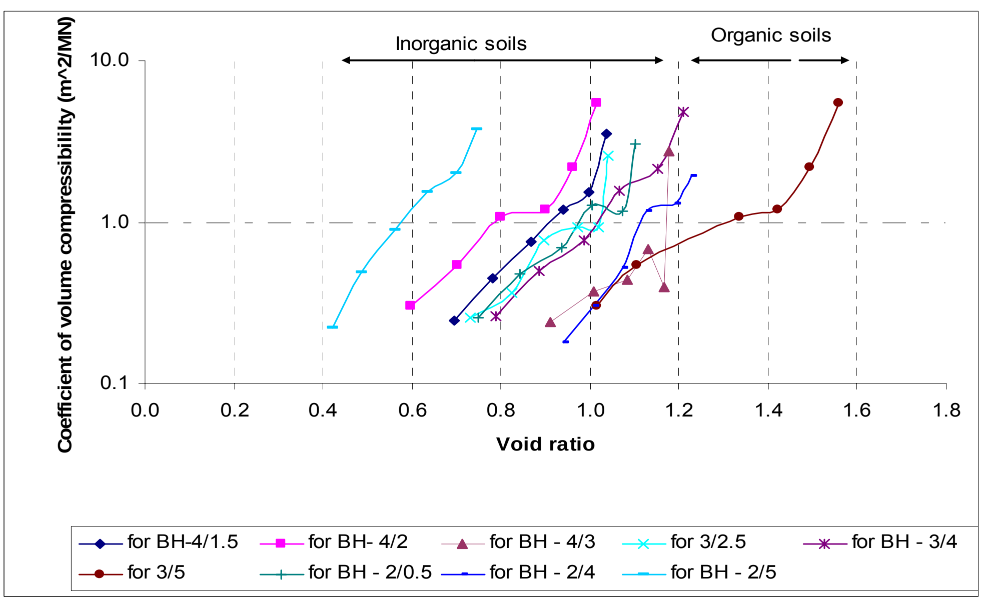


Figure 8.28: A plot of Coefficient of Volume Compressibility vs Void ratio for soil samples of BH – 02 to 04.

8.8.3 Compression index:

The value of compression index (Cc) for the organic and inorganic soil samples are given in Table 8.3. Table shows that the value of Cc for organic and inorganic soils ranges from 0.27 to 0.38 and 0.27 to 0.39 respectively. This compares well with usual range of Cc value that remains in the range of 0.20 to 0.50 for normally consolidated clay (Sing, 1992).

Various correlations are available between Cc and index properties of soils for both inorganic and organic soils noted in chapter four. In this study, some relationships are developed among Cc and initial void ratio, liquid limit, moisture content of the soil samples for both organic and inorganic soils. The relation between compression index (C_c) and initial void ratio for remoulded soil samples is for inorganic soils; $C_c = 0.23\ (e_o +0.36)$

And organic soils $C_c = 0.195\ (e_o +0.5)$

However, different researchers have established correlation between compression index (C_c) and initial void ratio (e_o) as noted in equation number 4.2.12, 4.2.13, 4.2.16 4.2.17, and 4.2.26 of chapter four. These equations are plotted in Figure 8.29 for making comparisons. It is observed that the test results show slightly higher compression index than Serajuddin & Ahmed, Azzouz et al. and Hough's equation but lower than Nishidi's equation.

Besides this, another correlation between compression index (Cc) and liquid limit (LL) has been established and presented in equations

For inorganic soil samples, $Cc = 0.01(LL – 10)$

For organic soil samples, $Cc = 0.0014(LL +174)$

The above mentioned equations are compared with the empirical equations suggested by Serajuddin & Ahmed (1967), Skempton (1944) and Terzaghi & Peck (1967) noted in equation number 4.2.11, 4.2.14 and 4.2.15 of chapter four respectively is shown in Figure 8.30. Figure shows that the equation for test results is close to Terzaghi & Peck's equation but slightly higher than Skempton and Serajuddin & Ahmed's equations.

Another correlation has been found between compression index (Cc) and moisture content (w) for both organic and inorganic soils and these relationships are as follows:

For inorganic soils, Cc = 0.006 (w+12)

For organic soils, Cc = 0.004 (w+22.5)

Meanwhile, the above mentioned equations have been compared with the Azouz et al. (1976) equations shown in Figure 8.31 that are noted equation number 4.2.25 and 4.2.27 of chapter four for organic peat soils and Chicago clay respectively. Figure shows that test results for both organic and inorganic soils are well matched with Azouz et al. (1976) equation but gives slightly lower value. Because for organic soils, the value of compression index depends on the amount of organic matter present in the soils, its mineralogy and particle size distribution in the soil samples. So these properties may not be similar to Bangladesh soils and Chicago soils, so the equations for compression index relating to moisture content are not be identical.

Table 8.3: Consolidation Parameters of soils collected from boreholes no. 2, 3 & 4.

BH No	Sample identification no.	Initial void ratio, eo	Compression Index, Cc	Recompression index, Cr	Swell index, Cs	Secondary compression index, C_α
	BH - 02/C6200/D - 0.5	1.19	0.391	0.135	0.0614	0.0037
	BH - 02/C6200/D - 4.0	1.28	0.304	0.092	0.059	0.043
	BH - 02/C6200/D - 5.0	0.83	0.27	0.0873	0.0573	0.0022
	BH - 03/C6480/D - 2.5	1.11	0.33	0.085	0.043	0.05
	BH - 03/C6480/D - 4.0	1.35	0.376	0.132	0.078	0.002
	BH - 03/C6480/D - 5.0	0.94	0.27	0.069	0.034	0.049
	BH - 04/C6600/D - 1.5	1.13	0.311	0.109	0.074	0.0031
	BH - 02/C6600/D - 2.0	1.16	0.37	0.116	0.076	0.0038
	BH - 04/C6600/D - 3.0	1.255	0.38	0.091	0.101	0.061

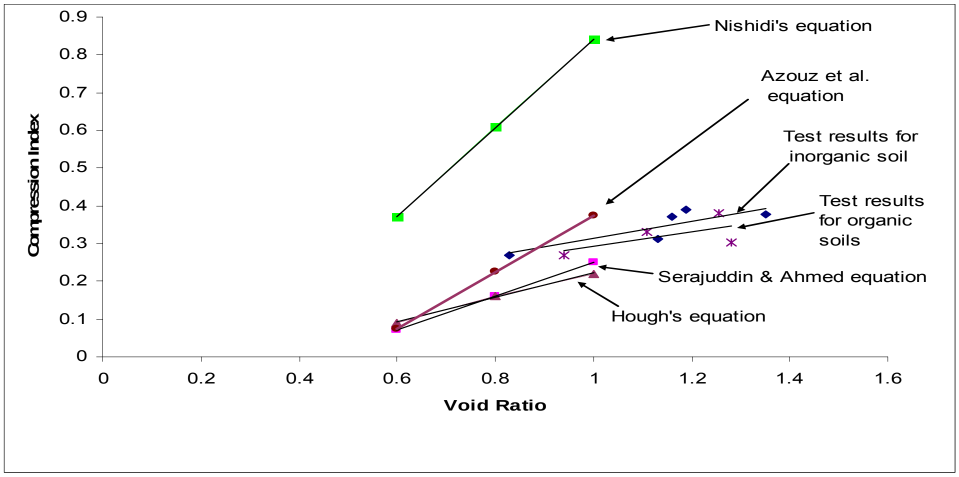


Figure 8.29: Relationship between Compression index and Initial void ratio for organic and inorganic soils.

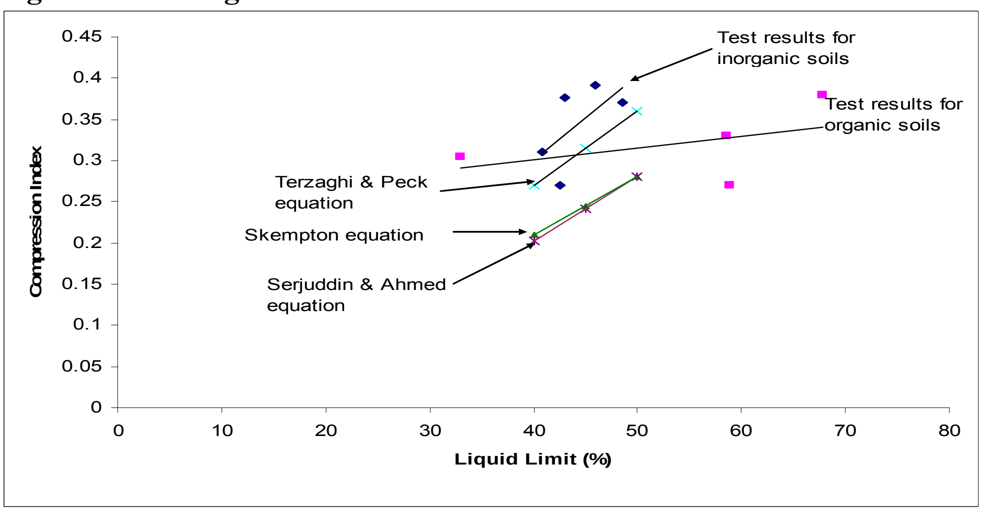


Figure 8.30: Relationship between Compression index and Liquid limit for organic and inorganic soil

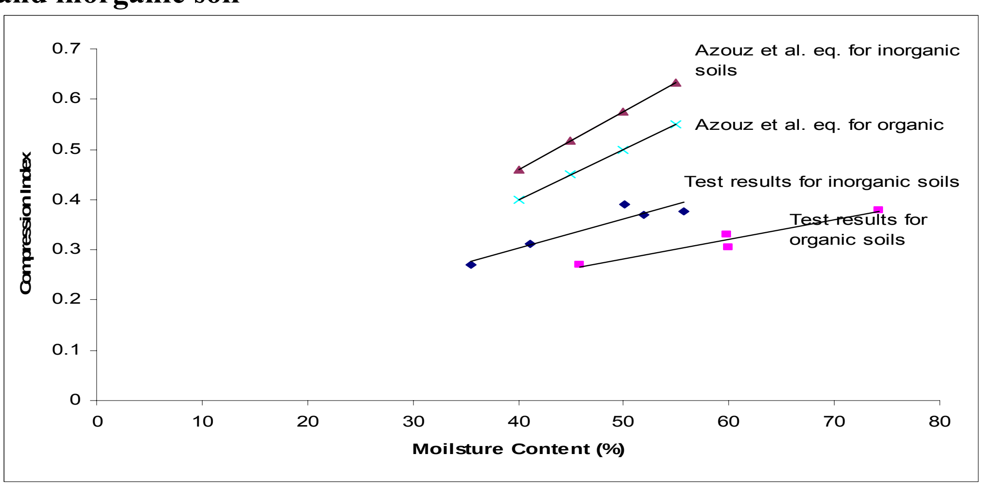


Figure 8.31: Relationship between Compression index and Moisture content for organic and inorganic soil.

8.8.4 Recompression index:

The value of recompression index (Cr) for both organic and inorganic remoulded soils is also given in Table 8.3. Results show that the value of Cr ranges from 0.0873 to 0.135 and 0.069 to 0.092 for inorganic and organic soils respectively. The recompression index value is correlated to some index properties of soils. The relationship between Cr and void ratio for inorganic and organic soils are expressed in the following equations:

For inorganic soils, $Cr = 0.093(eo + 0.115)$

For organic soils, $Cr = 0.07(eo + 0.12)$

The above mentioned equations are compared with the Azouz et al. (1976) equation noted in equation number 4.2.36 at chapter four is shown in Figure 8.32. It is seen that the derived equation is similar to the Azouz et al. equation but shows slightly lower Cr value. Because different factors like soil mineral composition, void ratio, and moisture content affect on recompression index of soils which varies for Bangladesh soils and Chicago soils.

Besides these, another relationship is found between Cr and liquid limit of both organic and inorganic soils that are expressed in the following equations

For inorganic soils, $Cr = 0.0022 (w_L + 8)$

For organic soils, $Cr = 0.0002 (478 - w_L)$

The above mentioned derived equations are compared with the Azouz et al. (1976) equation and Balasubramaniam & Brenner (1981) equation (noted in equation numbers 4.2.38 and 4.2.40 of chapter four) shown in Figure 8.33. However, the test results show the higher and lower recompression index value compared to the Azouz et al. equation and Balasubramaniam equation respectively.

Again, the relationship between Cr and moisture content (w) of inorganic and organic soil samples are expressed in equations

For inorganic soils, $Cr = 0.002 (w + 9)$

For organic soils, $Cr = 0.008 (w + 47.5)$

The above mentioned derived equations are compared with the Azouz et al. (1976) equation and Balasubramaniam & Brenner (1981) equation (noted in equation numbers 4.2.37 and 4.2.39 of chapter four) shown in Figure 8.34. It is seen that the derived equations for both organic and inorganic soils shows slightly lower value compared to the Azouz et al. equation and Balasubramaniam & Brenner equation.

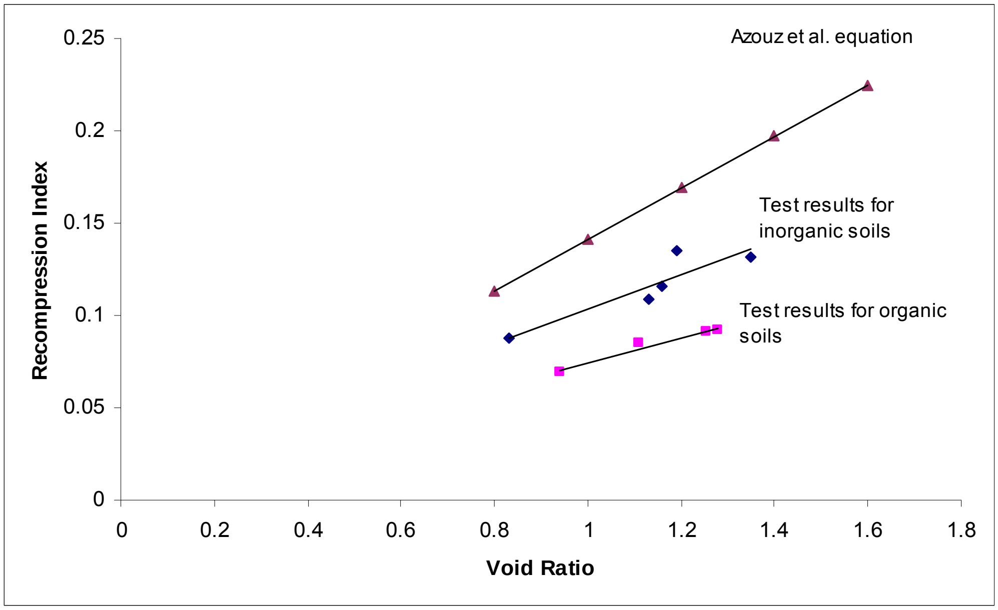


Figure 8.32: A plot of Recompression index vs Void ratio for inorganic and organic soils.

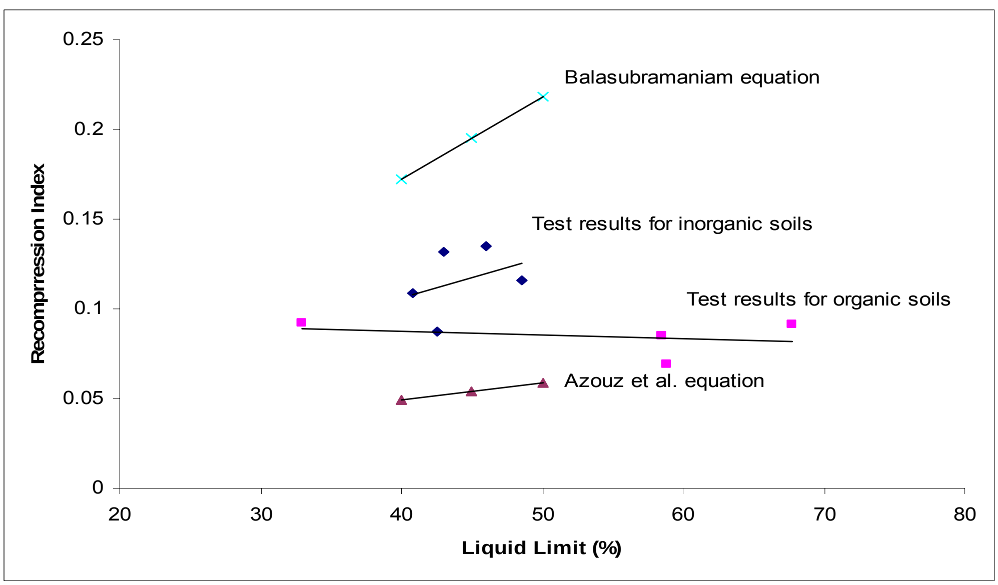


Figure 8.33: A plot of Recompression index vs Liquid Limit for inorganic and organic soils

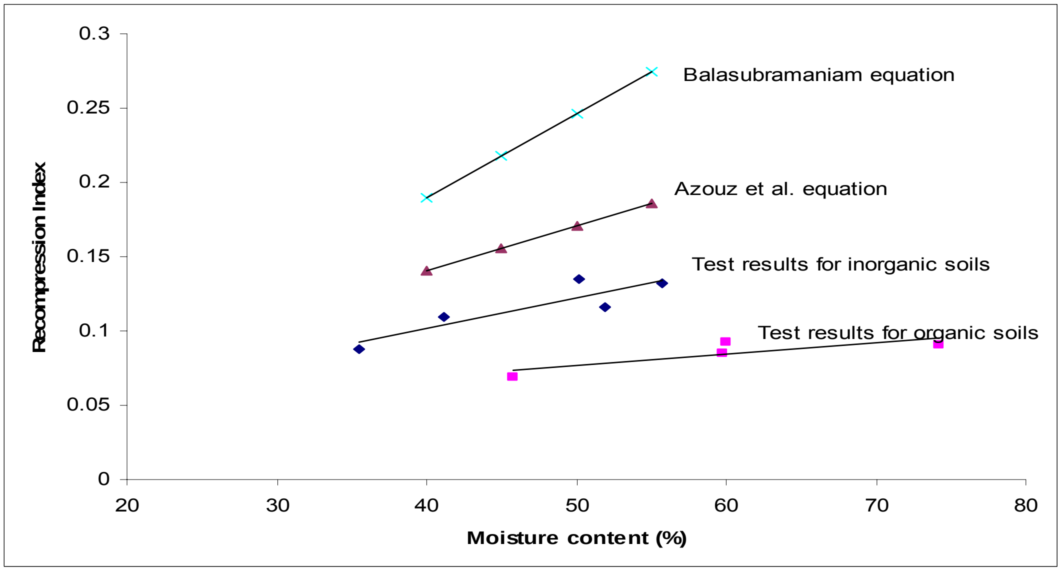


Figure 8.34: A plot of Recompression index vs Moisture content for inorganic and organic soils

8.8.5 Secondary Compression index

The secondary compression index (Cα) was determined for nine soil samples in which four are organic soils and five are inorganic ones. The values are already shown in Table 8.3. It is seen that the range of secondary compression index varies from 0.002 to 0.0038 and 0.043 to 0.061for inorganic and organic soils respectively. However, Lame & Whitman (1979) and Cernica (1995) have suggested some typical values of $C\alpha$ are noted in chapter three of Table 3.6. The test results are compared with the suggest values of above mentioned researchers and it is observed that the test results are within the mentioned limit.

Moreover, the value of $C\alpha$ has been correlated with Cc for both organic and inorganic soils are shown in Figure 8.35 and the correlations are presented in the following equations:

for inorganic soils, $C\alpha = 0.0074\ (Cc + 0.054)$

for organic soils, $C\alpha = 0.13\ (Cc + 0.077)$

The figure shows that derived equation for inorganic soils is very close to the Terzaghi et al. (1996) equations noted in equation number 4.2.31 of chapter four. On the other hand, the equation for organic soils is also compared with the Mesri & Godlewski's (1977) figure and Mesri's (1986) equation noted in figure 4.6 and equation number 4.2.33 respectively of chapter four. Results show that the value of $C\alpha$ is higher than the other correlations. This is because the value of Cα depends on the amount of organic matter, types of the organic matter, particle size distribution among the samples soil mineral composition, void ratio and moisture content in the soil samples. So these parameters are not similar to the Bangladesh soils and other places of soils in which the empirical correlations were derived.

8.8.6 Swelling index

The value of swelling index for both inorganic and organic soils is already given in Table 8.3. Results show that the value ranges from 0.0573 to 0.078 and 0.034 to 0.101for inorganic and organic soils respectively. Relationship between swelling index and compression index has been established for both inorganic and organic soils are given in the following equations.

For inorganic soils, $Cc = 2.33 (Cs + 0.078)$

For organic soils, $Cc = 1.36 (Cs + 0.18)$

The above mentioned correlation for inorganic soils has been compared with the Nagaraj & Murthy (1985) equation noted in equation number 4.2.29 of chapter four is shown in Figure 8.36. It is seen that the most of the organic soils are within the specified limits but those for inorganic soils are not within that limits.

8.8.7 Coefficient of Permeability

The coefficient of permeability (K) has determined on the basis of Cv and Mv for both organic and inorganic soil samples. The values of k are given in Appendix G, Table A - 8.2. Results show that the k value ranges from 4.42×10^{-11} m /s to 1.2×10^{-9} m /s and 1.17×10^{-10} m /s to 3.42×10^{-8} m/s for inorganic and organic soils respectively. The observed k values are compared with the usual range of k values for Bangladesh soil types reported by Aminullah (2004) noted in article 3.6.6.1 of chapter three are almost similar. It may be noted that the inorganic soils are practically impervious and organic soils are very low permeable to impervious. However, the variation of permeability with void ratio is given in Figure 8.37. Results show that the organic soils are very distinct as they have higher void ratio and higher permeability. Meanwhile for inorganic soils have low void ratio and the variation of k does not change remarkable with the change in void ratio. On the other hand, for organic soils k has changed remarkable with the change in void ratio. Because the organic soils have high void ratio as a result after applying the pressure on it, water can move rapidly in organic soils than inorganic one. For this reason, the coefficient of permeability is higher in organic soils.

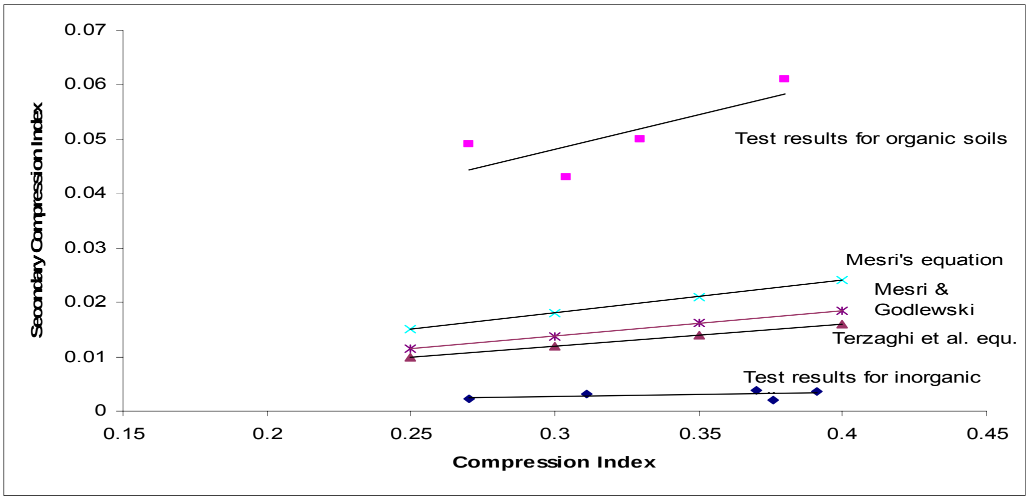


Figure 8.35: A plot of Secondary compression index vs Compression index for inorganic and organic soil.

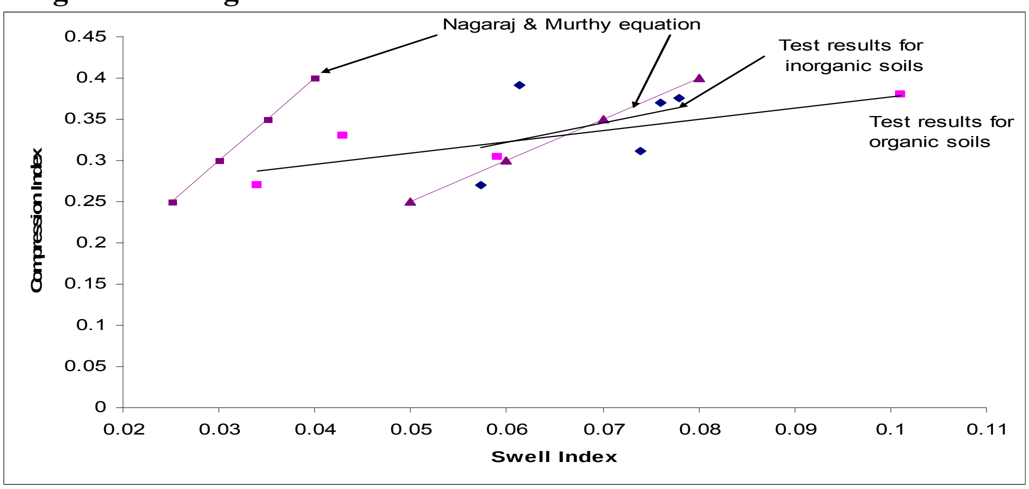


Figure 8.36: A plot of Compression index vs Swell index for inorganic and organic soils.

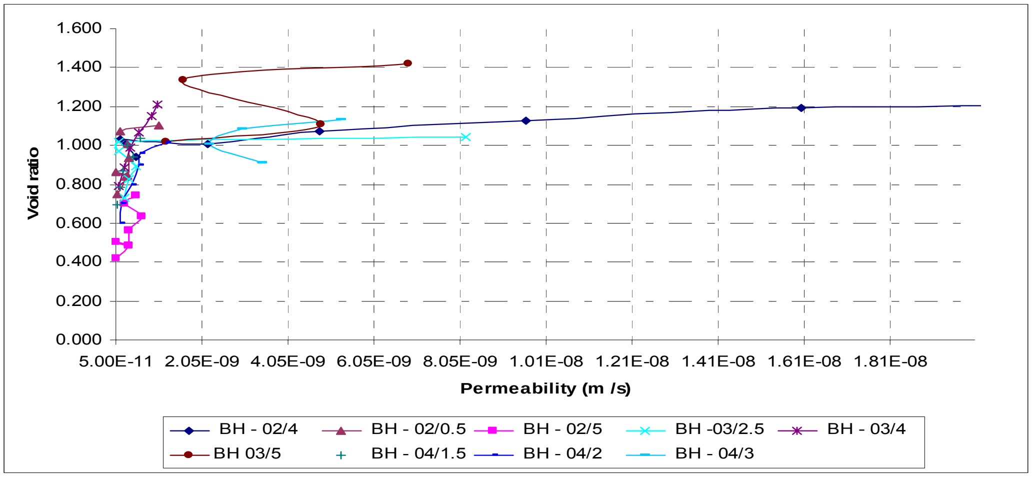


Figure 8.37: A plot of Secondary compression index (Cα) vs Compression index (Cc) for inorganic and organic soils

Table 8.4: Summary of testing methods and evaluated design input parameters

Name of input parameter for design	**Name of the test**	**Appropriate standard (BS /ASTM/ others)**	**Number of tests have been done**	**Test Results used as design input parameters (upper & lower range of values)**
Liquid limit (LL), Plastic Limit (PL),	Atterberg limit test with Casagrande's apparatus (lab. Test)	ASTM D 4318 – 84	36 Numbers	w_L = 29% to 67.8% I_P = 16.8 % to 55.83%
Moisture content (ω)	Moisture content test. (lab test)	BS 1377-2:1990	36 Numbers	For inorganic soils ω = 25% to 60% For organic soils ω = 80% to 260%
Organic content test	Organic content determination test (lab test)	ASTM : D 2974 - 87	14 Numbers	2.27% to 30.47%
Density(γ), dry density (γ_d)	Proctor test	BS1377-4:1990	36 Numbers	γ = 11.42 to 19.9 kN/m3 γ_d = 3.2 to 15.8 kN/m3
Specific gravity (Gs) of soil	Specific gravity test (lab test)	BS 1377 – 2:1990, Clause 8	36 Numbers	For inorganic soils Gs = 2.55 to 2.7 For organic soils Gs= 2.18 to 2.42
Initial void ratio (e_o)	1. Relationship among unit wt, void ratio, moisture content & specific gravity.	BS1377-5:1990	36 Numbers	Inorganic soils eo = 0.5 to 1.7 For organic soils eo = 1.9 to 5.8
Particle size distribution (PSD)	Particle size distribution test (sieve analysis & Hydrometer analysis)	BS1377-2:1990, Clause 9	15 Numbers	Sand = 4 to 19% Silt = 29 to 50% Clay = 32% to 49%

Undrained shear strength (Cu)	1. UCS test	1. BS1377-7:1990	36 Numbers	5 to 70 kPa
Undrained modulus (Eu)	1.Triaxial test	BS1377-7:1990	9 Numbers (remoulded sample)	1.88 to 3.6 MPa
Undrained cohesion & angle of internal friction (ϕ)	1.Triaxial test (UU)	BS 1377-7:1990	9 Numbers (remoulded sample)	Cu = 18 to 28 kPa ϕ = 2.3 to 10.3 degree
Peak & Residual Drained Cohesion (C') and angle of internal friction (ϕ ')	1.Direct shear box test 2.Ring shear test	1.BS1377-7:1990, Sec -4 2.BS1377-7:1990, Sec -6	9 Numbers (remoulded sample)	C' = 2.96 to 6.53 kPa ϕ ' = 12.85 to 24.4 kPa Cr' = 0.11 to 2.43 kPa ϕ r' = 4.68 to 7.87 degree

Cont…

Name of input parameter for design	Name of the test	Appropriate standard (BS /ASTM/ others)	Number of tests have been done	Test Results used as design input parameters (upper & lower range of values)
Compression index (Cc)	1.Consolidation test 2.Correlations	BS1377-5:1990	9 Numbers (remoulded sample)	Cc = 0.27 to 0.39
Secondary compression index (C α)	1.Consolidation test 2.Correlations	BS1377-5:1990	9 Numbers (remoulded sample)	Inorganic soils C α = 0.002 to 0.0038 For organic soils C α = 0.043 to 0.061
Coefficient of consolidation (C_v)	1.Consolidation test	BS1377-5:1990	9 Numbers (remoulded sample)	Inorganic soils Cv = 0.421 to 2.25 m^2/yr For organic soils Cv = 21.95 to 39.63 m^2/yr
Coefficient of volume compressibility (M_v)	1.Consolidation test	BS1377-5:1990	9 Numbers (remoulded sample)	Inorganic soils Mv = 0.044 to 5.45 m^2/MN For organic soils Mv = 0.04 to 4.41 m^2/ MN
Recompression index (Cr)	1.Consolidation test	BS1377-5:1990	9 Numbers (remoulded sample)	Inorganic soils Cr = 0.0873 to 0.135 For organic soils Cr = 0.069 to 0.092
Swell index (C_s)	1.Consolidation test	BS1377-5:1990	9 Numbers (remoulded sample)	Inorganic soils Cs = 0.0573 to 0.078 For organic soils Cs = 0.034 to 0.101
Permeability (kv)	1.Consolidation test	BS1377-5:1990	9 Numbers (remoulded sample)	Inorganic soils k = 4.42E-11 to 1.2E-9 m/s For organic soils k = 1.17E-10 to 3.42E-8 m/s.

Remarks: These results are derived for limited number of soil samples.

9.1 Summary

All possible laboratory tests were conducted on the soil samples collected from four boreholes of one road embankment. Both organic and inorganic soils were found. The details index properties, shear strength properties and compressibility properties are evaluated. All of the data have been analyzed and numerous empirical correlations have also been developed. Though these correlations have developed for limited numbers of data, those correlations can be used for calculations of engineering properties of soils for Bangladesh. Besides these, the developed correlations are also compared with those suggested by different researchers for different kinds of soils. In most of the cases, correlations suggested by this study are nearly similar to the established correlations. The lower and upper limits of the evaluated properties of soils that will act as a database and used in case study for design of embankment are also given in Table 8.4.

Chapter 9: Test Results and Discussions for Subgrade & Improved Subgrade Materials

9.1 General

The field and laboratory test results and their analysis for subgrade soils and improved subgrade materials are discussed in this chapter. Since, the design of pavement is primarily depends on the strength and stiffness parameters of subgrade layer, so these properties are evaluated by conducting the field & laboratory testing's on N4, N302, R301 and Z3024 roads. The analytical pavement design needs the resilient modulus of subgrade soils as well as improved subgrade materials (sand) which can be assessed by cyclic triaxial test. The determination of resilient modulus of subgrade soils and sand are discussed in this chapter. Besides these, the effects of mica content on resilient modulus of sand have also been discussed.

9.2 Subgrade soils

Based on the test results undertaken on the above mentioned roads, the subgrade soils in Bangladesh are found silty clay, sandy silty clay and silty sand. Their classifications, index properties, strength and stiffness properties are discussed in the followed subsections.

9.2.1 Index properties of subgrade soils

Atterberg limits of subgrade soils collected from three roads were evaluated by respective laboratory tests. The position of soil samples are plotted in the Casagrande's plasticity chart that is shown in Figure 9.1. Figure shows that the subgrade soils of Road number R 301and N4 are inorganic clay of intermediate plasticity while this for Z 3024 is low plasticity. As per RMSS subgrade soil classification system noted in Figure 4.11, this subgrade of low to intermediate plasticity indicates that the subgrade soils have low CBR value and suggested to stabilize them with lime. Moreover, using the empirical correlation noted in equation 4.3.2, the subgrade insitu CBR values was estimated for Z3024, R301 and N4 were given in Table 9.1. Meanwhile the field moisture content found for N4, R301 and Z3024 roads are ranged from 14.9% to 15.3%, 14.6% to 15.8% and 15.7% to 16.9% respectively.

9.2.2 Particle Size Distribution of subgrade soils

The gradation envelopes determined by the particle size distributions test for subgrade soils are shown in Figure 9.2 (a) to 9.2(c). From the gradation envelope of N4 road, observed that about 90% to 100% soil particles are smaller than 0.3 mm. All of the soil particles are smaller than 0.6 mm.

From the gradation curve of subgrade soils of R301 road observed that the maximum sizes of soil particles are 0.3 mm. The value of D_{60} and D_{30} of the soil samples are 0.09 to 0.12 and 0.025 mm respectively. On the other hand, the mean size of the particles, [16]D_{50} varies from 0.034 mm to 0.049 mm. The gradation envelope of soil samples for Z3024 lies in between 30% to 45%, 45% to 55% and 55% to 60% passing through 0.025mm, 0.05mm and 0.10mm respectively. The value of effective size like D_{60}, D_{30} and mean size D_{50} of the soil particles varies from 0.1 to 0.12mm, 0.025mm and 0.034 mm to 0.071 mm respectively.

[16] D_{50} = particle diameter at which 50% is finer

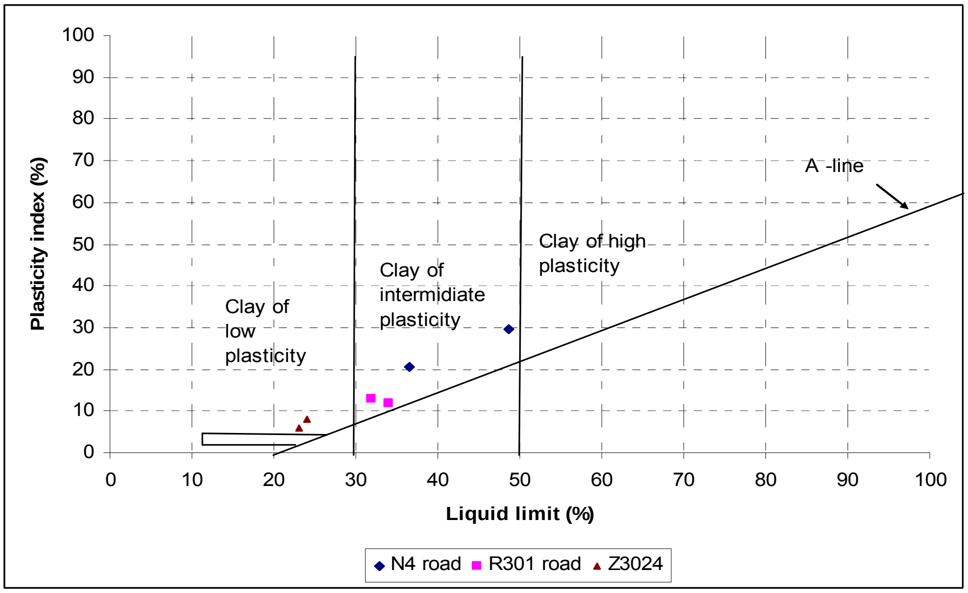
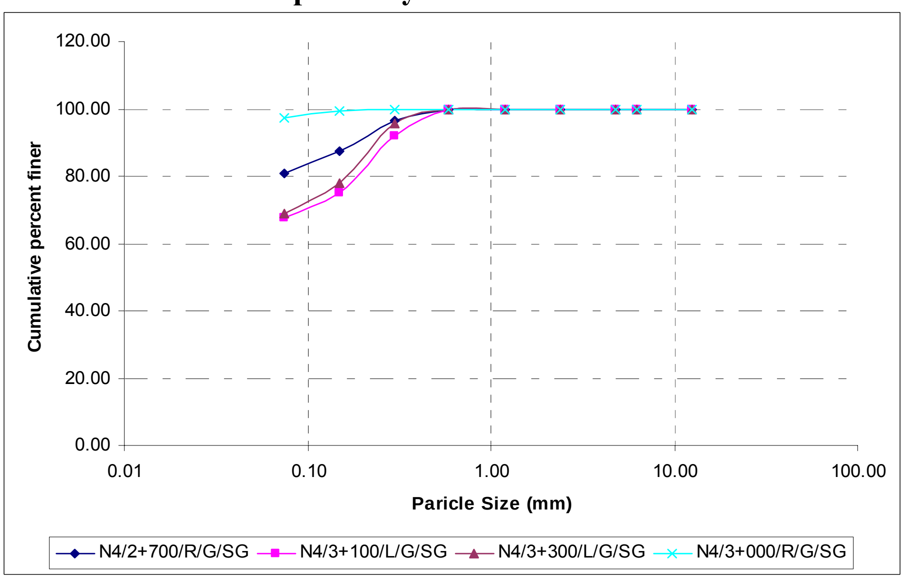
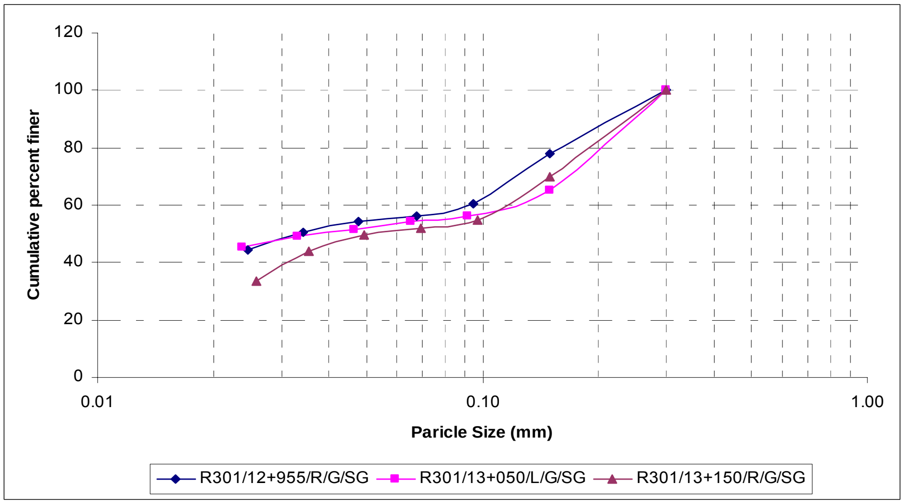


Figure 9.1: A plot of Plasticity index vs Liquid limit of soil samples and location of soils in the plasticity chart.





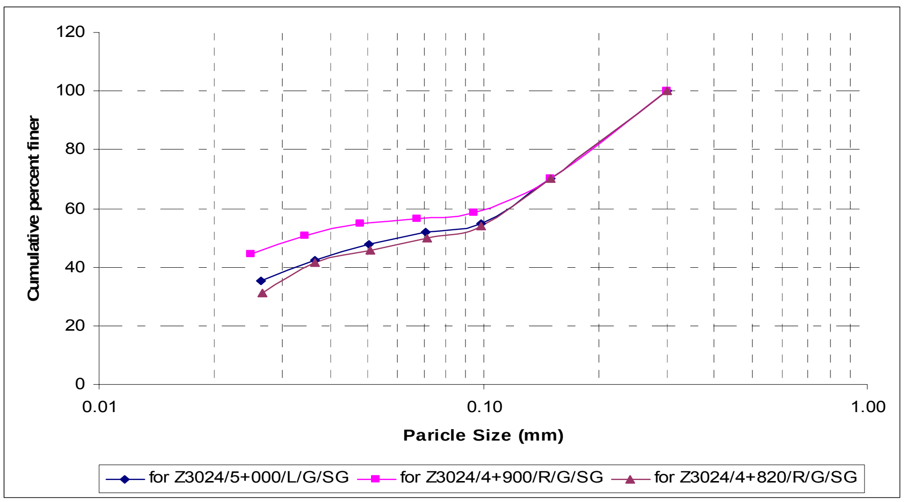


Figure 9.2 (a), (b) & (c): Particle Size Distribution chart for subgrade soil samples of three roads

9.2.3 Relative Compaction

The relative compaction of the subgrade layer for N4, R301 and Z3024 is found 82.4% to 83.9%, 89.1% to 91.6% and 83.6% to 86.7% of MDD achieved by BS light compaction respectively. But Overseas Road Note 31 recommended that subgrade should be compacted during construction to a relative compaction of at least 100% of maximum dry density achieved in the British Standard (light) Compaction (standard Proctor test) or at least 93% of the maximum dry density achieved in the British Standard (Heavy) Compaction (modified Proctor test). On the other hand, RHD specifications suggested that the relative compaction of subgrade soils should be 98% of MDD by BS light compaction.

So, the value of relative compaction found for all roads are however, lower than the both specifications which effects on the insitu CBR value of the subgrade soils. The details subgrade soils properties are shown in Table 9.1.

Table 9.1: Subgrade soil properties

Road Name	Sample number	Soaked CBR (%)	Insitu CBR (%), using equation no.4.3.2	Maximum dry density (MDD), kg / m3	Optimum moisture content (OMC), %	Plasticity index (PI),%	Field dry density (FDD), kg/ m3	Relative density, %
N4	N4/2+700/R/G/SG	2.52	7.73	1870.00	11.01	29.78	1541.0	82.4
	N4/3+100/L/G/SG	0.26	6.81	1889.00	11.13	20.39	1585.0	83.9
R301	R301/12+955/R/G/SG	5.23	3.84	1669.18	16.3	13.00	1528.8	91.6
	R301/13+050/L/G/SG	2.35	2.81	1665.98	19.3	12.00	1484.5	89.1
Z3024	Z3024/4+820/R/G/SG	1.26	4.34	1818.16	14.00	8.00	1575.8	86.7
	Z3024/5+000/L/G/SG	1.42	4.51	1848.59	13.53	6.00	1545.8	83.6

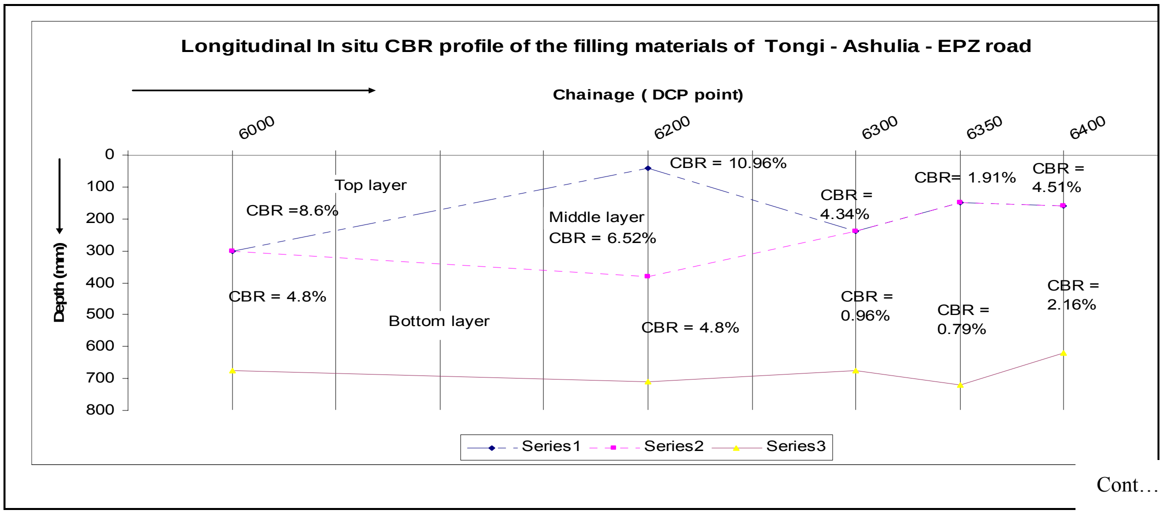
9.2.4 Insitu CBR value

The insitu CBR value is measured by Dynamic Cone Penetrometer (DCP) test conducted on the above mentioned roads. A typical DCP test results and hence calculation of DCP value as well as layer thickness procedure is given in Appendix H Figure B- 9.1. Thus insitu CBR value for N4, N302, R301 and Z3024 roads is calculated from DCP values using different empirical equations suggested by different researchers are shown in Appendix G, Table A- 9.1. These DCP values and hence calculated CBR values of the different layers are presented in the above mentioned table. It is seen that CBR value found from different empirical correlations and charts are different. As discussed in article 4.3.1 of chapter four, CBR value is calculated using Riley et al. (1987) equation and hence reported. From the table, it is seen different roads have different subgrade soils that shows range of insitu CBR value from 3.4% to 3.62%, 6.43% to 7.92%, 5% to 22.27% and 3.81% to 9.67% for N4, N302, R301 and Z3024 road respectively. The above mentioned insitu CBR value will change due to the seasonal variation of moisture content. The worst condition was occurred during the monsoon when the roads are fully inundated in Bangladesh. So the laboratory soaked CBR value is considered for design of new pavement that is discussed in the following section.

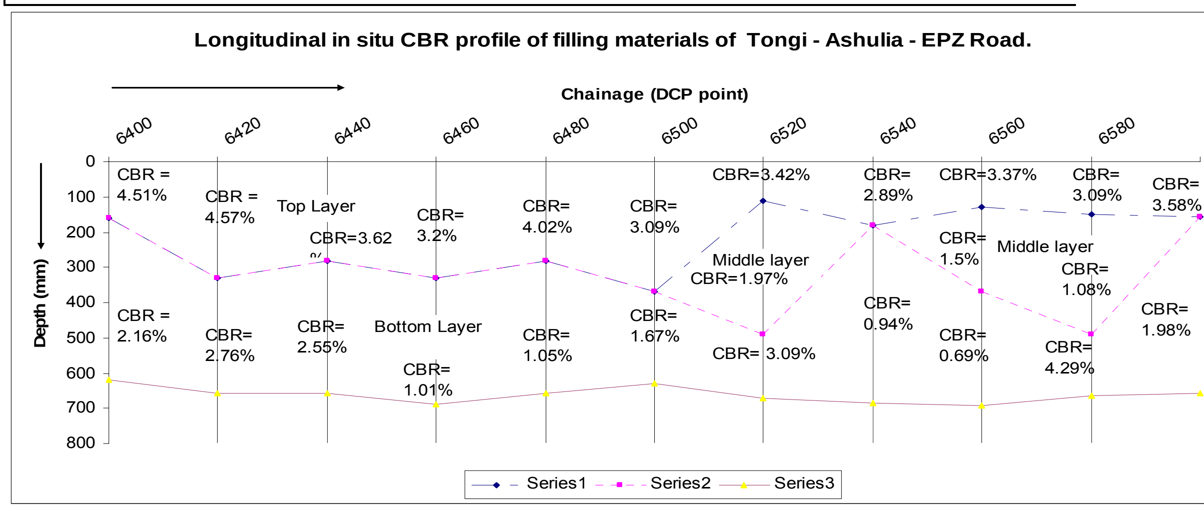
Moreover, an intensive evaluation was done on N302 road from top of the earthen shoulder which provides the strength value of subgrade as well as fill materials of embankment. Their strength is determined by DCP test and longitudinal profile of soils strength measured by CBR value is shown in the following Figure 9.3. From these longitudinal profiles of subgrade soils observed two types of soils are identified but in some chainage another type of soils in between them is also identified. The insitu CBR values of top and bottom layer ranged from 1.91% to 10.96% and 0.61% to 4.8% respectively. These different values of CBR indicate that the top soils are better quality in terms of strength or it is compacted properly than the bottom ones.

9.2.5 Laboratory soaked CBR value

Insitu CBR value depends on the subgrade soil types, moisture content and compaction of soils. So the above mentioned insitu CBR value has been changed due to the seasonal variation of moisture. The worst case scenario have occurred in Bangladesh during the monsoon when road is inundated by flood. For this reason, laboratory four days soaked CBR value recommended by ASTM D 1883 – 05 was done on subgrade soils. The laboratory four days soaked CBR values already shown in Table 9.1 ranges from 0.26% to 2.52%, 2.51% to 3.28%, 2.35% to 5.23% and 1.42% to 1.26% for N4, N302, R301 and Z3024 road respectively. So it is evident that soaked CBR value depends on the types of the subgrade soils. However, Overseas Road Note 31(RN 31) suggested that if the subgrade CBR value is less than 2%, special treatment is required and if it is greater than 2%, it may be used for design. But as per RHD Standard, if the soak CBR value is less than 5%, special treatment is suggested. So considering the above mentioned specifications, special treatment have to be provided for subgrade soils of Z3024 and N4 road to increase its strength or by providing additional sand blanket layer known as improved subgrade layer or stabilizing with lime.


Longitudinal In situ CBR profile of the filling materials of Tongi - Ashulia - EPZ road
Chainage (DCP point)
6000
6200
6300
6350
6400
Depth (mm)
0
100
200
300
400
500
600
700
800
CBR = 10.96%
Top layer
CBR =8.6%
Middle layer
CBR = 6.52%
CBR = 4.34%
CBR= 1.91%
CBR = 4.51%
CBR = 4.8%
Bottom layer
CBR = 4.8%
CBR = 0.96%
CBR = 0.79%
CBR = 2.16%
Series1
Series2
Series3

Cont…


Longitudinal in situ CBR profile of filling materials of Tongi - Ashulia - EPZ Road.
Chainage (DCP point)
6400
6420
6440
6460
6480
6500
6520
6540
6560
6580
Depth (mm)
0
100
200
300
400
500
600
700
800
CBR = 4.51%
CBR = 4.57%
Top Layer
CBR=3.62
CBR= 3.2%
CBR= 4.02%
CBR= 3.09%
CBR=3.42%
CBR= 2.89%
CBR=3.37%
CBR= 3.09%
CBR= 3.58%
Middle layer
CBR=1.97%
CBR= 1.5%
Middle layer
CBR= 1.08%
CBR = 2.16%
CBR= 2.76%
CBR= 2.55%
Bottom Layer
CBR= 1.01%
CBR= 1.05%
CBR= 1.67%
CBR= 3.09%
CBR= 0.94%
CBR= 0.69%
CBR= 4.29%
CBR= 1.98%
Series1
Series2
Series3

Cont…

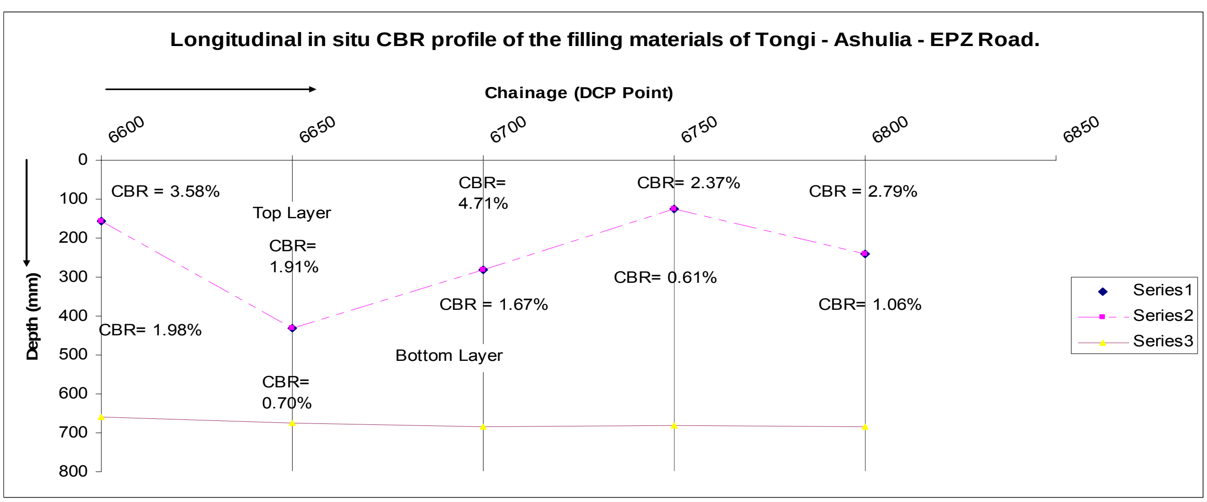

Longitudinal in situ CBR profile of the filling materials of Tongi - Ashulia - EPZ Road.
Chainage (DCP Point)
6600
6650
6700
6750
6800
6850
Depth (mm)
0
100
200
300
400
500
600
700
800
CBR = 3.58%
Top Layer
CBR= 1.91%
CBR= 4.71%
CBR= 2.37%
CBR = 2.79%
CBR= 0.61%
CBR = 1.67%
CBR= 1.06%
CBR= 1.98%
Bottom Layer
CBR= 0.70%
Series1
Series2
Series3

Figure 9.3: Longitudinal in situ CBR profile of the subgrade as well as filling soils of Tongi – Ashulia – EPZ (N4) Road.

9.2.6 Resilient Modulus of Subgrade soils

The resilient modulus of subgrade soils is determined by cyclic triaxial test. This test was done on four unsoaked subgrade soil samples (sample number one to four) and one soaked sample. The soils used in this test are silty clay collected from Bangladesh. The samples were prepared 100 mm diameter and 200 mm height with compacted to BS standard compaction (2.5 kg hammer). The moisture content of the subgrade soil samples were 28.55%, 27.50%, 25.93% and 27.64% for samples one to four. Meanwhile the moisture content of the soaked sample was found 35.26%.

At the beginning of the test, in accordance with AASHTO T307 test protocol, the sample was conditioned with 500 repetitions of load comprising cell pressure, cyclic and contract stresses of 41.4 kPa, 24.8 kPa and 2.8 kPa respectively. The soil samples number one to three and soaked sample are subjected to sequential load designed by AASHTO T 307. As it is known that the resilient modulus is the function of bulk stress, so its value varies with deviator stresses. However, at the end of the test, resilient modulus was found 22.815 MPa, 27.545 MPa, and 28.82 MPa for subgrade soil sample one to three respectively. The resilient modulus of subgrade soil sample subjected to four days soaking was found 20.0 MPa.

It is mentioned that soil sample number four is subjected to 10,000 cyclic loads representing the field loading condition which comprises 41.4 kPa confining stress, 55.2 kPa deviator stresses with 5.5 kPa contract stress. This user defined test condition gives the resilient modulus of subgrade soils is almost 27 MPa. The details of the test results are shown in Appendix- G Table A-9.2.

As the resilient modulus is the function of deviator stress, so the variation of resilient modulus with deviator stress is shown in Figure 9.4. Figure shows that the resilient modulus value decreases with increase in deviator stresses. It is also observed that soaked sample gives the lower resilient modulus value than the unsoaked samples. This variation pattern is compared with the figure 3.4 developed by Lee et al. (1997) noted in chapter three that shows the similar pattern of variation for cohesive soils.

On the other hand, the variation of deviator stress with resilient strain is shown in Figure 9.5 which is similar to the figure 3.5 developed by Lee et al. (1997). It is observed that percent resilient strain increases with increase in deviator stress. It is also noticed that the soaked sample gives higher [17]resilient strain compared to that of unsoaked samples for same deviator stress.

Again, the relationship between resilient modulus and resilient strain shows resilient modulus decreases with increase in resilient strain that is shown in Figure 9.6.

The relationship between cumulative strain and load repetitions is shown in Figure 9.7 which shows that 10% cumulative strain occurs at almost 800 repetitions of load and after that load repetitions; cumulative strain increase rapidly.

As discussed in article 3.7.2 of chapter three, keeping all other factors constant the resilient modulus is a stress dependent parameter (Seed et al. 1962; Monismith et al. 1967; Brown et al. 1975; Drumm et al. 1990). A simple expression relating to resilient modulus to the maximum deviator stress (σ_d) is commonly used

$M_R = k1\ (\sigma_d)^{k2}$

The value of constant k1 & k2 are determined from the relationship between resilient modulus and deviator stress shown in Table 9.2. So, the relationship between deviator stress and resilient modulus for subgrade soils (silty clay) of Bangladesh is

$M_R = 152.63\ (\sigma_d)^{-0.413}$

Where M_R is in MPa and σ_d is in kPa.

The total axial stress (sum of traffic stress and insitu stress) exerted on the subgrade (bottom of subbase layer) was found 10.35 kPa (details calculations are shown in Appendix F). Using the above mentioned relationship, the corresponding resilient modulus was found 58 MPa. So, for achieving this resilient modulus for sustainable pavement design in Bangladesh, an improved subgrade layer has to be provided on top of subgrade soil or stabilization of subgrade has to be done. Because resilient modulus (soaked) of the subgrade soils (silty clay) of Bangladesh was found 20 MPa.

[17] Resilient strain means recoverable strain

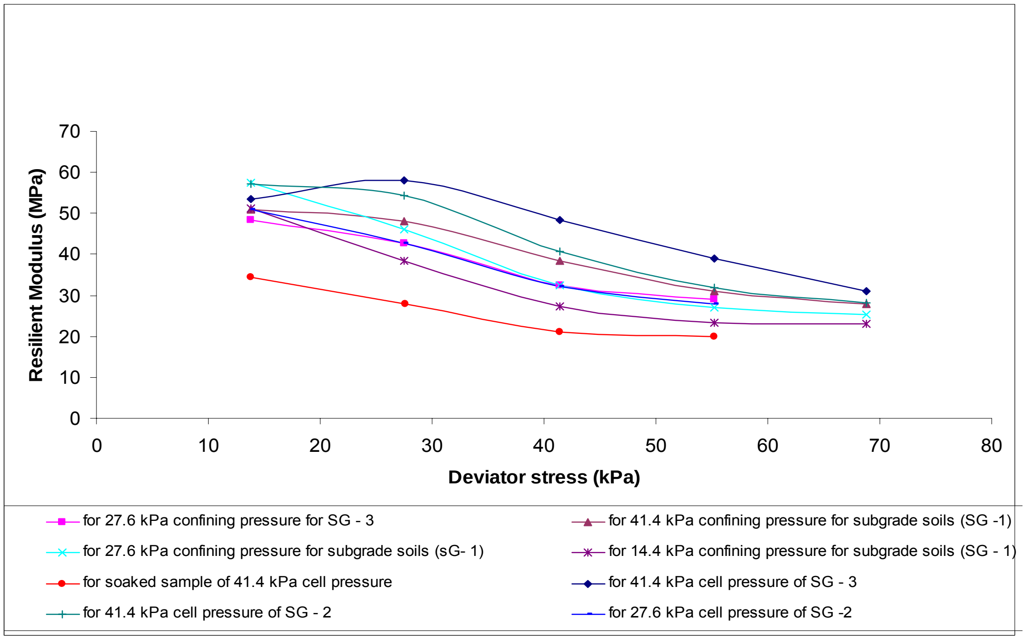


Figure 9.4: A Plot of variation of Resilient Modulus with Deviator stress for different confining pressure.

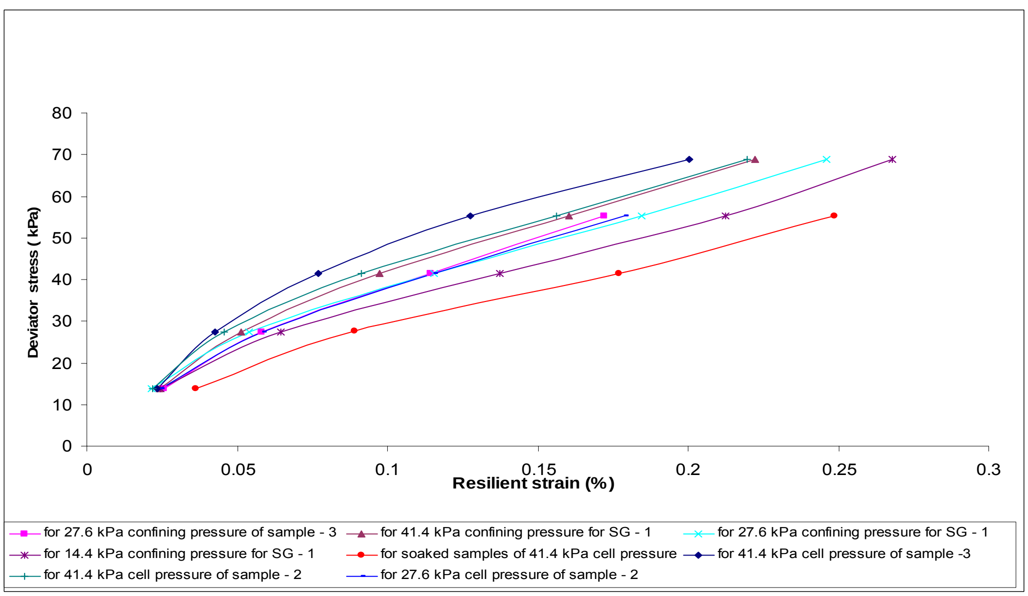


Figure 9.5: A Relationship between Deviator Stress and Resilient Strain for different confining pressure.

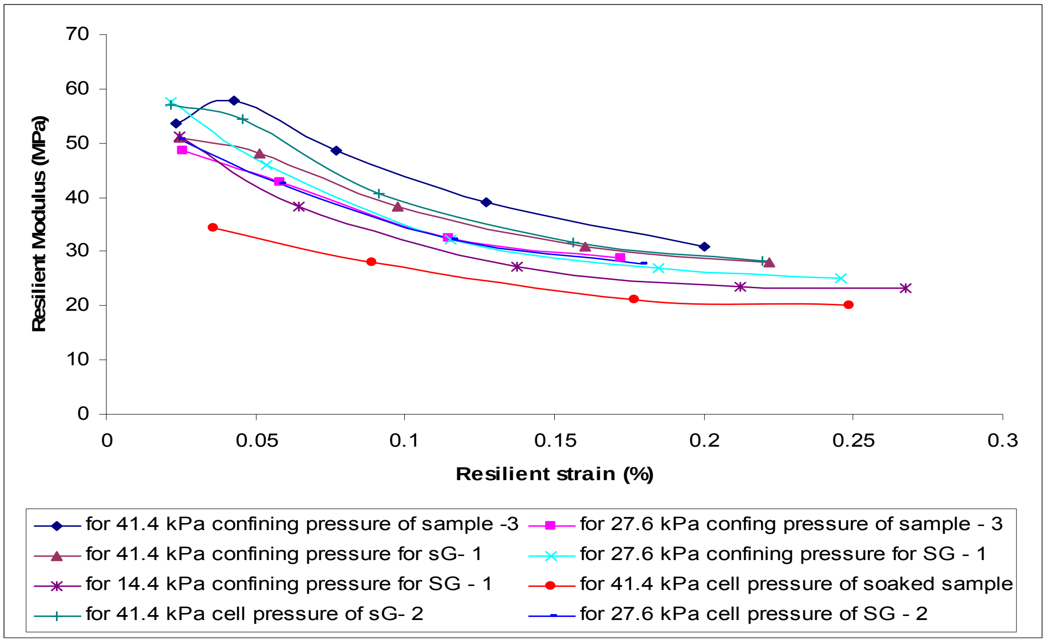


Figure 9.6: A Relationship between Resilient Modulus vs Resilient Strain for different confining pressure for four unsoaked & one soaked soil samples.

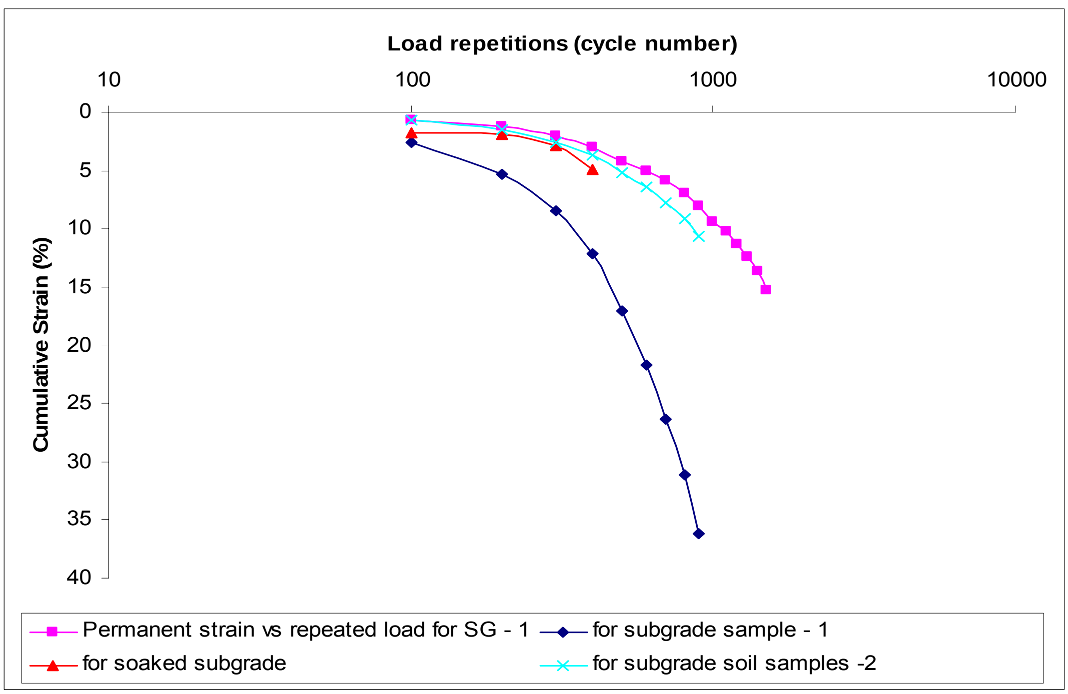


Figure 9.7: A Plot of Cumulative Strain vs Load repetitions for four unsoaked and one soaked soil samples.

Table 9.2: k- values for non – linear modeling for subgrade soils of Bangladesh ($M_R = k1\sigma_d^{k2}$)

Sample number	k1	k2	R^2
Unsoaked -1	202.96	-0.492	0.85
Unsoaked- 2	178.45	-0.4317	0.81
Unsoaked- 3	126.21	-0.3179	0.55
Soaked	102.91	-0.4114	0.96
Average	152.63	-0.413	

9.2.6.1 Relationship between Resilient Modulus and CBR value

The CBR values were determined for the similar subgrade samples of which cyclic triaxial test were done. The soaked CBR value was found 1.98% whereas resilient modulus of the similar sample was found 21.104 MPa. The relationship between CBR values and Resilient Modulus values for deviator stress of 41.4 kPa is given in Figure 9.8. In that figure, the relationship established by Powel et al. (1984) and Brown et al. (1990) between CBR and Resilient Modulus value was also shown for comparison. It is noticed from the figure that the test results have shown the relationship between these parameters is M_r = 10.27 x CBR which is almost similar to the Brown et al. (1990) equation (M_r = 10 CBR), widely used in design.

Again, this resilient modulus was compared with the Selig et al. (2003) chart noted in figure 3.3 of chapter three. It is seen from the chart that resilient modulus of subgrade soils lies in between 20 MPa to 60 MPa for subgrade soils of firm clay to stiff clay and corresponding soaked CBR values 2% to 5%. So, from the test results of subgrade soils of Bangladesh observed that soaked CBR values laid in between the limits (2% to 5%) and the resilient modulus of subgrade soils are laid in between 20 MPa to 58MPa which fairly satisfied the established relationship.

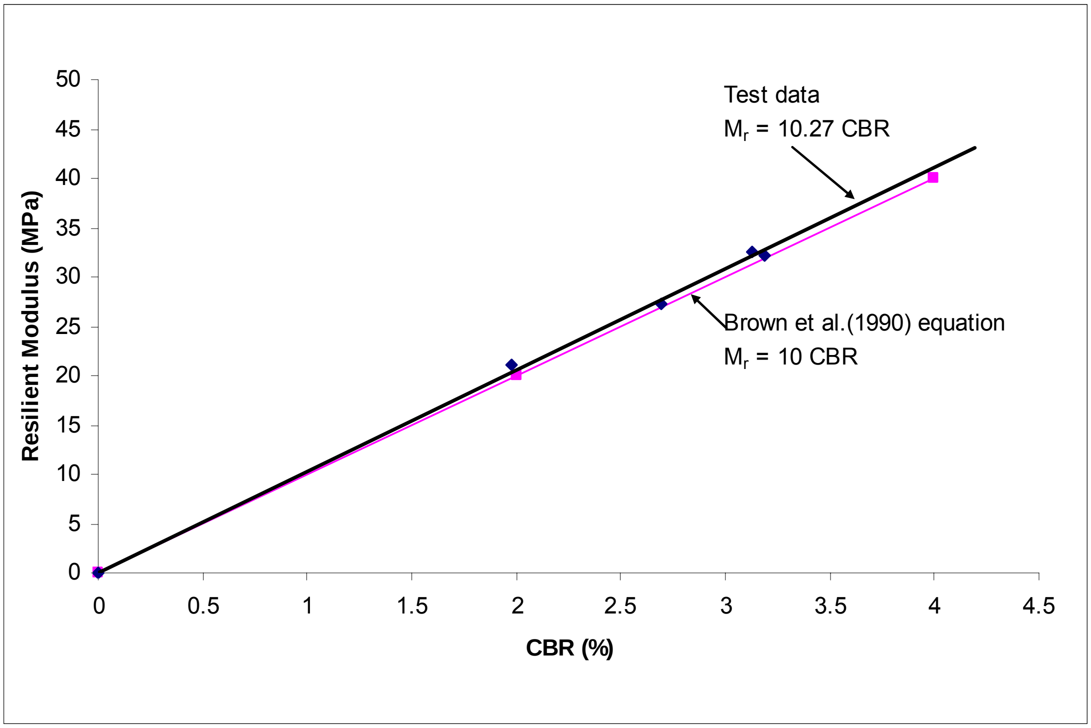


Figure 9.8: Relationship between Resilient Modulus and CBR value for subgrade soil (silty clay) samples for Bangladesh.

9.2.6.2 Relationship between Resilient modulus and DCP value

The DCP test was done on the subgrade soil samples at which cyclic triaxial test were performed. The relationship between resilient modulus and DCP value (mm / blow) is shown in Figure 9.9. The resilient modulus is considered for deviator stress of 41.4 kPa and confining pressure of 41.4 kPa. The test results are compared with the empirical correlation suggested by Chai & Roslie (1998) and George & Uddin (2000) noted in equation number 4.4.7 and 4.4.10 respectively at chapter four. These equations were developed for subgrade soils. Figure shows that relationship between resilient modulus and DCP value for subgrade soils (silty clay) of Bangladesh lies in between the relationship established by those researchers. The relationship between resilient modulus and DCP value for subgrade soils of Bangladesh is expressed in an equation:

M_r (MPa) = 367.11 (DCP)$^{-0.4796}$

The objective of establish the above relationship for subgrade soils of Bangladesh is that this is the equation can be used to determine the resilient modulus of subgrade soils from DCP value. As cyclic triaxial equipment is unavailable and DCP is widely used in

Bangladesh. So using the DCP value of subgrade, resilient modulus can be easily estimated with the equation suggested by this study. This resilient modulus of subgrade soils is the prime input parameter for analytical pavement design.

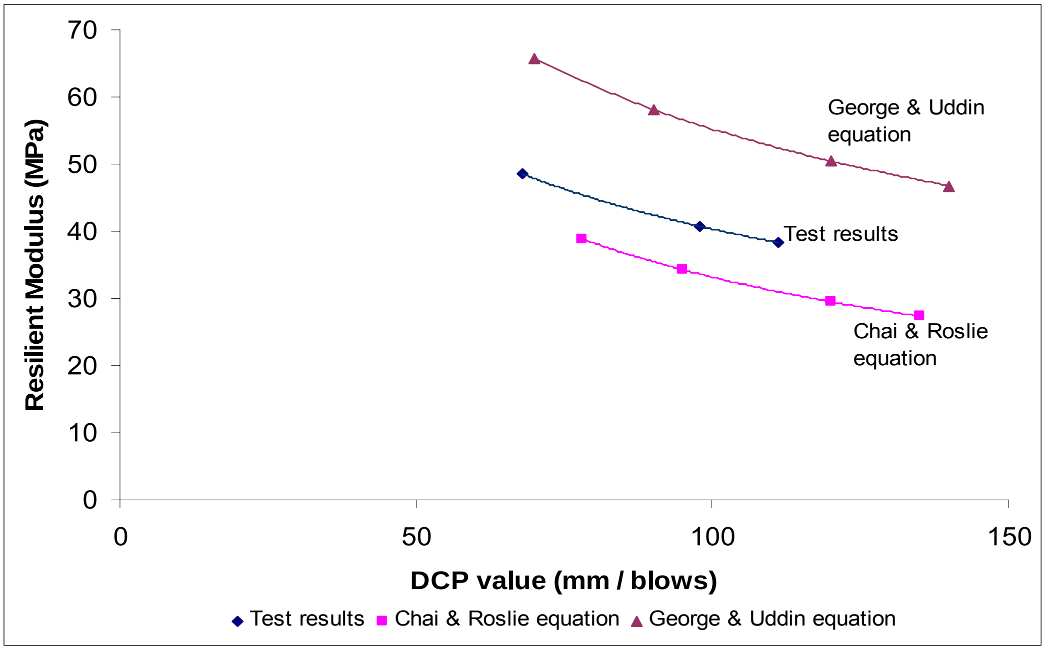


Figure 9.9: Relationship between Resilient Modulus and DCP value for subgrade soils

9.3 Improved subgrade layer

The strength properties of subgrade soils for the above mentioned roads are not comply with the specifications; for this reason an improved subgrade layer (ISG) was provided on these roads. The thickness of the ISG layer ranges from 150 mm to 500 mm found from N4 and R301 roads. This layer thickness depends on the category of the roads as well as subgrade soils properties.

9.3.1 Index Properties

The materials of ISG layer was found fine to medium granular sand. Since the materials are non plastic, so Atterberg limit tests are not conducted. The insitu moisture content found for N4 and R301 roads was ranged from 10.9% to 13.4% and 11.2 % to 11.8% respectively.

9.3.2 Particle Size Distribution

The gradations of the improved subgrade layer materials for different roads are shown in the following Figures 9.10 (a) to (b). As per RHD specifications, three grading envelopes (A, B and C) are suggested for gradation of improved subgrade layer materials. Results show that the particle size distributions for ISG layer materials of N4 road are not satisfied the grading envelope 'A' and 'B' but partially satisfied grading envelope 'C'. The same scenario was observed for ISG materials of R301road where none of the materials comply with the specified gradations. All of the materials are laid outside the gradation envelope. It may be concluded that mostly finer sand particles (fine sand) were used in improved layer of these roads and the materials are likely to uniform graded instead of well graded. So, all of the improved subgrade materials do not comply with the specifications. This grading has a great effect on strength of materials like CBR. Materials having well graded and larger particle sizes give the higher strength of the layer. It is mentioned that no improved subgrade layer was found in Z3024 road.

9.3.3 Relative Compaction

The insitu density of the ISG layer was determined in the field by sand replacement method for N4 and R301 roads. Hence, the relative density of the ISG layer for N4 road was found 91.30% to 91.72% while this for R301 road was found 89.64% of MDD achieved by BS light compaction. But Overseas Road Note 31 recommended that subgrade should be compacted during construction to a relative density of at least 100% of maximum dry density achieved in the British Standard (light) Compaction or at least 93% of the maximum dry density achieved in the British Standard (Heavy) Compaction.
So, comparing the test results with standard; it is seen that the relative density of the both roads is found below the specifications which mean less compaction was achieved during construction. It is also seen that ISG materials of N4 road is more compacted than this of R301 road.

9.3.4 Insitu CBR

The insitu CBR values of the improved subgrade layer found from the DCP test for N4, N302 and R301 roads are shown in Appendix- G, Table A - 9.1. Results show that insitu CBR value ranges from 6.44% to 46.75%, 7% to 17.43% and 19.7% to 45.08% respectively. But some excessive insitu CBR values are observed for N4 and R301 roads as in these cases some hard materials are laid under the DCP cone; results take more blows to penetrate it. However, the above mentioned insitu CBR values cannot be used as design input parameters as these values depend on the moisture content. So the above mentioned insitu CBR has been changed due to the seasonal variation and worst scenario occurred during the monsoon when roads are inundated by flood. Considering this, four days soaked CBR value has been determined for improved subgrade layer which is used as design input parameters is discussed in the following section.

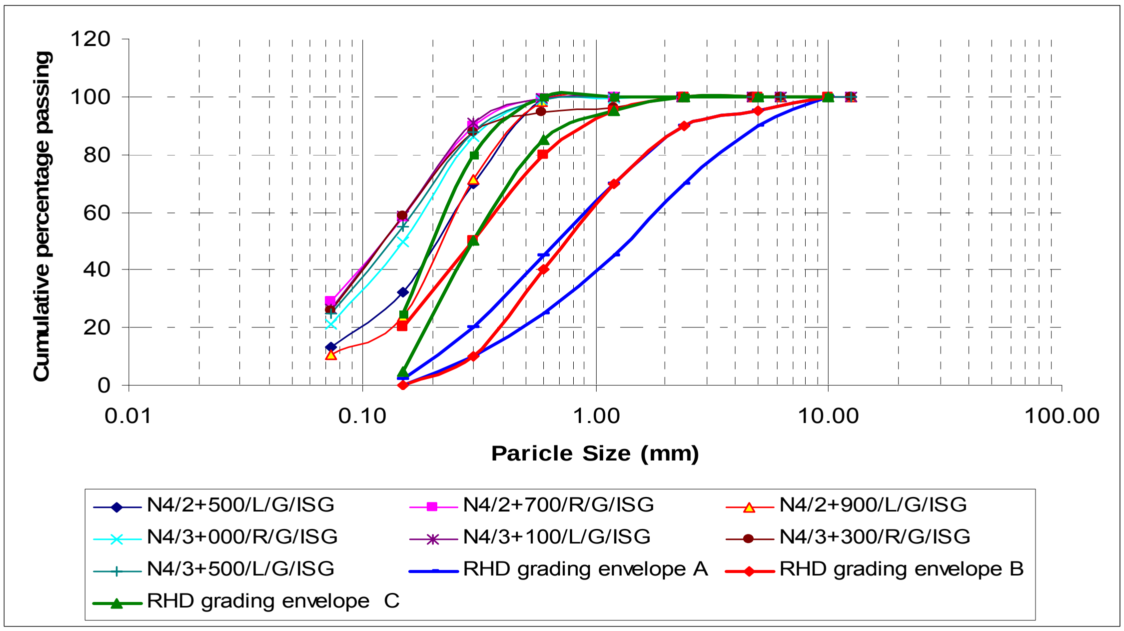


Figure 9.10 (a): Particle Size Distribution Chart for improved subgrade layer for Road Number N4.

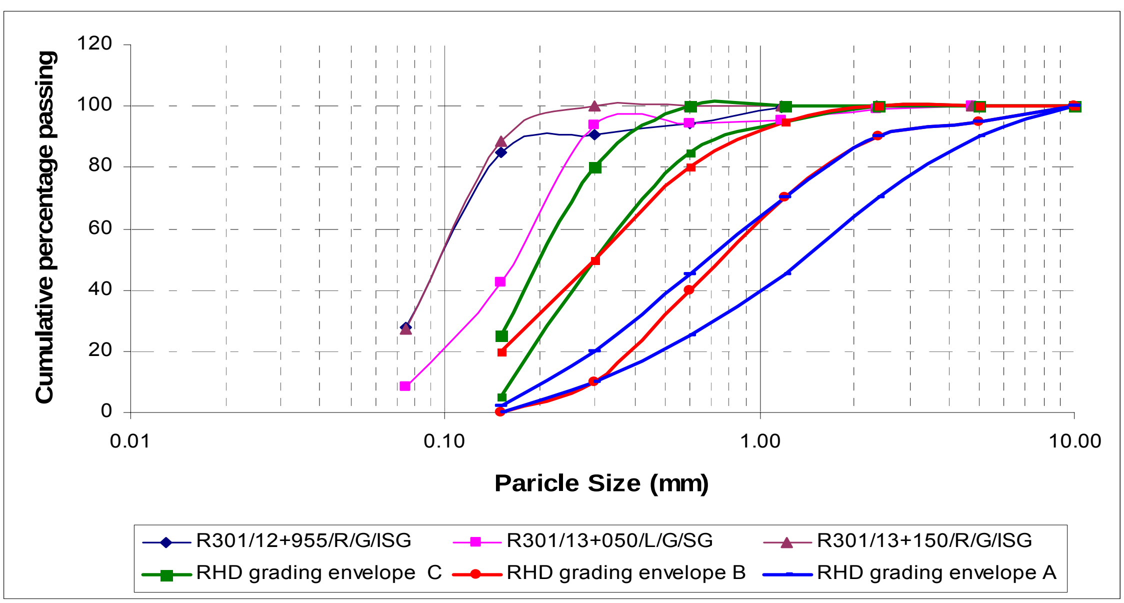


Figure 9.10(b): Particle Size Distribution Chart for improved subgrade layer for Road Number R301.

9.3.5 Laboratory Soaked CBR

The four days soaked CBR values of improved subgrade materials for N4, N302 and R301 roads are 14.45% to 14.58%, 4.5% to 12.5% and 9.85% respectively. So, all the soaked CBR values are above the specification of subgrade suggested by Overseas Road Note 31 and RHD standard specifications.

A good relationship has been observed between maximum dry densities (MDD) and soaked CBR for improved subgrade materials is shown in Figure 9.11. The relationship can be expressed in an equation:

sCBR = 0.0264 x MDD – 32.449

Where, sCBR = Four days soaked CBR (%), MDD = Maximum dry density (kg / m3)

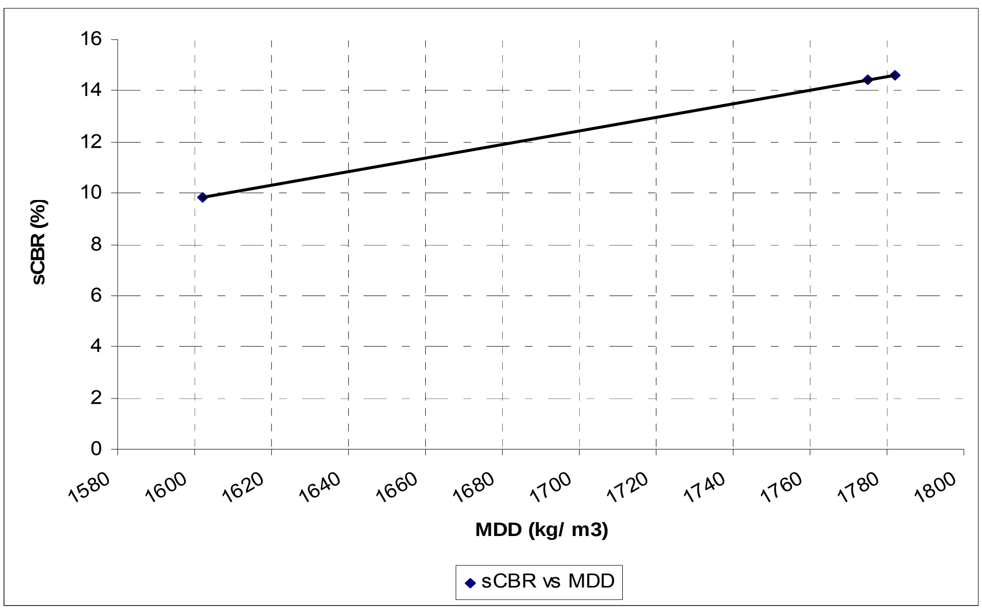


Figure 9.11: A plot of Maximum Dry Density (MDD) vs Soaked CBR for improved subgrade materials.

9.3.6 Resilient Modulus

The resilient modulus of ISG layer materials composed of fine to medium sand have been determined by cyclic triaxial test. The gradation of the materials used in this test is shown in Appendix H, Figure B - 9.2. The materials satisfied the grading envelope 'C' suggested by RHD specifications for use in ISG layer. The effective size of the particles

D_{60}, D_{10} and the mean size of the particles D_{50} was found 0.25 mm, 0.12mm and 0.225 mm respectively. The uniformity coefficient and coefficient of curvature was found 2.08 and 1.08 respectively. The materials was compacted to BS standard light compaction (standard proctor test) for representing the field conditions. As it was observed that the relative compaction of ISG layer for three roads in RHD was found less the 100% of MDD by BS standard light compaction.

At the beginning of the test, as per AASHTO T307 test protocol, the sample was conditioning by 500 repetitions of load with the confining pressure, cyclic and contract stress are 41.4 kPa, 24.8 kPa and 2.8 kPa respectively. After applying the two cycles of loading, the test has automatically stopped as the permanent strain exceeded 5% at third stage of loading. At the end of the test, resilient modulus was found 35.036 MPa for deviator stress of 27.6 kPa and confining pressure 41.4 kPa.

9.3.6.1 Effect of mica on resilient modulus of sand

Both compaction and mica contents of sand play a vital role on resilient modulus. As discussed in article 3.6 of chapter three, mica is available in most river bed sand. So its effect on stiffness properties was investigated. The test was done on sand samples mix with 0%, 5%, 10% and 15% of mica. It is mentioned that the samples were compacted with vibrating hammer to ensure the maximum compaction. The effect of different percentage of mica content on resilient modulus of sand is shown in Table 9.3. The table shows that the sand samples compacted with standard compaction (2.5 kg hammer) can't sustain beyond two stage of loading whereas those compacted with vibrating hammer can sustain three stage of loading. After these stages the tests were automatically stopped due to the permanent strain exceed 5%. It also shows that the value of resilient modulus is higher for sand compacted with vibrating hammer than standard compaction. Besides these, it also shows that resilient modulus decreases with increase in mica content of sand.

The relationship between resilient modulus and deviator stress for different mica content in sand is shown in Figure 9.12. Figure shows that increasing the mica content in sand mica mixture, the resilient modulus decreased. It is mentioned that the resilient modulus could not be determined for sand with 15% mica, because strain exceed 5% at the conditioning stage of loading.

The same relationships between resilient modulus and resilient strain & deviator stress and resilient strain were observed for increasing the mica content in sand mica mixture that were shown in Figure 9.13 and 9.14 respectively.

9.3.6.2 Relationship between resilient modulus and DCP value for sand

A relationship has established between resilient modulus and DCP value for sand used as improved subgrade layer is shown in Figure 9.15. The same relationship established by George & Uddin (2000) for coarse grained soils (sand) that is reported in equation 4.4.11 of chapter four is also shown in that figure for making comparison. Figure show that for same DCP value; test results show slightly lower resilient modulus than George and Uddin's equation. Both of the curves indicate that resilient modulus decreases with increases in DCP value. The relationship between resilient modulus and DCP value is expressed in an equation

$M_R = 358.69\ (DCP)^{-0.62}$

Where, M_R is in MPa and DCP is in mm/blow.

The above mentioned relationship can be used for estimation of resilient modulus from DCP value for improved subgrade layer.

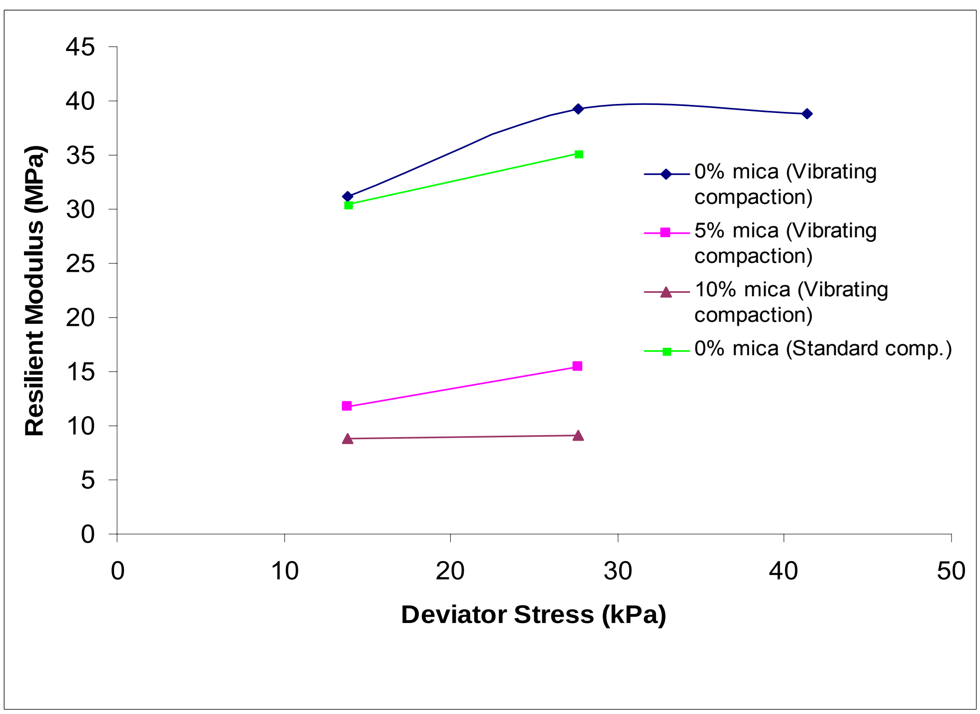


Figure 9.12: Relationship between Resilient Modulus and Deviator Stress for sand mix with different percentage of mica

Table 9.3: Resilient Modulus of sand and mixture of sand with different percentage of mica

Sample	Type of Compaction	Confining pressure (kPa)	Deviator stress (kPa)	Bulk Stress (kPa)	Permanent Strain (%)	Resilient Strain (%)	Resilient Modulus (MPa)
Sand	Standard Proctor	41.4	13.8	138	2.0	0.0409	30.42
		41.4	27.6	151.8	2.4	0.0707	35.11
Sand	Vibrating hammer	41.4	13.8	138	2.7	0.039	31.13
		41.4	27.6	151.8	2.8	0.063	39.24
		41.4	41.4	165.6	4.3	0.096	38.81
Sand +5% mica	Vibrating hammer	41.4	13.8	138	4.3	0.105	11.77
		41.4	27.6	151.8	4.5	0.161	15.37
Sand +10% mica	Vibrating hammer	41.4	13.8	138	3.2	0.141	8.79
		41.4	27.6	151.8	4.1	0.274	9.05
Sand +15% mica	Vibrating hammer	41.4	13.8	138	Sample failed at conditioning loading step due to permanent strain exceed 5%		

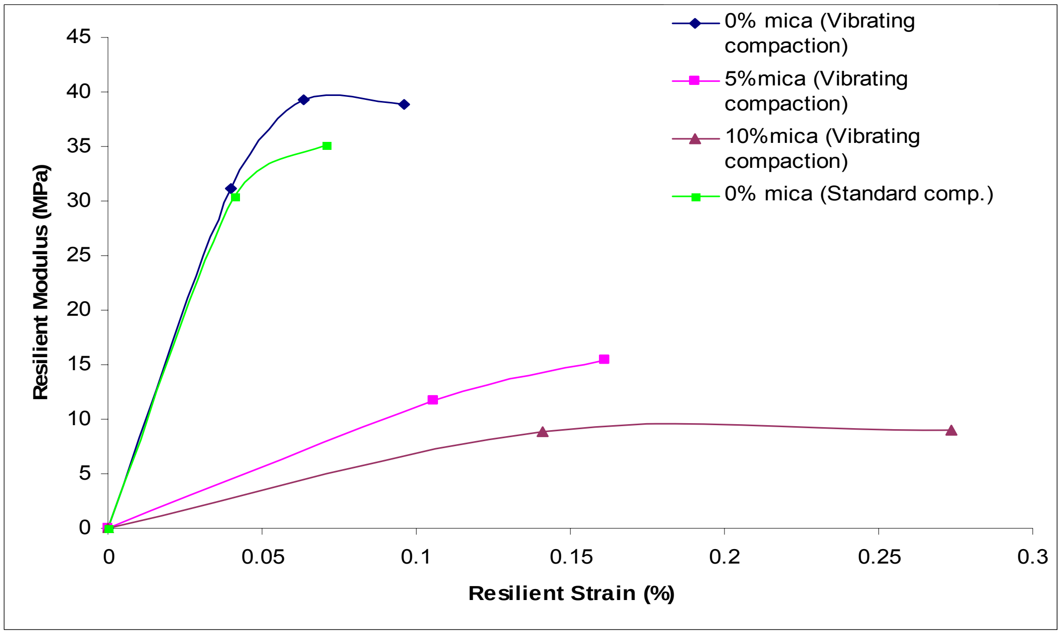


Figure 9.13: Relationship between Resilient Modulus and Resilient Strain for sand mica mixture

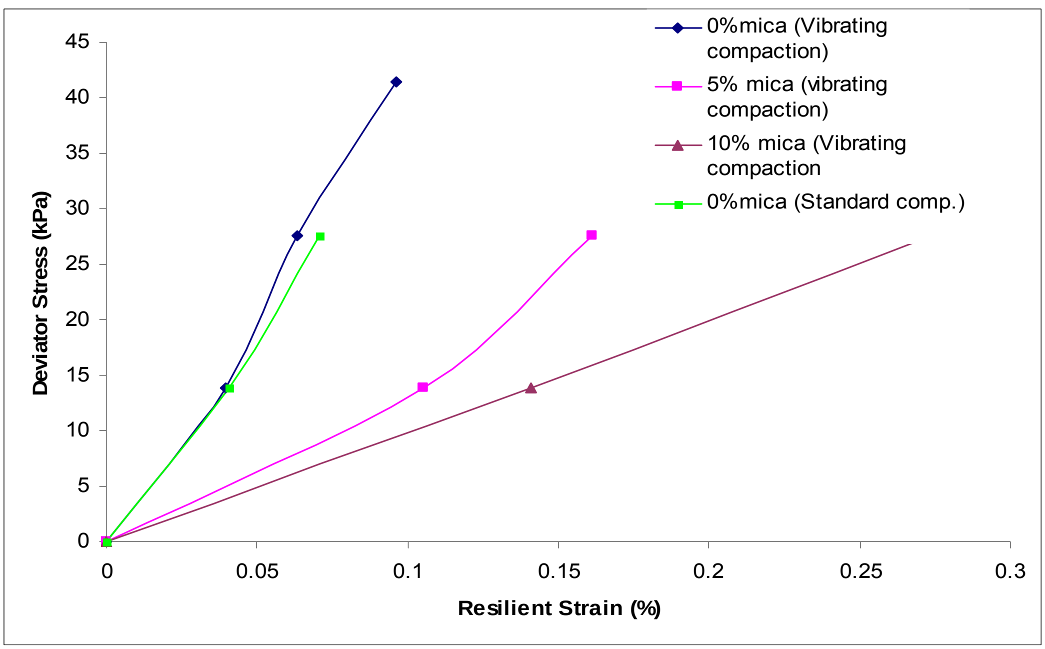


Figure 9.14: Relationship between Deviator stress and Resilient Strain for sand mica mixture

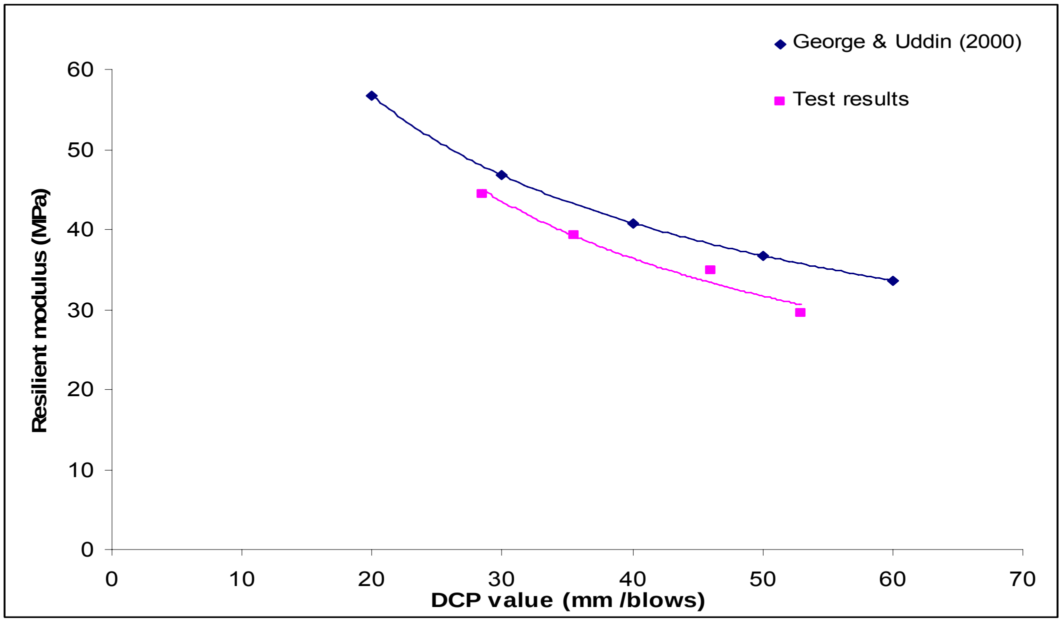


Figure 9.15: Relationship between Resilient modulus and DCP value for sand

Table 9.4: Summary of testing methods and evaluated design input parameters

Name of input parameter for design	**Name of the test**	**Appropriate standard (BS /ASTM/ others)**	**Number of tests have been done**	**Test Results used as design input parameters (upper & lower range of values)**
CBR value	Lab. Soaked CBR.	ASTM D 1883 – 92.	Subgrade = 12 Nos ISG = 9 Nos	For subgrade soils CBR = 2% to 5.23% For ISG layer CBR = 4.5% to 14.58%
Resilient Modulus (M_r)	1.Cyclic triaxial test	AASHTO T-307	4 nos subgrade soil samples & 4 nos sand samples.	subgrade soils M_r = 20 to 58 MPa Improved subgrade M_r = 30 to 36 MPa

Remarks: The results are derived for limited numbers of subgrade soils and sand samples

9.4 Summary

The higher strength of subgrade will make the pavement cost effective and longevity. The design of pavement depends on the strength of the subgrade soils. Both insitu and exsitu strength (CBR) of subgrade soils as well as ISG materials are determined. As subgrade soils are very susceptible to moisture content, exsitu (laboratory) soaked CBR value determined for the above mentioned roads will be used for design of new roads. These soaked CBR value should be however, equal to the insitu CBR value that can be achieved by stabilizing the subgrade soils. The resilient modulus of subgrade soils and ISG [18]materials are also determined in laboratory. The effect of mica on sand is also investigated as micaceous sand is available in Bangladesh. The results show that resilient modulus of the sand dramatically decreased with increase in mica content. The relationship between resilient modulus and CBR value is established for subgrade soils of Bangladesh. In addition to that, relationships between resilient modulus and DCP value for both subgrade soils and ISG materials are established.

[18] ISG materials means fine to medium sand

Chapter 10: Test Results and Discussion for Pavement Granular & Bituminous Materials

10.1 Introduction

The test results of both field and laboratory investigation and analysis of the results for pavement granular & bituminous materials are discussed in this chapter. All necessary field investigation like coring, sample collection, insitu density of different granular layers and DCP test were conducted on existing pavement of N4, N302, R301 and Z3024 roads. Meanwhile, necessary laboratory tests for all granular and bituminous materials were conducted on those collected materials from the above mentioned roads. The stress – strain properties of non conventional granular brick aggregates, determined by cyclic triaxial test are also discussed in this chapter. The details of the field and laboratory investigations undertaken in chapter seven are discussed in the following subsections.

10.2 Granular Materials

The granular materials found in the three roads investigated comprised crushed brick aggregates (non conventional aggregates), crushed stone aggregates and gravels, a mixture of sand and brick aggregates. These materials are used in two layers of pavement named base layer and subbase layer. Two different types of granular materials are found in base layer of national highway named lower base and upper base materials. The base and subbase layers are called primary and secondary load spreading layer of the pavement.

10.2.1 Granular Subbase

The subbase layer was found in N4, N302 and R301 roads except Z3024 road where one or two brick flat soling layer was found. Characterization, strength and stiffness properties of these materials are discussed in the following subsections.

10.2.1.1 Materials Used

Granular subbase layer comprised of crushed brick aggregates and sand. These two types of materials are mixed at specified gradation according to the requirements of either Overseas RN31 or RHD specification as both are applicable for Bangladesh.

10.2.1.2 Materials Strength Properties

The strength properties of granular aggregate are described by Aggregate crushing value (ACV) and Ten percent fines value (TFV). It was found that ACV for the materials of R301 road ranged from 33.51% to 39.06% and the same value for N4 road was found 30%. Moreover, TFV for the materials of N4 and R301 roads ranged from 95 kN to 100 kN and 34.56 kN to 51.95 kN respectively. RHD specification suggested that ACV for subbase materials should not be greater than 38% and TFV should not be less than 75 kN. So, comparing the test results with the above mentioned specifications, the materials of N4 comply with the standard but those of R301 do not comply with the standard.

10.2.1.3 Gradation

The particle size distribution of the materials for N4 and R301 are shown in Figure 10.1 (a) and 10.1(b) respectively. Both upper and lower limit of gradation envelope suggested by Overseas Road Note 31(TRL) and RHD are also shown in each graph. From the gradation curves, it is seen that all of the materials lie within the boundary lines of specification which means that materials gradation satisfied the TRL and RHD requirements. On the other hand, it is also observed that materials are well graded that provide maximum strength when compacted with vibrating compactor.

10.2.1.4 Relative Compaction

The insitu density was determined in the field by sand replacement method. The maximum dry density of the same materials was also determined in the laboratory. Based on these two, relative compaction of the subbase materials was found almost 86% to 88% and 87% to 91% of MDD compacted by vibrating hammer for road N4 and R301 respectively. But both TRL and RHD specifications suggested that relative density should not be less than 98% of MDD compacted by vibrating hammer. So, the materials for both roads do not comply with the criteria suggested by the both standards. This is one the reasons for subbase materials to provide lower insitu strength. Besides this, those comparatively loose materials will exhibit higher strain or deflection due to the repeated wheel load and resulting shows lower stiffness value. Nonetheless, all the roads are poorly compacted and likely to exhibit reduced durability and increased susceptibility to rutting.

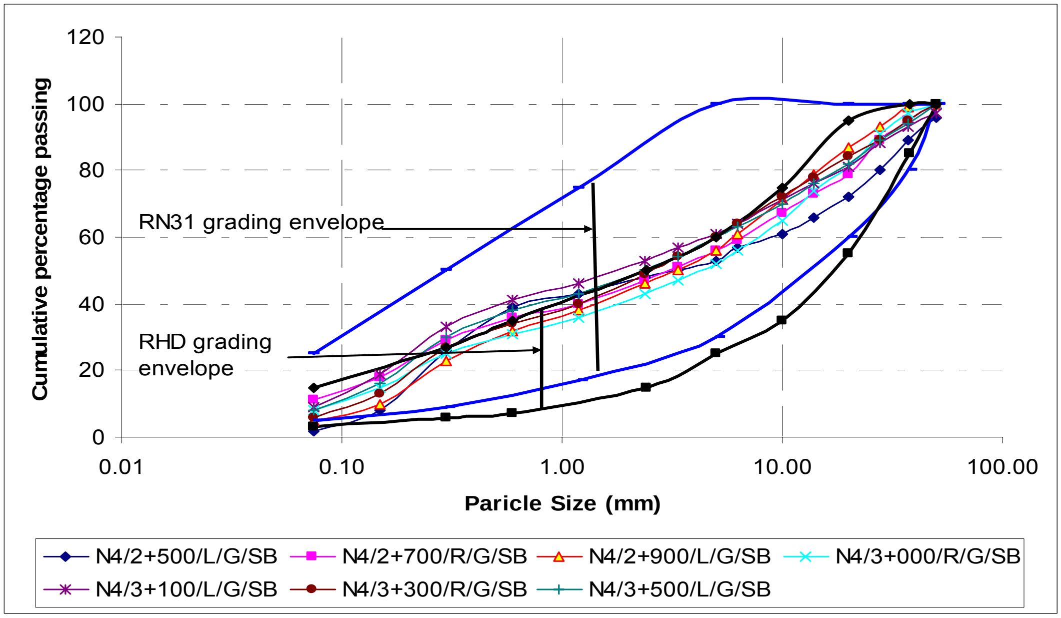


Figure 10.1(a) : Particle Size Distribution for granular subbase materials of N4 Road

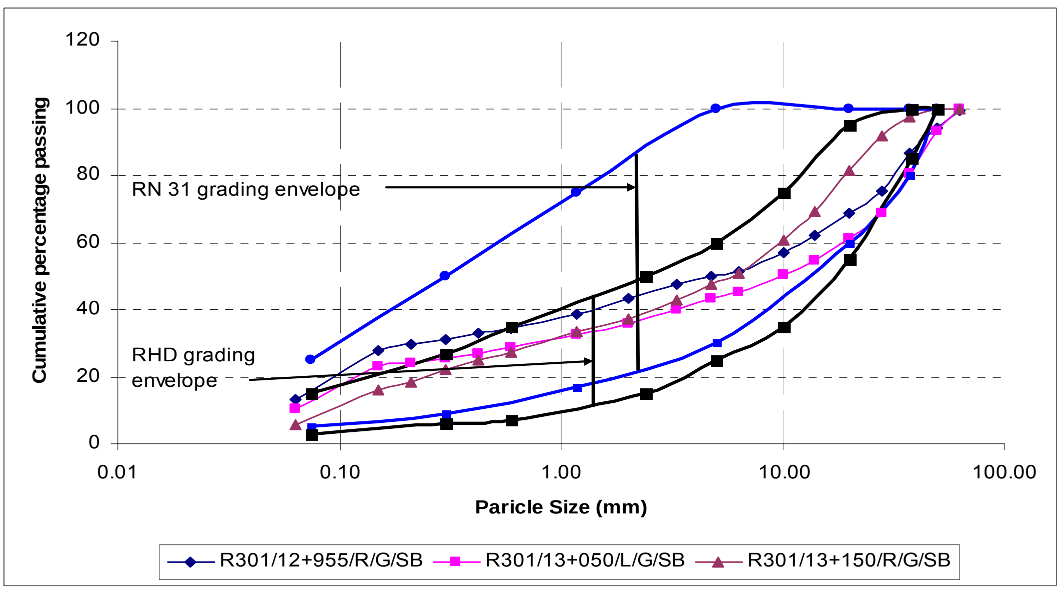


Figure 10.1 (b): Particle Size Distribution for granular subbase materials of R301 Road

10.2.1.5 Insitu California Bearing Ratio (CBR)

The insitu strength of granular subbase materials is measured by CBR value. This insitu CBR value was indirectly measured from the DCP test using Riley et al. (1987) equation, Kleyn and Van Hearden chart and Harrison's equations for N4, N302 and R301 roads shown in Appendix G, Table A- 9.1. In most of the cases, Riley et al. (1987) equation shows lowest CBR value. So, the lowest insitu CBR value is considered for design. Results show that the values are ranged from 55 % to 100%, 25% to 46% and 56% to 100% for the mentioned roads respectively. These insitu CBR values are well above the recommended values of CBR suggested by [19]Overseas RN 31 and [20]RHD specifications.

10.2.1.6 Soaked CBR

CBR test is universally acknowledged and used in the design of parameters for roads and airfields. However, in order to see the true value, test should be done to simulate worst conditions. Considering the worst case scenario of the subbase materials, four days soaked CBR value was determined and reported in Table 10.1. The soaked CBR value of subbase materials compacted by vibrating hammer for N4, N302 and R301 ranged from 38% to 46%, 27% to 53% and 31% to 74% respectively. Overseas Road Note 31 suggested that the soaked CBR of the subbase materials should be greater than 30% whereas RHD specification suggested that value should be greater than 25% when compacted to 98% of MDD as determined by vibrating hammer. The soaked CBR values of the above mentioned roads were satisfied with the specifications suggested by TRL as well as RHD. As the pavement design considers the worst case condition of materials, so these soaked CBR values are used as design input parameter.

10.2.2 Granular Base Layer

The base layer was constructed in all roads named N4, N302, R301 and Z3024. Two types of base named upper base and lower base layers were found in through out the N4 road and in one section of R301 road under study area. Only one base layer was found in N302, Z3024 and most part of R301roads. The materials found from four roads and their characteristics, strength and stiffness properties are discussed in the following subsections.

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[19] Overseas RN31 suggested that CBR for subbase layer should be greater than 30%.
[20] RHD specifications suggested that CBR for subbase layer should be greater than 25%

10.2.2.1 Materials Used

The base layer of pavement comprises crushed stone or gravel or crushed brick aggregate (as per specified gradation). Two layers of base course materials were observed in N4 and R301 road but single base course materials was observed in both the N302 and Z3024 roads. The upper and lower base layers are comprised of crushed stone or gravel and crushed brick aggregate respectively. The upper base layer was subjected to higher tyre pressure compared to the lower base, hence stiffer materials (crushed stone aggregate) are used in that layer. It is mentioned that the tyre pressure is decreased with increasing depth. So considering the economy of design, available and cheapest materials (brick aggregate) are used in lower base layer, as crushed stone aggregate is expensive in Bangladesh.

Table 10.1: Properties of Granular materials collected from N4, R301 & Z3024 roads

Layer Name	Road No	Sample No	Soaked CBR (%) Test result	Soaked CBR (%) RHD Spe.	MDD (kg / m^3)	ACV (%) Test result	ACV (%) RHD Spe.	TFV (kN) Test result	TFV (kN) RHD Spe.	FDD (kg /m^3)	Relative compaction (%) Test result	Relative compaction (%) RHD Spe.
Sub-base	N4	N4/2+700/ R/G/SB	45.5	>25	1810	30.0	<38	95.0	> 75	1552	85.75	>98
		N4/3+100/ L/G/SB	38.00		1785	30.0		100		1567	87.79	
	R301	R301/12+955/R/G/SB	31.25		1720	39.1		34.5		1572	91.40	
		R301/13+050/L/G/SB	73.79		1768	33.5		51.9		1534	86.76	
Base	N4	N4/2+700/ R/G/[21]RB1	119.0	>80 (Upper base) > 50 (Lower base)	2200	23.0	<30 (Upper base), <35 (lower base)	140	> 125 (Upper base) > 90 (Lower base)	1940	88.18	>98
		N4/2+700/ R/G/[22]RB2	29.00		1790	31.0		95		1663	92.91	
		N4/3+100/ L/G/RB1	112.5		2200	24.0		140		1955	88.86	
		N4/3+100/ L/G/RB2	128.5		1800	31.0		110		1619	89.94	
	R301	R301/12+955/R/G/ RB1	75.51		1718	31.3		44.9		1557	90.63	
		R301/12+9	71.34		1740	30.7		55.3		1446	83.10	

[21] RB1 = upper base layer of pavement.
[22] RB2= lower base layer of pavement.

	55/R/G/RB2						
	R301/13+050/L/G/RB1	34.97	1735	29.8	78	1592	91.76
Z3024	Z3024/4+820/R/G/RB	78.28	1710	40.4	72.1	1572.69	91.97
	Z3024/5+000/L/G/RB	62.56	1674	32.6	68.3	1513.09	90.39

Remarks: MDD = maximum dry density, FDD = field dry density, ACV= Aggregate crushing value, TFV = Ten percent fines value, Relative compaction = (FDD/MDD) X 100%

10.2.2.2 Materials Strength Properties

The mechanical strength requirement of the aggregate which indicates the durability of the aggregate is measured by Aggregate Crushing value (ACV) and Ten Percent Fines value (TFV). The test results are given in Table 10.1. Overseas Road Note 31 suggested that TFV for road base materials should be greater than 110 kN whereas RHD specification suggested that TFV should be greater than 125 kN and 90 kN for upper and lower base course respectively. Besides this, RHD specification also suggested that ACV for upper and lower base materials should be less than 30% and 35% respectively. So, comparing the test results with the above mentioned specifications, the materials of both upper and lower base course of N4 road comply with the both of specifications but TFV for the materials of R301 road and Z3024 road do not comply with the specifications. So, it is observed that sub-standard materials are used in both regional highway (R301) and Zilla road (Z3024).

10.2.2.3 Gradation

The particle size distribution of base course materials of N4, R301 and Z3024 roads are shown in Figure 10.2 (a), (b) and (c) respectively. The grading envelope of base course materials suggested by Overseas Road Note 31, RHD and ASTM also gives upper & lower boundaries. The gradation curve of materials used in N4 road, comply with RHD specifications but do not fully comply with Overseas RN 31 and ASTM specifications. It is seen that the gradation limit of RHD specifications is much wider than this of Overseas RN31 and ASTM specifications. It is also seen from the figure that all of the materials are well graded.

Most of the grading curves of the materials used in R301road comply with the RHD specifications and partly comply with ASTM specification. Materials coarser than 10 mm sizes comply with RN31 specification curve (for maximum sizes of particles 37.5mm) and below this size fraction, it does not meet with that specification. The same grading envelope observed for the materials of Z3024 road where most of the grading curves comply with RHD specifications but none of them fully comply with Overseas RN31 and ASTM specifications. It is also seen that most of materials are well graded.

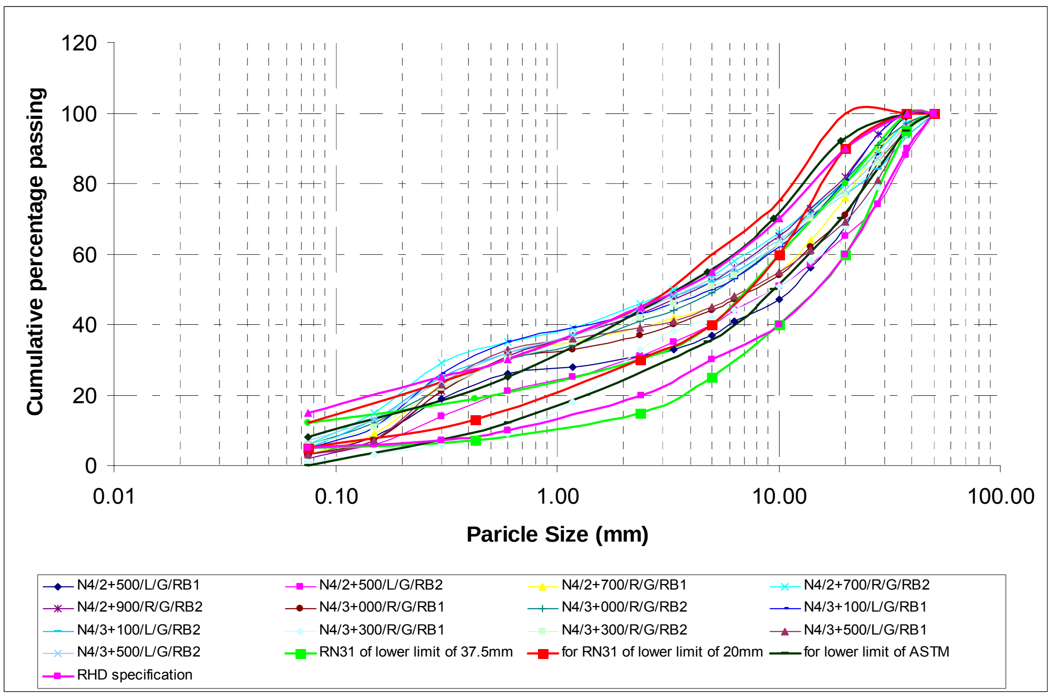

Figure 10.2 (a): The grading curves for the base course materials of N4 road and grading envelope suggested by ASTM, RN31 & RHD standard.

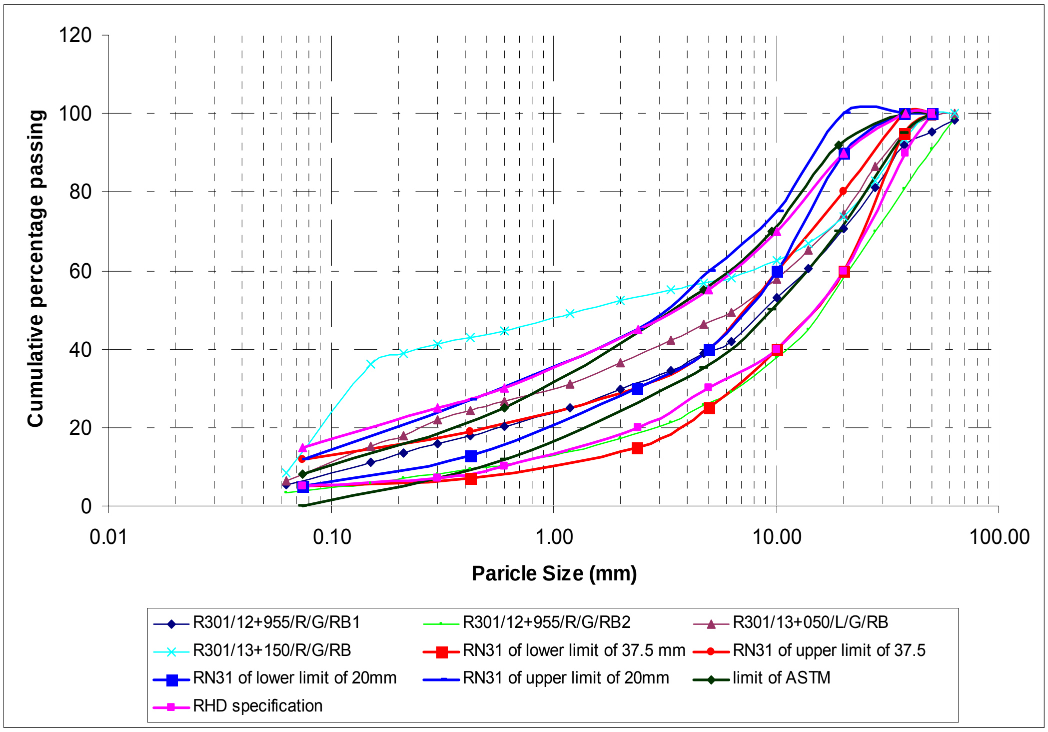

Figure 10.2 (b): The grading curves for the base course materials of R301 road and grading envelope suggested by ASTM, RN31 & RHD standards.

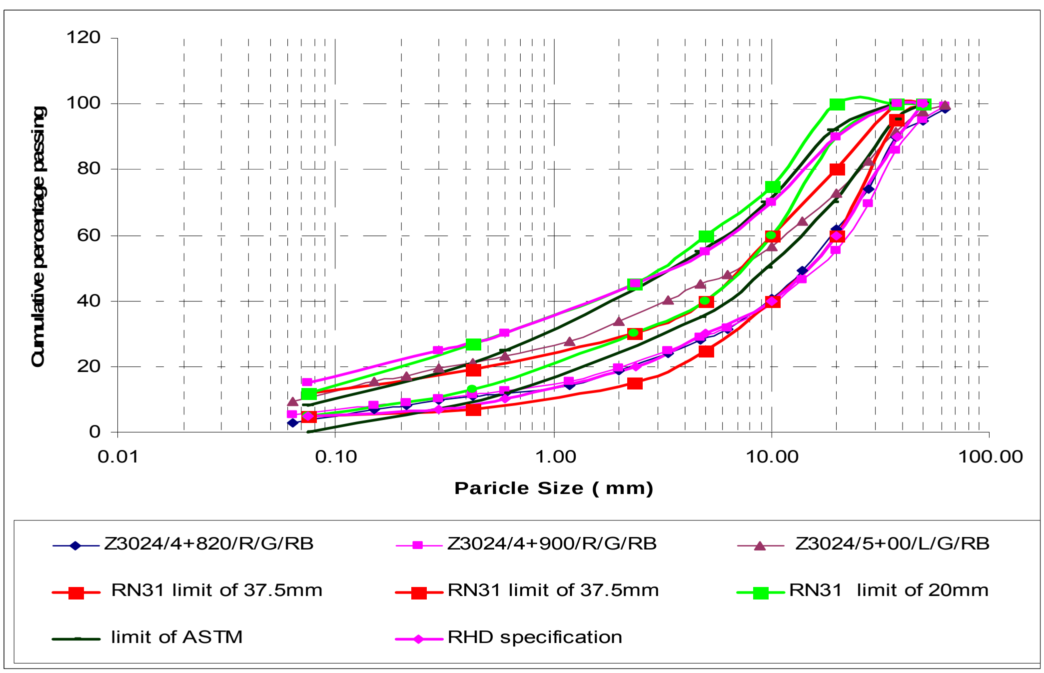

Figure 10.2(c): The grading curves for the base course materials of Z3024 road and upper & lower limits for ASTM, RN31 & RHD standards.

10.2.2.4 Relative Compaction

The relative compaction of the base course materials are shown in Table 10.1. The relative compaction of the base course materials was found 88% to 93%, 83 % to 92% and 90% to 92% of MDD compacted by vibrating hammer for N4, R301 and Z3024 roads respectively. It is seen that Zilla roads are marginally better compacted than National and Regional roads. Differences are not significant. But both TRL and RHD specifications suggested that the relative density should not be less than 98% of MDD compacted by vibrating hammer. So, the materials for all three roads do not meet the criteria suggested by the both standards. This is one the reasons for base materials to provide lower insitu strength. Besides this, those comparatively loose materials will exhibit higher strain or deflection subjected to the repeated wheel load and resulting lower stiffness value. Nonetheless, all the roads are poorly compacted and likely to exhibit reduced durability and increased susceptibility to rutting.

10.2.2.5 Insitu CBR

The insitu CBR value for base course materials was estimated through correlations with DCP test. The CBR value was determined from DCP value using the Riley et al. (1987) equation, Kleyn & Van Hearden chart and Harrison equation and reported in Appendix G, Table A - 9.1. The lowest insitu CBR value is considered for design. The CBR value for upper and lower base layers of N4 road ranged from 62% to 100% and 68% to 100% respectively. But in some cases, insitu CBR value of upper base layer shows comparatively lower because of the relative movement of granular crushed stone aggregate when DCP test was started from top of the granular base. Again, insitu CBR value for base course of N302 road ranged from 53% to 100%. The same value for R301 road for base course materials ranged from 73% to 100% while this for Z3024 was found 100%.

10.2.2.6 Soaked CBR

Considering the worst case scenario of base course materials, four days soaked CBR value were determined and also reported in Table 10.1. Results show that soaked CBR value of upper and lower course base materials for N4 road ranged from 113 % to 119% and 29% to 129% respectively. Again the soaked CBR value of upper base course materials for R301 road ranged from 35% to 71% while this for lower base course was found 75.51%. On the other hand, the soaked CBR value of the base course materials for N302 and Z3024 are ranged from 66% to 85% and 63% to 78 % respectively. Overseas Road Note 31 suggested that the soaked CBR of the base course materials should be greater than 80% whereas RHD specification suggested that the value should be greater than 80% and 50% for upper and lower base course when compacted to 98% of MDD as determined by vibrating hammer. So, both upper and lower base course materials for N4 road comply with the specifications but none of the soaked CBR values for R301 comply with the specifications. Besides these, some of the soaked CBR value of N302 meet the specification but none of those values for Z3024 meet the specifications. Moreover, this laboratory soaked CBR value for upper base course materials of N4 road are well above the insitu CBR value while this for lower base course of N4 road and other two roads are lower than the insitu CBR.

10.2.2.7 Resilient Modulus

The resilient modulus of non conventional brick aggregates used in road subbase and base of Bangladesh was determined by cyclic triaxial test. Subbase and base materials composed of mixture of brick aggregates with sand and brick aggregates respectively as per specified gradation suggested by TRL. The test was conducted on four and five samples for subbase and base materials respectively. Four samples were tested for subbase and base materials each following the fifteen sequences of test condition suggested by AASHTO T307 protocol. One base sample was tested by providing the maximum axial stress and confining pressure following the user defined condition of that testing protocol. The relationship between resilient modulus and deviator stress for subbase and base materials are shown in Figure 10.3 and 10.4 respectively. Results show that resilient modulus increases with increase in deviator stress for both of the granular materials. The test results for subbase and base materials are given in Appendix G, Table A- 10.1 and A- 10.2 respectively. The average resilient modulus corresponding to maximum deviator stress for subbase and base materials was found 180 MPa and 195 MPa respectively. The maximum permanent deformation for subbase and base materials were found 3.2% and 3% respectively.

The relationship between deviator stress and resilient strain for different confining pressure for subbase and base materials are shown in Figure 10.5 and10.6 respectively. Results show that for small confining pressure, smaller increase in deviator stress results higher increase in resilient strain. On the other hand, at high confining pressure, higher increase in deviator stress results smaller increases in resilient strain.

The relationship between resilient modulus and resilient strain for subbase and base materials is shown in Figure 10.7 and 10.8 respectively. Results show similar relationship like deviator stress and resilient strain.

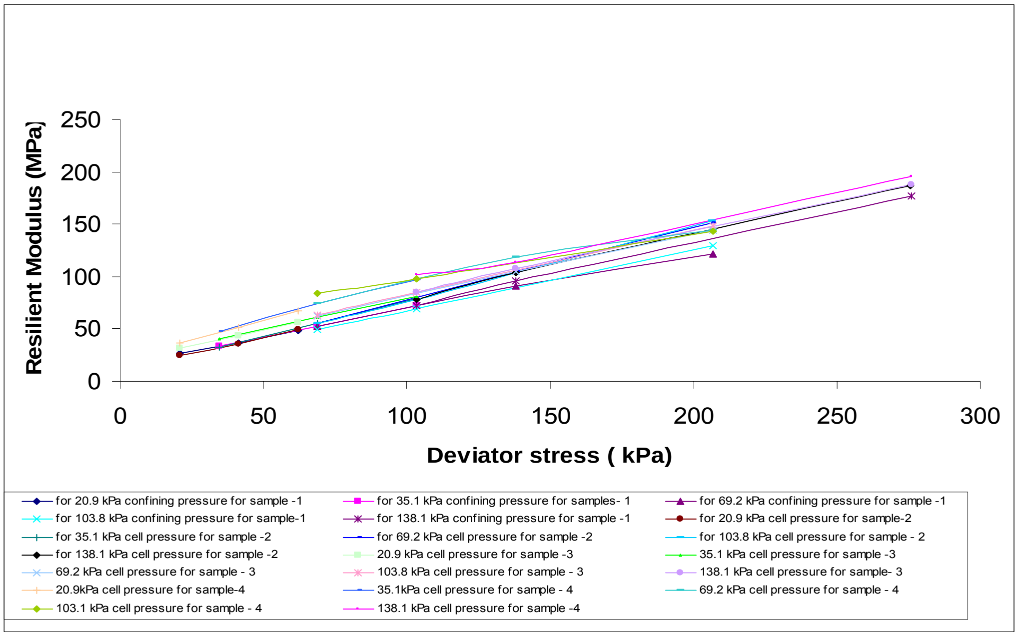

Figure 10.3: Relationship between Resilient Modulus and Deviator Stress for subbase materials

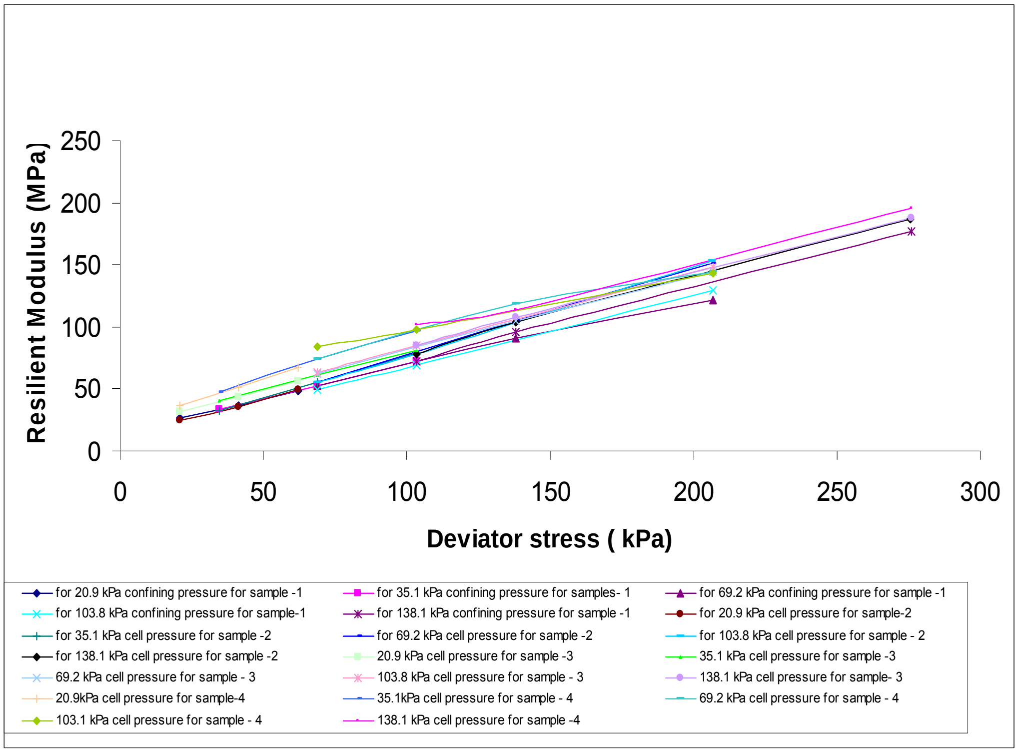

Figure10.4: Relationship between Resilient Modulus and Deviator Stress for base materials.

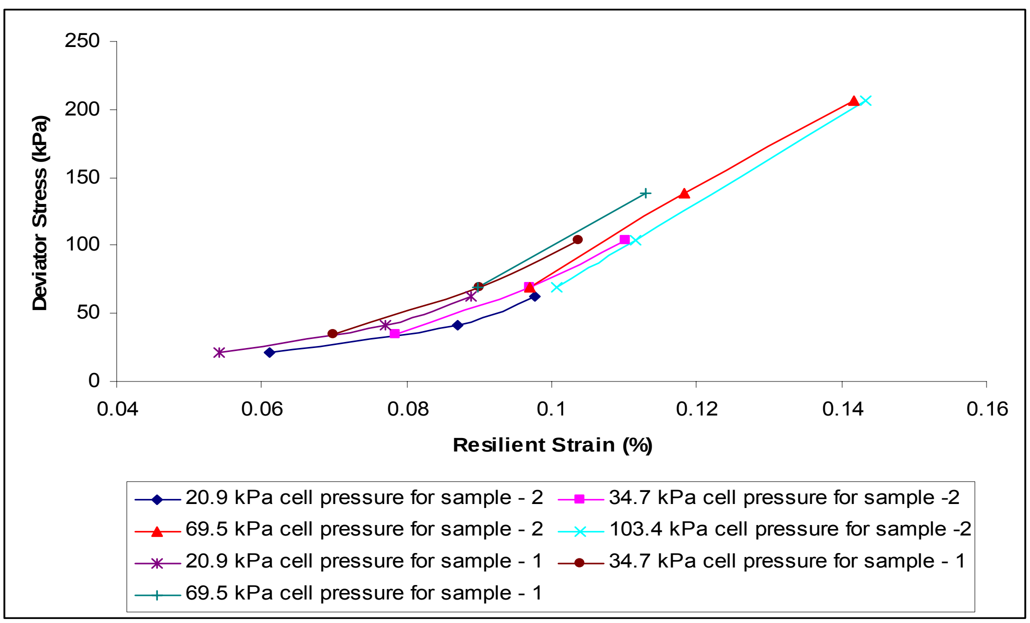

Figure 10.5: Relationship between Deviator Stress and Resilient Strain for subbase materials

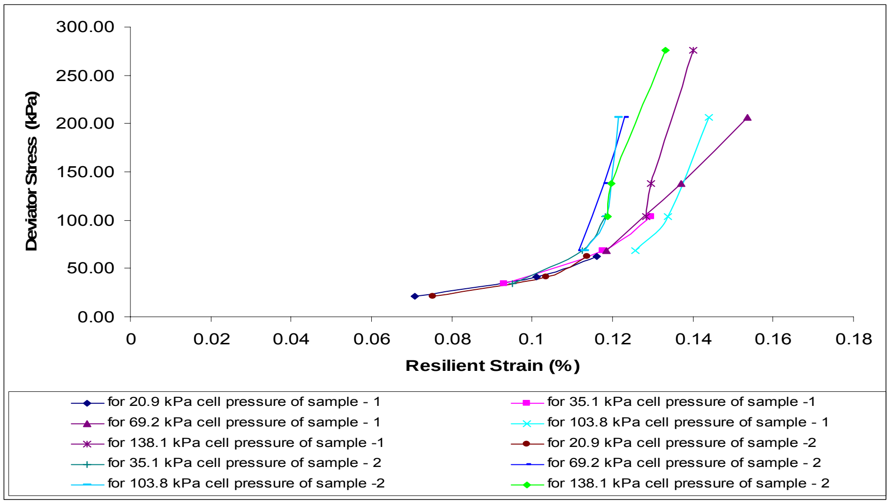

Figure 10.6: Relationship between Deviator Stress and Resilient Strain for base materials.

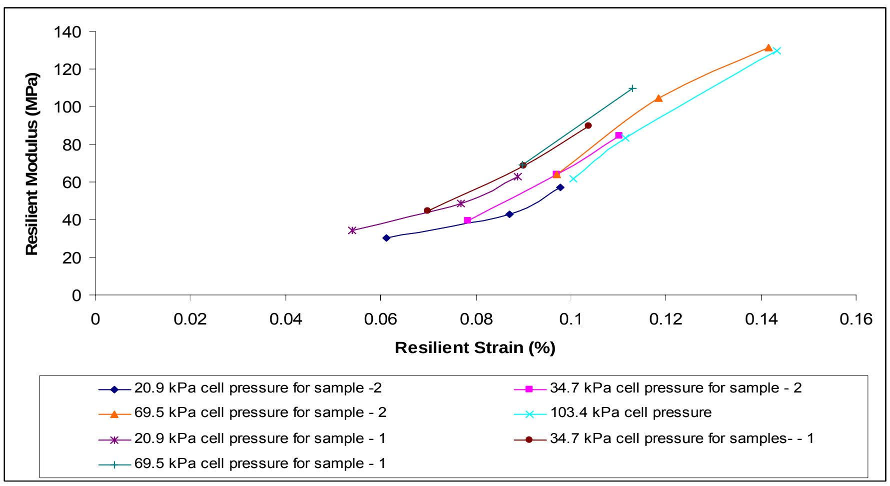

Figure 10.7: Relationship between Resilient Modulus and Resilient Strain for subbase materials

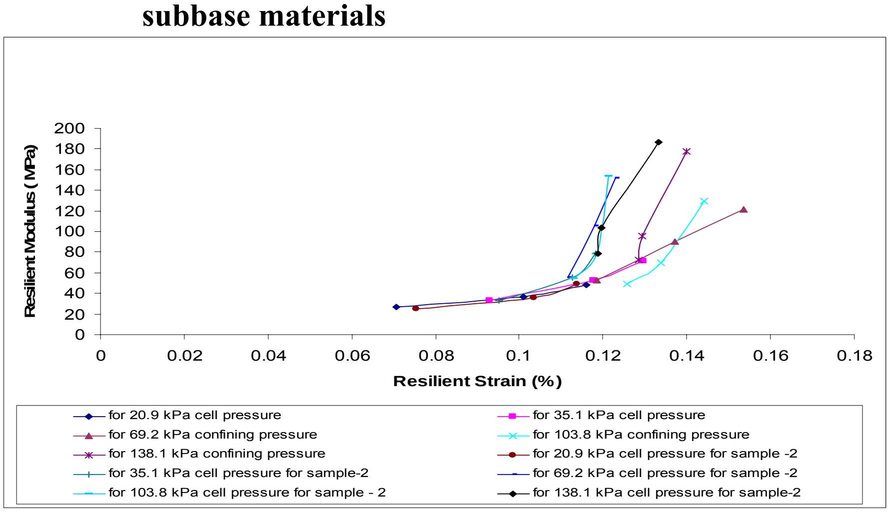

Figure 10.8: Relationship between Resilient Modulus and Resilient Strain for base materials.

10.2.2.7.1 Resilient Modulus modelling for granular brick subbase materials

Several resilient modulus models have been successfully used to describe the stress-strain behaviour of granular materials. Among them k – θ model has been widely used for granular subbase materials:

$M_r = k1\ (\theta)^{k2}$

Where, M_r = resilient modulus (MPa), θ = bulk stress (kPa) and

k1, k2 = regression parameters

In this study, k – θ model was used for mixture of brick aggregates and sand as per specified gradation. The values of k1 and k2 were determined for samples 1, 2, 3 & 4 using the relationship between resilient modulus and bulk stress noted in Figure 10.9. The values of k1, k2 and k - θ model for these samples are summarised in Table 10.2. Table shows that the value of k1 decreases with increases in k2 value and vice versa.

The k - θ model for unbound granular subbase materials has been compared with that derived by Flintsch et al. (2005) equation is shown in Figure 10.10. The test results show the lower resilient modulus for same deviator stress value than those suggested by Flintsch et al.. Flintsch et al. equation was derived for stone aggregate whereas the test results for this study are derived for brick aggregate.

10.2.2.7.2 Resilient Modulus of granular brick base materials

Similar k - θ model were used to describe the stress- strain behaviour of unbound granular brick base materials. The values of k1 and k2 were determined for four samples using the relationship between resilient modulus and bulk stress noted in Figure 10.11. The values of k1, K2 and k - θ model for samples 1, 2, 3 and 4 are summarised in Table 10.2. Results show that the value of k1 decreases with increases in k2 value and vice versa.

The k – θ Model for brick aggregates were compared with Rada & Witczak model (1981) and Monismith model (1992) shown in Figure 10.12. Both Rada and Monismith models were derived for crushed stone aggregates road base materials whereas the test result was derived for brick aggregates. Both of the models give higher resilient modulus than results obtained during this study for the same bulk stresses. Because crushed stone aggregates base materials have higher strength and less abrasion value than brick aggregates.

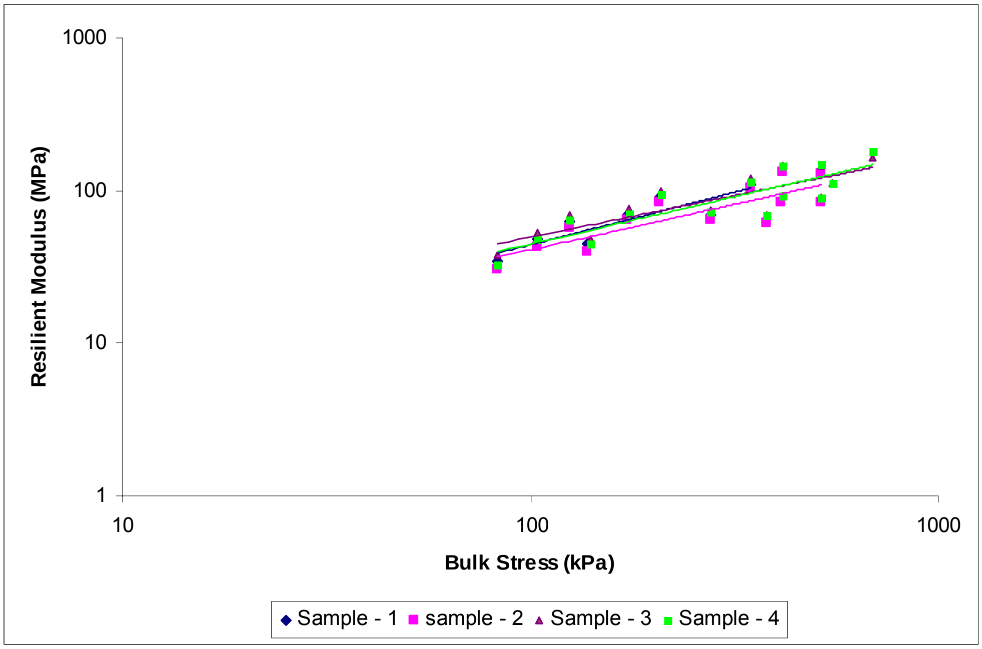

Figure 10.9: Relationship between Resilient Modulus (Mr) and Bulk Stress (θ) for subbase materials.

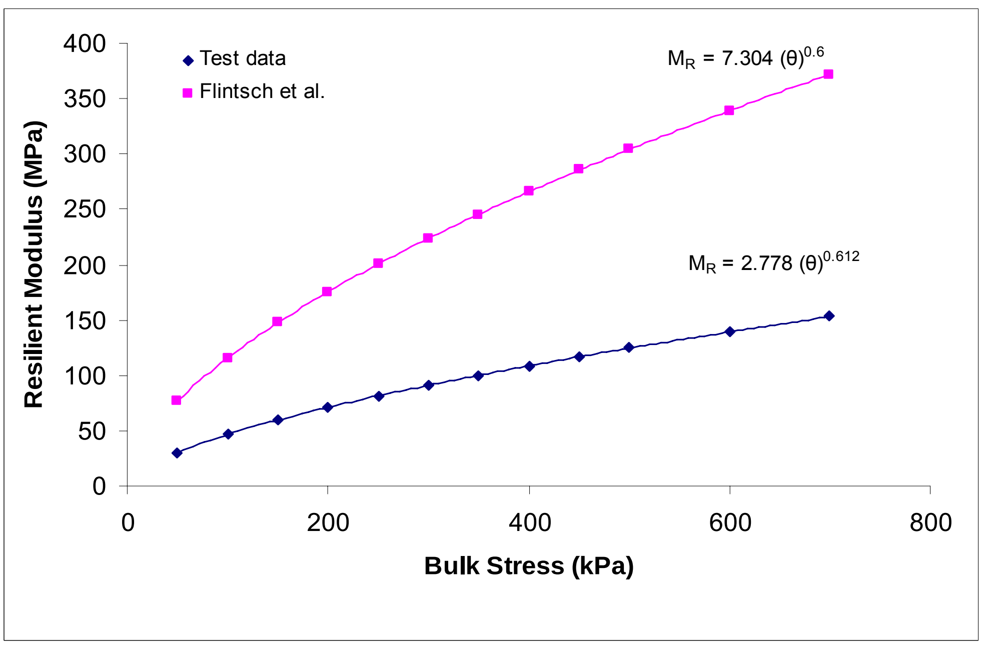

Figure 10.10: Comparison of k – θ model for Subbase materials

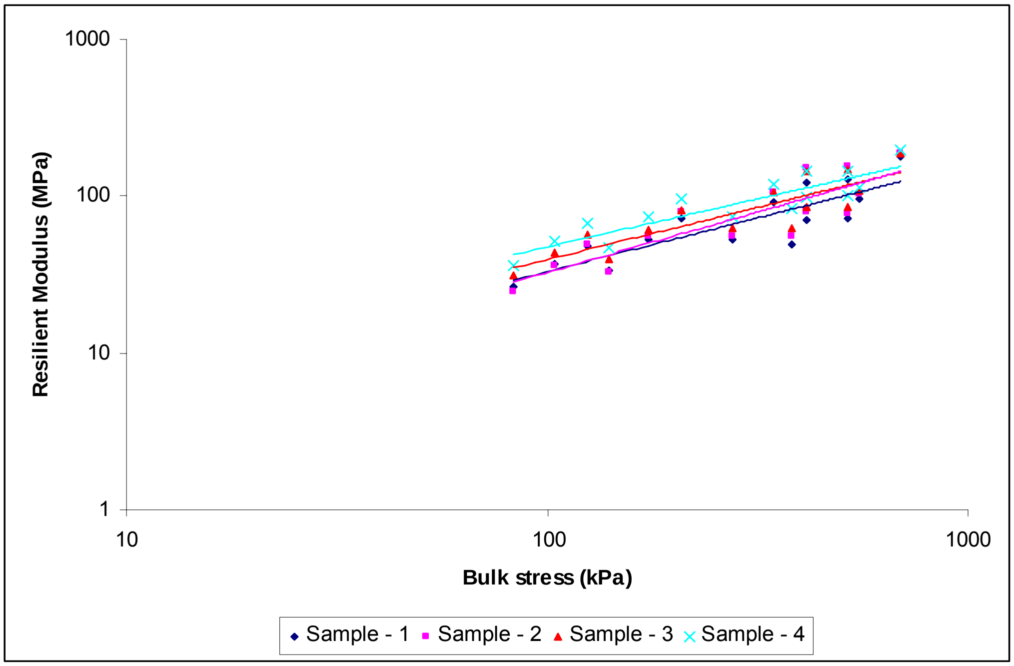

Figure 10.11: Relationship between Resilient Modulus (Mr) and Bulk Stress (θ) for granular brick base aggregate samples

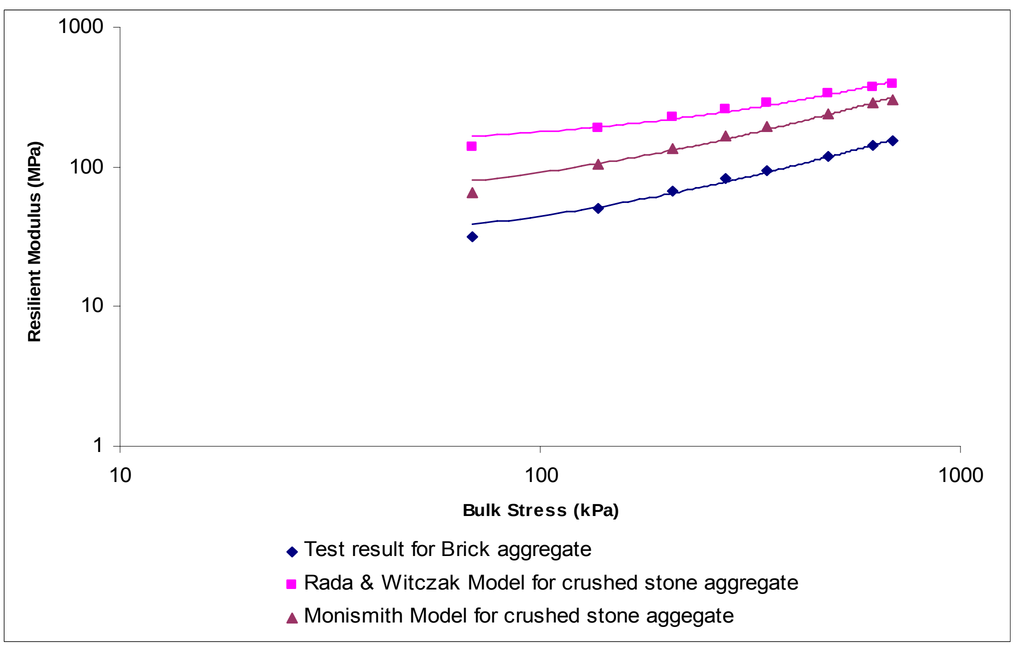

Figure 10.12: Comparison of k – θ model for brick aggregates and stone aggregates base

Table 10.2: Values of k1 & k2 for granular brick aggregate samples

Materials	Sample Number	K1	K2	R^2	k – θ Model
Subbase	1	1.8797	0.6857	0.78	
	2	2.6046	0.5976	0.72	
	3	4.0525	0.5438	0.73	$M_R = 2.778\ (\theta)^{0.612}$
	4	2.577	0.6192	0.76	
	Mean	2.778	0.612		
Base	1	1.419	0.6838	0.76	
	2	0.9527	0.7676	0.76	
	3	1.8362	0.6656	0.78	$M_R = 1.772\ (\theta)^{0.682}$
	4	2.879	0.6092	0.81	
	Mean	1.772	0.682		

10.3 Bituminous layer

This layer consists of combinations of hot mineral aggregates mixed with hot bitumen, sand and filler. The thickness of the bituminous surfacing depends on the traffic volume. Thick bituminous layer is constructed with two layers named base course and wearing course. Wearing course is the topmost layer laid on a base course. Both binder base course and wearing course were found at N4 road and only wearing course is found at R301 and Z3024 roads. As mentioned in chapter seven, coring and trial pits were constructed on N4, R301and Z3024 roads.

10.3.1 Materials and Thickness of the layer

The thickness of the bituminous layer was determined from cores. Cores were cut from N4, R301 and Z3024 roads. The bituminous layer thickness of N4 road was found 39 mm to 40.3 mm and 67.3 mm to 76 mm for wearing and base course respectively. The thickness of wearing course for R301 and Z3024 roads were found 37.7 mm to 66.7 mm and 14.3 mm to 33mm respectively. The materials found from the bituminous cores were bitumen and graded stone aggregates.

10.3.2 Extraction of the Bitumen

The bitumen was extracted from bituminous materials collected from the field by coring. The bitumen content for different samples was shown in Table 10.3. Results show that it ranges from 4.77% to 6.15% and 5.1% to 5.63% for wearing course and base course materials respectively of N4 road. Meanwhile, the bitumen content of wearing course was found 4.46% and 4.13% for R301 and Z3024 roads respectively.

10.3.3 Properties of recovered Aggregates

Aggregates were separated from bituminous materials after extraction of bitumen. These aggregates were subjected to dynamic loading and temperature fluctuation under insitu conditions results properties were changed from its original properties. The ACV and TFV test, particle size distribution test, elongation & flakiness index test were performed in the laboratory. The details are discussed in the following subsections.

10.3.3.1 Strength Properties

The mechanical strength of coarse aggregates measured by ACV and TFV are given in Table 10.4. Results show that the ACV and TFV ranged from 20% to 23% and 120 kN to 160 kN respectively. However, the aggregates ACV and TFV should be less than 25% and should be greater than 150 kN suggested by RN 31 and RHD specifications respectively. So, aggregates crushing values are within the acceptable limits but some TFV values are not satisfied the criteria.

10.3.3.2 Gradations

The particle size distributions of the aggregates used in wearing and base courses are shown in Figure 10.13 and 10.14 respectively. Figures show that the gradation of the aggregates of wearing course for N4 road complies with the RN31 specifications (mix designation WC1) but those for R301 and Z3024 do not meet with the specifications. As per RN31 specifications wearing course aggregates finer than 10 mm should be 70% to 90% whereas those for R301 and Z3024 were found 55% and 35% respectively. So, it is also seen that relatively coarser aggregates were used in wearing course for R301 and Z3024 roads. These coarser aggregates require less binder content due to the less surface areas.

Base course was found only N4 road and the gradation of the aggregates recovered from the respective cores observed that most of the aggregates meet with RN 31 specifications (mix designation BC1) but some of the aggregate gradation laid outside the grading envelope that are finer than the specifications and the materials are well graded.

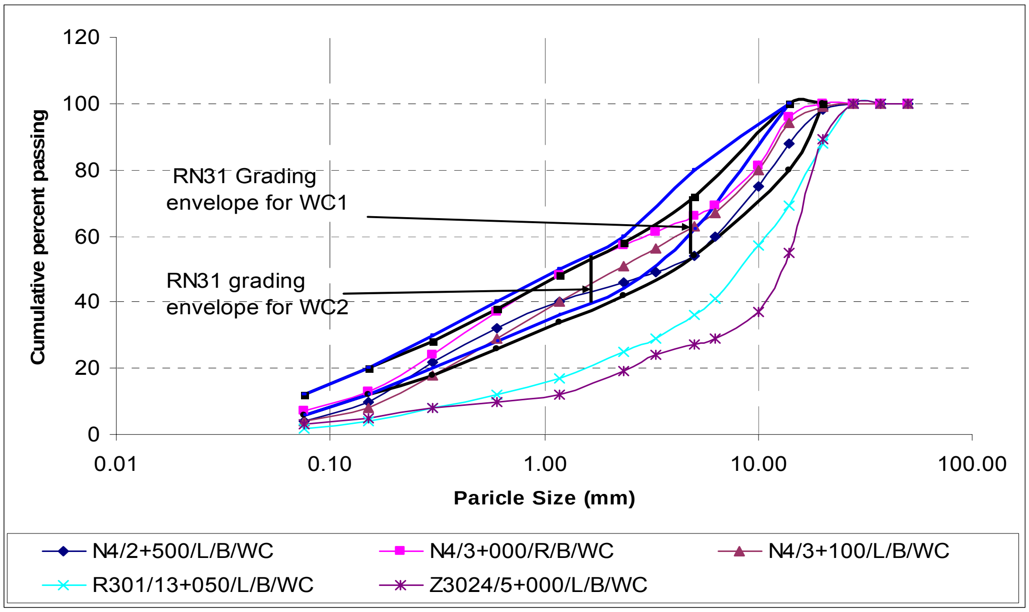

Figure 10.13: Particle size distribution chart for wearing course aggregates for three roads and gradation envelope of specifications.

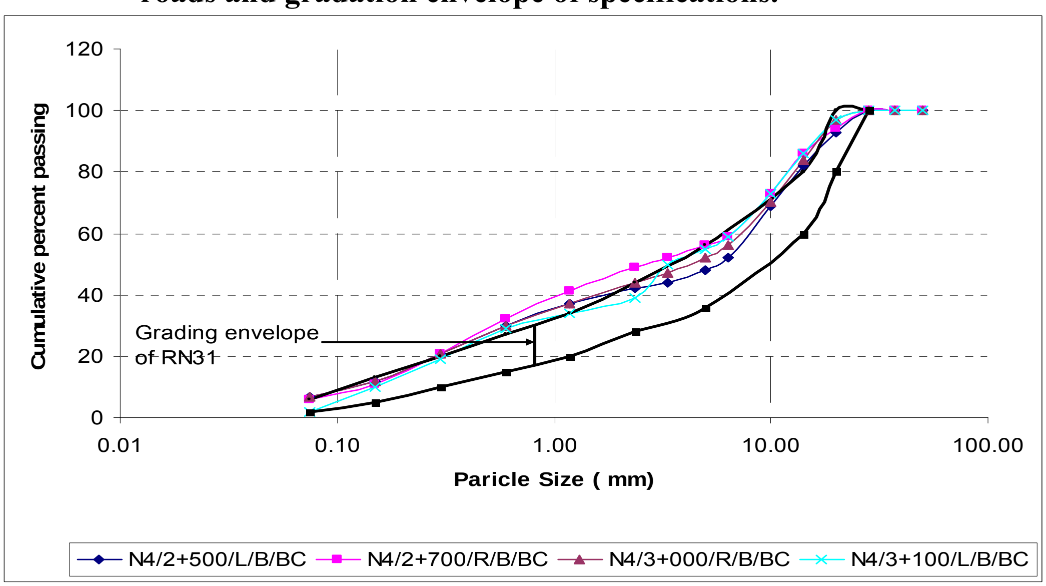

Figure 10.14: Particle size distribution chart for base course aggregates for N4 roads and gradation envelope of Road Note 31 specifications.

According to the gradation of wearing course materials for N4 road, bitumen content of that type of mix design recommended by Overseas RN31 should be 5% to 7% of total mix. But the extracted bitumen content as noted in section 10.3.2 for N4, R301 and Z3024 roads was found less than that required as per specification.

Meanwhile, according to the gradation of base course materials, bitumen content for mix designation of BC1 suggested by Overseas RN31 should be 4.8% to 6.1% of total mix but the extracted bitumen content was found 5.1% to 5.6%. So, bitumen content for binder course has satisfied the requirement suggested by RN31.

Nominal maximum sizes of aggregates for both wearing course and base course are found 28 mm (from gradation curve). Voids in the aggregates for wearing and base courses ranged from 24.7% to 28% and 23.1% to 23.6% respectively. RN31 suggested that minimum voids in the mineral aggregates for 28 mm nominal maximum particle size is 12.5%. So, the voids in the mineral aggregates are well above the specifications. This void in mineral aggregates (VMA) is very important for bituminous mixes as this should be larger enough to contain sufficient bitumen in compacted aggregates in continuous graded mixes. On the other hand, these excess voids indicate that bituminous materials are less compacted or the presence of less amount of filler materials in the mixes that affect on stability on bituminous materials.

10.3.3.3 Elongation & Flakiness Index

Shape properties of aggregates such as elongation and flakiness index play an important role on bituminous material's strength and stability. The test results for elongation and flakiness index for aggregates used in wearing and base course were found 20% to 34% and 14% to 26% respectively. However, Overseas RN 31and RHD specifications suggested that flakiness index value should be less than 45% and 30% respectively. So the test results are within the acceptable limit.

10.3.3.4 Properties of recovered Bitumen

The bituminous samples were in serviceable conditions at roads for the last two to four years. As a result the properties were changed from its original properties. The penetration and viscosity of the recovered bitumen were found 15 to 33 and 1066 [23]cst to 16965 cst respectively. The loss of heating for bituminous materials was found 0% indicates that volatile materials of bitumen were evaporated by weathering actions during the last two to four years. The properties of ductility, solubility and softening point were determined and their values ranged from 3.25 to 43.25, 98 % to 98.35% and 63^{o}C to 97^{o}C respectively. ASTM standard specifies that the values of the above mentioned properties should be minimum 100, minimum 99% and 45^{o}C to 52^{o}C respectively. So the above mentioned values do not comply with the specifications possibly due to the aging of the bitumen under serviceable conditions.

10.3.4 Strength & Stiffness Parameter of the layer

The stability and flow of the bituminous materials were determined by laboratory tests. The dynamic modulus of the bituminous layer was estimated using mixture properties are discussed in the following subsections.

10.3.4.1 Stability

The stability (adjusted) of the bituminous materials for both wearing and base course materials is shown in Table 10.3. Stability of bituminous materials increases with increasing the bitumen content up to a certain limit and again it decreases with increasing bitumen content for both wearing and base courses materials. RN31 recommended that minimum stability for materials should be 350 kg. The measured stability found from the test ranged from 272 kg to 2411 kg and 391 kg to 643 kg for N4 and R301 road respectively. However, the measured stability should be corrected due to the height of the specimen and this corrected or adjusted stability can be calculated by multiplying the measuring stability with correction factor. So the adjusted stability ranged from 714 kg to 1844.8 kg and 959 kg to 1042 kg for N4 and R301 road respectively. So, the stability found from the tests is well above the requirement of RN31.

10.3.4.2 Flow

The flow value for both wearing course and base courses is also given in Table 10.3. Results show that flow value increases with increasing bitumen content of the mixes. This value ranges from 9 mm to 14 mm and 14 mm to 19.5 mm for wearing and base

[23] cst = Centistokes

course respectively. However, RN 31 recommended that the flow value should not less than 2 mm and not more than 4 mm. So, the test results show higher flow value than the specifications which was not acceptable.

Table 10.3: Stability and Flow value for bituminous materials

Road Name	Sample number	Description	Bitumen content (%)	Stability (measured), kg	Stability (adjusted), kg	Flow, mm
N4	N4/2+500/L/B/WC&BC	WC	5.4	651	1711.9	14.0
		BC	5.63	2411	1844.8	16.0
	N4/2+700/R/B/WC&BC	WC	x	686	1685.11	12.0
		BC	5.6	1606	1244.67	19.5
	N4/3+100/L/B/WC&BC	WC	4.77	713	1192	13.0
		BC	5.1	1714	1462	16.0
	N4/3+000/R/B/WC&BC	WC	6.15	785	1962	13.0
		BC	5.4	2336	1811	18.0
	N4/2+900/L/B/WC&BC	WC	x	484	1601	13.5
		BC	x	1851	1356	17.5
	N4/3+300/R/B/WC&BC	WC	x	272	714	11.0
		BC	x	1751	1349	15.5
	N4/3+500/L/B/WC&BC	WC	x	675	1660	14.0
		BC	x	1486	1359	14.0
R301	R301/12+955/R/G/SG	WC	x	254	721	12.5
	R301/13+050/L/G/SG	WC	4.46	643	1042	14.0
	R301/13+150/R/G/SG	WC	x	391	959	14.0
Z3024	Z3024/5+000/L/B/WC	WC	4.13	111	618	9.0

Remarks: WC = Wearing course, BC = Binder course, L = Left side, B = Bituminous layer, x = test was not done
sample number: Road number / Chainage / left or Right side of pavement/ layer name/ Wearing or Binder course.

Table 10.4: Properties of aggregates extracted from bituminous materials

Road No.	Sample number	Description	ACV (%)	TFV (kN)	Elongation (%)	Flakiness (%)	Unit wt. (kg / m3)	Voids (%)	'E' for bituminous layer (Mpa)
N4	N4/2+700/R/B/WC&BC	B.C	21	150	34.00	22.	2040	23.1	1723.44

	N4/3+100/L/B/WC &BC	W.C	20	160	27	17	2000	24.7	1394.45
	N4/3+100/L/B/WC &BC	B.C	23	120	33	26	2030	23.6	1595.33
R301	R301/13+050/L/B/WC	W.C	21	150	20	14	1880	28.0	1904.39

Comments: E is the dynamic modulus of the bituminous layer is estimated using the Witczak prediction equation spelled out in correlation chapter four.

10.3.4.3 Dynamic Modulus of the bituminous layer

The dynamic modulus of the bituminous layer is estimated using the Witczak model that was stated in equation 4.5.14 of chapter four. This model uses the properties of bitumen, bitumen content and aggregates gradations. The estimated dynamic moduli of bituminous layer using the above mentioned model for N4 and R301 roads are found 1394.5 to 1904.4 MPa.

10.4 Summary

In general both the base and subbase materials partially complied with the requirement of particle size distribution suggested by RN31 and ASTM but fully complied with RHD specifications due to its wider range. In addition to this, compaction was about 8% lower for all of the locations of roads compared with the RHD specifications. Most of the CBR values for base course materials were found less than both RN31 and RHD specifications. Cyclic triaxial tests were used to ascertain resilient modulus of the materials including crushing bricks used in pavement construction. The properties of bituminous materials were evaluated for three different types of RHD roads. Due to the lack of equipment in Bangladesh, the resilient modulus of the bituminous materials was estimated using empirical correlation that found much lower value.

Chapter 11: Conclusions and Recommendations for Future Research

11.1Introduction

The main focus of this investigation was to ascertain properties of soils used in typical pavement construction and evaluate some of the important correlations used in pavement design. Conclusions drawn from the investigation are presented in terms of general conclusions together with additional specific conclusions about embankments, pavement and bituminous materials.

11.2 General Conclusions

A full suite of tests were conducted on the National, Regional and Rural (Zila) roads. In each case properties of soils, granular and bitumen materials were ascertained in accordance with internationals standard methods so that results could be compared with other researchers' findings in other parts of the world where similar conditions may exist. A summary of the results is presented in Appendix Table A - 11.1.

Limited facility is available in Bangladesh fro undertaking in-situ testing of pavement materials. The most useful tool available is the DCP. A modification of this study enable soils samples to be recovered from the DCP test for description and determinations of moisture content. This enabled a better soil profile to be developed.

Although an extensive range of tests could be conducted in Bangladesh, only a few tests could be conducted since the available apparatus number and expertise was limited.

Limited information was available regarding previous pavement and embankment designs of completed schemes.

11.3Embankment soils

Embankment subsoil (foundation soils) properties were evaluated at Tongi – Ashulia – EPZ Road (N302) showed that soft silty clay, clayey silts and black organic clayey soils existed up to the depth of 5 meters (limit to depth investigated). Some strata such as the black organic soils had high organic content and moisture content, resulting high void ratio and very low shear strength. These soils also exhibited high secondary compression index which indicates long term creep type of settlement of embankment. The investigations also showed that the embankment was made of the adjacent flood plain soils, with similar properties in many parts (i.e. low strength) same soils that found in adjacent ground surface.

The undulations of the overlying pavement confirm the variable quality of construction and the ongoing settlement. Parts of the embankment have already suffered from instability in the past.

11.4 Pavement materials

11.4.1 Pavement Subgrade soils

As noted earlier, available facility for undertaking testing for ascertaining soils properties are limited in Bangladesh. In terms of pavements, one of the most useful tests that can be conducted is the laboratory based soaked CBR tests. Results of these tests showed that the subgrade soils of the tested road were not satisfactory, in the main, for the adopted design. This will clearly lead to reduction in speed, safety of road users and increased maintenance cost.

Some of the tentative correlations, due to limited extent of investigations showed that Bangladesh specific correlations may be developed. In general they tended to follow the other researchers' findings, but with differences in specific factors. Thus although tentative, some improvements to pavement design may be possible.

11.4.2 Pavement granular materials

The granular pavement layer consists of non conventional unbound brick aggregates used in subbase as well as base layer. The soaked CBR values of subbase layer using brick aggregates were within the allowable limit specified by the different standards (Overseas Road note 31, AASHTO standard & RHD standard) but these values for base layer are not sufficiently high. Meanwhile materials gradations of subbase layer for both National & Regional highway were in compliance with the specifications but those for base layer were not fulfil the requirements fully. In general there were a number of instances where the subbase and base course properties did not comply with the relevant specifications.

11.4.3 Bituminous materials

Two layers of bituminous materials were found in national highway (high volume of traffic) whereas one layer wearing course materials were found in regional and Zilla road. The bitumen content and the properties of extracted bitumen and aggregates showed that the stiffness modulus of this layer determined by empirical correlations was found to below. This indicated that greater thickness of the bituminous layers than existed needed to be used. Both the stability and flow of the bituminous materials also did not comply with the required standards.

11.5 Recommendations

The following recommendations are suggested regarding soils and granular materials used in embankment and pavement design and construction based on the test results done on the three categories of roads under Roads and Highways department of Bangladesh:

- A database of previous, ongoing and future investigations should be created and made available to pavement engineers. This database should contain information about soils and other materials used, together with design and construction methodologies used. It should also include post design monitoring and performance information.
- The foundation soils of embankment should be removed and replaced with more competent soils or it should be stabilised.
- There is need for better quality control of both design and construction of all aspects of pavements.
- The embankment top to a suitable depth should be compacted well enough to support road pavement.
- Correlations of local soils should be developed and used in pavement design. In order for this to be done, an extensive study is required. This should lead to designs that are more pertinent to Bangladesh.
- Durability of bituminous layers needs to be addressed through better design.

11.6 Proposed Future Research

Sufficient data were not collected due to the limitation of the resources and time constrain to the researcher. Besides this, appropriate field and laboratory equipments were also unavailable. Therefore, further research is needed for details evaluation of soils and materials used in embankment and pavement design for Bangladesh.

- According to the soil classifications, Bangladesh is divided into six zones. But all field and laboratory tests were conducted on soils of Zone 3 and accordingly database was developed. Therefore details studies of engineering properties of soils for other five zone of Bangladesh have to be performed in accordance with the suggested methodology of this research. Therefore, a database have to be prepared consisting of properties of soils for all six zones of Bangladesh for future design and construction of road pavement built on embankment.
- In this study, insitu shear strength properties of subsoil's were not determined by insitu test due to the limitation of equipment at BRRL. So Standard penetration test (SPT), Cone penetration test (CPT) and Field vane shear test have to be conducted on the subsoil to determine the insitu shear strength of the soil strata for establishment of correlation between strength and index properties. These developed correlations will be used for future embankment design project at RHD.
- In Bangladesh DCP test is frequently used for estimation of CBR value. But no correlations were developed for Bangladesh soils yet. So, a relationship have to be established between DCP value and soaked CBR value for subgrade, ISG, subbase and base layer of pavement considering the effects of moisture content, dry density and soil types.
- Analytical pavement design needs resilient modulus of subgrade soils, granular and bituminous materials. In Bangladesh, resilient modulus cannot be determined due to the lack of equipment but soaked CBR value are widely used for subgrade soils. So relationships between two parameters have to be established for different classes of subgrade soils likely to be found in Bangladesh.
- The resilient modulus is dependent on effect of compaction and moisture content. As a flood prone country as well as bad workmanship the effect of compaction and moisture content as well as soaking on resilient modulus of granular materials will have to be investigated.
- Pavement rehabilitation depends on accurate measurement of deflection of different layers that can be done by FWD. So, intensive investigations with the FWD on pavement layer have to be done to characterize the insitu stress – strain

behaviour of the pavement as well as determination of backcalculated resilient modulus. Hence a relationship between the back calculated resilient modulus and laboratory measured resilient modulus will have to be established.

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APPENDIX – A: The availability and deposits of Granular materials used in Road construction

1. **Boulders, gravels, pebbles and shingles**

There are some renewable boulders and gravels quarries in the north and northeast border of the country. At the northern parts, the materials are collected by divers using buckets to dredge the Dharla river bed or by hand during the low water level. The deposits are replenished during the monsoon also known as channel lag deposits. The deposits ranges in thickness from 10 cm to 100 cm in thickness and probably the deposits are pebbles and gravels.

The north east border of Bangladesh named Sylhet region is the most important road materials source in Bangladesh. In that area, majority of hard rocks, limestone, gravels along with coarse sands are found and some black and white types of rocks are found.

2. **Limestone**

The limestone is deposited in the north, North West and a southern island of Bangladesh. These areas are known as Takerghat, Bangaghat, Jaflong, Joypurhat Bagalibazar and St Martin Island. The deposit of lime stone in Bangladesh is about 14.38 m.cum (GSB, 1980). At present only Tekerghat quarry is under lime production and annual production of this project is about 43000 to 62000 tonnes. The bulk amount of reserve is at Joupurhat and Bagalibazar amounting 130M.tonnes which are 400 to 580 m and 60 – 70 m below the ground surface respectively (Humphreys et al., 1994).

Another lime stone quarry is in Jaflong which is about 30 m long, 10m wide and consists of three faces; two are 10 – 12 m in height with the third is only around 5 m height (GSB, 1994).

The south east island of Bangladesh known as St. Martins Island, situated in the Bay of Bangal is composed of grey to whitish grey massive shelly limestone. The shelly limestone is roasted locally for lime and it is estimated that there are reserves of approximately 1.8 million tonnes (GSB, 1980) available in the island.

3. **Hard rock**

Hard rock is deposited in the northern district named Dinajpur of Bangladesh. A joint venture project between Bangladesh and North Korea to extract the hard rock from underground reserve is known as Madhapara hard rock project. This hard rock consists of granodiorite, quartzdiorite and gneiss has been discovered at a shallow depth of 128 m. Besides this, hard rock are also deposited in Ranipukur and Pirgong in Rangpur district at a depth of 171 m and 265 m respectively and from Bogra, Joypurhat – Jamalgonj and Kansat of Rajshahi district at depths of 2150m, 600 – 667 m and 615 m respectively (Rahman, 1997). The Madhapara hard rock has been used as construction materials for commercial buildings, housing apartments, construction of bridges, Dams, river dykes, railway ballast river erosion and manufacturing of concrete sleepers and road construction purposes.

4. Sands

As noted earlier, there are many of rivers in Bangladesh and most of the rivers come from neighbouring countries and their source of origins are hills and mountains. Every year during the rainy season, huge amount of sands have been carried from India and deposited in Bangladesh. But in dry session, water levels in the most of the rivers drop considerably. Sands can be collected from the river beds and bank at low water. Sands are used as filler in bituminous materials. It is also widely used in different road layers. The fineness modulus (F.M) of sand found from north part of the country is about 2 to 2.8 which are the most useful in all construction purposes.

5. Stabilized materials

Stabilisation of soils is necessary when the available subgrade soil has insufficient bearing capacity to support the pavement. This stabilization may be done using either mechanically and chemically or both. The most commonly used stabilizing agents used in Bangladesh are cement, lime, fly ash (FA), rice husk ash (RHA), cement kiln dust (CKD) and fibres from industries.

APPENDIX – B: Correlations for Soils and Materials

1. SPT value related to the cohesionless soils

Mayerhof (1956) suggested the following approximate equations for computing the drained angle of friction from the relative density (D_r)

For granular soil with fine sand and more than 5% silt

ϕ'=25 + 0.15 D_r

For granular soil with fine sand and less than 5% silt

ϕ'=30 + 0.15 D_r

Where D_r is expressed in percent and ϕ' is the drained angle of friction.

Again, the angle of friction of granular soils, ϕ' has been correlated to the standard penetration number. Peck, Hanson and Thornburn (1974) suggested a correlation between N_{cor} and ϕ' in a graphic form, which can be approximated as follows (Wolff,1990):

ϕ' (deg) = 27.1 + 0.3 N_{cor} - 0.00054 N^2_{cor}

Where, N_{cor} is the corrected penetration number.

The standard penetration number is a useful guideline in soil exploration and assessment of subsoil conditions, provided the results are interpreted correctly. All the equations and correlations relating to the standard penetration numbers are approximate; because soil is not homogeneous, a wide variation in the penetration number obtained from the field (N_f). For soil deposits that contain large boulders and gravel, N_f may be erratic.

2. Relationship between Unconfined Compressive Strength and CBR value

The Unconfined Compressive Strength (UCS) is often used as a mean of soil improvement whereas the California Bearing Ration (CBR) is normally used for design of pavement.

Therefore, Kitazume and Satoh (2002) had carried out a series of laboratory test to determine the relationship between the CBR value and UCS value (q_u) and the proposed relationship is as follows:

q_u (kN/ m^2) = 26.7 CBR

3. Relationship between Undrained shear strength and CBR value

The strength of clayey subgrade soil can be characterised in terms of fundamental parameters that is used in the improved methods of pavement design. The strength parameter of cohesive subgrade soil has related to undrained shear strength by Lister (1970), Division head of the pavement design division of Highways Department of TRRL. For development of the following correlations, numbers of tests were performed by penetrometer on the remoulded clay and undisturbed over consolidated clay.

For remoulded soils

Cu = 23 x CBR

For undisturbed overconsolidated soils

Cu = 11.5 x CBR

Where, Cu is in kPa.

Besides these, Carter and Bentley (1991) have derived an empirical equation for saturated clay in undrained conditions is given as follows

Cu = 11.11 x CBR

Where, Cu is in kPa.

APPENDIX – C: Sample Numbering & labeling Convention for soils:

Bore holes number / location of bore holes (chainage) / depth of collected soil samples. Bore hole number is identified as bore hole number one = BH – 01, bore hole number two = BH – 02, bore hole number three = BH – 03 and bore hole number four = BH – 04 And location of bore hole is identified by chainage of the road. The depth of the collected soil samples is measured from ground surface in meter. A typical examples of soil sample collected from 3 meter depth at bore hole number two located at chainage 6+200 is denoted by BH – 02/ C6200/D- 3.

APPENDIX – D: Numbering & labelling convention for granular & bituminous materials and subgrade soils:

The procedure for numbering and labeling of samples have clearly written on the sample bag and on a label to be put inside the sample bag with permanent marker using the following convention:

However, numbering and labeling of samples have clearly written on the sample bag and on a label to be put inside the sample bag with permanent marker using the following convention:

Road Number / chainage / lane / material / layer

- Chainage is measured by km from start of the road towards end point direction.

- Lane will be either left, right or single (“L”, “R”, or “S”) facing in the direction of increasing chainage
- Material will be either “B” (bituminous),“G” (granular), ISG (improved subgrade) and “S” (subgrade)
- Layer will be:

WC – bituminous wearing course, BC – bituminous base course, RB – granular road base, HBB – herring bone brick, SB – granular sub-base, ISG – improved sub-grade, SG - sub-grade

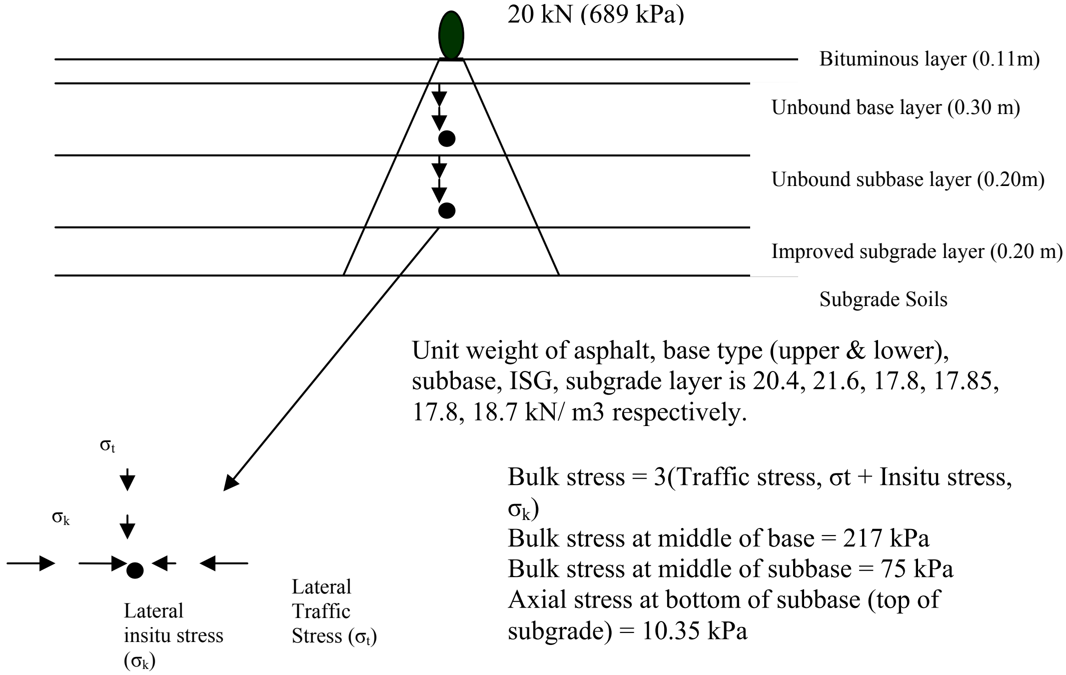
A typical reference would therefore be: **N4/3+000/R/G/RB.** The samples should be clearly labeled in accordance with the convention mentioned above and transported to the laboratory at the end of each day’s work.

APPENDIX – E: Description of the DCP sampler:

A new soil sampler was made first time to collect soil sample after doing the DCP test to determine the soil moisture content. The outer & inner diameter of the sampler is 18 mm & 12 mm respectively. The length of the soil sampler is 65 mm. The sampler is joined with 55mm long end threaded rod, one end is joined with sampler and another end is joined with DCP penetration rod.

APPENDIX – F

Calculation of insitu and traffic stresses for a typical pavement section under standard wheel load:



Unit weight of asphalt, base type (upper & lower), subbase, ISG, subgrade layer is 20.4, 21.6, 17.8, 17.85, 17.8, 18.7 kN/ m3 respectively.

Bulk stress = 3(Traffic stress, σt + Insitu stress, σ_k)
Bulk stress at middle of base = 217 kPa
Bulk stress at middle of subbase = 75 kPa
Axial stress at bottom of subbase (top of subgrade) = 10.35 kPa

APPENDIX –G: Appendix Tables

Table A - 2.1: Selection of methods of boring and sample collection

Criteria	Wash boring			Cable percussion boring			Hand auger		
	Weigh tage (%)	Priorit- y (0- 5)	Score	Weightag e (%)	Priority (0- 5)	Score	Weig- htage (%)	Priori -ty (0- 5)	Score
Accuracy	100	3	300	100	5	500	100	3	300
Availability	100	5	500	100	5	500	100	5	500
Available expertise for carrying out test	50	4	200	50	0	0	50	4	200
Cost	75	3	225	75	2	150	75	5	375
Easy to use	50	4	200	50	3	150	50	3	150
Training	100	4	400	100	4	400	100	4	400
	Total Score		1825	Total Score		1700	Total Score		1925
Priority Position	2nd			3 rd			1st		
Remarks: 0 = lowest priority and 5 = highest priority. Lower priority is attributed to expensive test.									

Table A- 2.2: Selection for the method of CBR value

Criteria	DCP test and correlations			Field CBR test		
	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score
Accuracy	100	2	200	100	4	400
Availability	100	5	500	100	0	0
Available expertise for carrying out the test	50	4	200	50	3	150
Cost	75	4	300	75	2	150
Easy to use	50	4	200	50	3	150
Training	100	5	500	100	2	200
	Total Score		1900	Total Score		1050
Priority Position	1st			2nd		

Table A- 2.3: Selection of methods of insitu density measurement

Criteria	Sand replacement method			Core cutter method			Nuclear density method		
	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score
Accuracy	100	5	500	100	4	400	100	4	400
Availability	100	5	500	100	3	300	100	0	0
Available expertise for carrying out	50	4	200	50	4	200	50	0	0
Cost	75	4	300	75	2	150	75	3	225
Easy to use	50	4	200	50	3	150	50	5	250
Training	100	4	400	100	4	400	100	0	0
	Total Score		2100	Total Score		1600	Total Score		875
Priority Position	1st			2nd			3rd		

Table A- 2.4: Determination of the method of pavement deflection

Criteria	Benkleman Beam			Dynaflect and Road Rater			FWD		
	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score
Accuracy	100	3	300	100	4	400	100	5	500
Availability	100	5	500	100	0	0	100	0	0
Available expertise for carrying out the test	50	4	200	50	3	150	50	3	150
Cost	75	3	225	75	3	225	75	2	150
Easy to use	50	4	200	50	1	50	50	2	100
Training	100	5	500	100	0	0	100	0	0
	Total Score		1925	Total Score		825	Total Score		900
Priority Position	1st			3rd			2nd		

Table A- 2.5: Selection of methods of undrained shear strength (Cu)

Criteria	Field vane shear test			SPT		
	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score
Accuracy	100	3	300	100	3	300
Availability	100	0	0	100	5	500
Available expertise for carrying out the test	50	3	150	50	4	200
Cost	75	2	150	75	2	150
Easy to use	50	3	150	50	3	150
Training	100	0	0	100	4	400
	Total Score		750	Total Score		1700
Priority Position	4th			3rd		

Cont…

Criteria	Quick undrained Triaxial test			UCS test		
	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score
Accuracy	100	5	500	100	3	300
Availability	100	5	500	100	5	500
Available expertise for carrying out the test	50	2	100	50	4	200
Cost	75	3	225	75	4	300
Easy to use	50	3	150	50	5	250
Training	100	5	500	100	5	500
	Total Score		1975	Total Score		2050
Priority Position	2nd			1st		

Table A- 2.6: Selection of methods of drained cohesion and angle of internal friction

Criteria	Direct shear box test			CU Triaxial test with pore pressure measurement		
	Weightage (%)	Priority (0- 5)	Score	Weightage (%)	Priority (0- 5)	Score
Accuracy	100	2	200	100	5	500
Availability	100	4	400	100	3	300
Available expertise for carrying out the test	50	4	200	50	0	0
Cost	75	3	225	75	2	150
Easy to use	50	4	200	50	3	150
Training	100	4	400	100	4	400
	Total Score		1625	Total Score		1500
Priority Position			1st			2nd

Table A - 4.1: Empirical correlations suggested by various authors

Suggested equation(s)	Soil type(s)	Reference	Comments
$q_u = 0.58\ p_a \times N^{0.72}$ Where, p_a atmospheric pressure and q_u and pa have the same unit.	Sandy clay, silty clay, Chicago clay and clay with low to high plasticity of soil.	Kulhawy & Mayne (1990)	Based on theoretical data
$q_u = k\ N_{cor}$ kPa Where k is the proportionality factor and its value = 12 recommended by Bowles (1996).	Saturated cohesive soil of very soft to hard consistency.	Peck et al. (1990)	-
$q_u = f \times N$ (kpa) Where, f 'value ranging from 13.33 to 25.00	Different types of soils	Sanglerat (1972)	-
f = 25,	For clay		
f = 20	silty clay		
f = 13.33	Silty sandy soil		
$q_u = f \times N$ (kpa)	Bangladesh soils	Serajuddin and Chowdhury (1996)	Based on laboratory data
Where, f =16	Clay & silt of high plasticity with LL ≥ 51%		
f = 15	Clay & silt of medium plasticity with LL = 36 – 50%		
f = 13	Clay & silt of low plasticity with LL ≤ 35%		
$q_u = f\ .\ N$ (kPa Where, f = 12.50 for very soft to soft clay & f = 13.33 for medium stiff to hard clay	Cohesive fine grained soils	Terzaghi and Peck (1948 & 1967)	-

Cont…

Suggested equation(s)	Soil type(s)	Reference	Comments
$q_u = N / 2.5$, ksf for silty clay $= N / 2$, ksf for clay.	For silty clay and clayey soil.	Mc Earthy (1977)	Based on laboratory data
$q_u = N / 4$ (ksf)	-	Bowles (1988)	
$qu = N / 2.77$ (ksf)	Dhaka clay (Dhaka metropolitan area)	Ullah (2001)	Based on laboratory data
q_u (kPa) = 25 N q_u (kPa) = 15 N q_u (kPa) = 7.5 N	For highly plastic clay, Medium plastic clay, Low plastic clay & silt,	Sowers (1979)	-
$q_u = 24$ N	clayey soils	Nixon (1982)	-
q_u (kPa) = 10 to 20 N	Preconsolidated silty clay in Farakka west Bengal, India	Murthy (1993)	Based on laboratory test.
$q_u = 9.7\ N_{field}$, r = 0.83 , n= 113 $q_u = 13.63N_{60}$, r = 0.80 Where, n is the number of data	For Highly plastic clay (CH),	Sivrikaya and Togrol (2002)	Based on large number of statistical data
$q_u = 6.7\ N_{field}$, r = 0.76 $q_u = 9.85\ N_{60}$, r = 0.73	For low plastic clay (CL),		
$q_u = 8.64\ N_{field}$, r = 0.80, n = 226, $q_u = 12.36\ N_{60}$, r = 0.78	For fine grained soil		
$q_u = (0.19PI + 6.2)\ N_{60}$, r = 0.80,n = 30 $N_{60} <25$	For fine grained soil,		

Table A- 4.2: Correlation between Ncor & unconfined compressive strength (Peck et al. 1974)

Consistency	Ncor	q_u (kPa)
Very soft	0 -2	<25
Soft	2- 4	25 -50
Medium	4 – 8	50 – 100
Stiff	8 – 15	100 – 200
Very stiff	15 – 30	200 – 400
Hard	>30	>400

Table A- 7.1: Trial Pit Log sheet (Ch 2+700)

Samples & in situ tests			Pit size: 0.75 m x 0.75 m x 1.15m), Offset from CL: 2.90 m	Location: Ch 2+700 Trail Pit: (TP/N4/C2700/R)	
Depth (m)		Sample Number	Description of strata	Legend	Depth (m)
Start	End				
0.00	0.13	N4/2+700/R/B /WC + BC	Bituminous layer (Stone aggregate mix with bitumen)		0.13
0.13	0.27	N4/2+700/R/G /RB1	Aggregate base layer - 1 (Stone aggregate mixed with sand)		0.14
0.27	0.40	N4/2+700/R/G /RB2	Aggregate base layer - 2 (Brick aggregate mixed with sand)		0.13
0.40	0.65	N4/2+700/R/G /SB	Sub base layer (Brick aggregate mixed with sand)		0.25
0.65	0.72	N4/2+700/R/G /EGL	Existing granular layer (Brick aggregate mixed with sand)		0.07
0.72	1.09	N4/2+700/R/G / ISG	Local sand (FM <1.0)		0.37
1.09	Cont...	N4/2+700/R/S /SG	Brown soft clay		

Note: Sample Number: Road No/ Chainage/ Left or Right side/ Layer type/ Layer name

Layer type: B – Bituminous, G – Granular

Layer Name: WC – Wearing course, BC – Binder course, RB – Road Base, SB – Subbase, ISG – Improved Subgrade, SG – Subgrade

Table A -7.2: Trial Pit Log sheet (Ch. 2+900)

Samples & in situ tests			Pit size: 0.75 m x 0.75 m x 1.30m) Offset from CL: 2.90 m	Location: Ch 2+900 Trail Pit: (TP/N4/C2900/R)	
Depth (m)		Sample Number	Description of strata	Legend	Depth (m)
Start	End				
0.00	0.13	N4/2+900/L/B /WC + BC	Bituminous layer (Stone aggregate mix with bitumen)		0.13
0.13	0.32	N4/2+900/L/G /RB1	Aggregate base layer - 1 (Stone aggregate mixed with sand)		0.19
0.32	0.48	N4/2+900/L/G /RB2	Aggregate base layer - 2 (Brick aggregate mixed with sand)		0.16
0.48	0.71	N4/2+900/L/G /SB	Sub base layer (Brick aggregate mixed with sand)		0.23
0.71	1.3	N4/2+900/L/G /ISG	Local sand (FM < 1.0)		0.59
1.3		N4/2+900/R/G / ISG	Local sand (FM <1.0)		

Table A- 7.3: Trial Pit Log sheet of R301 (Ch. 13+050)

Samples & in situ tests			Pit size: 0.75 m x 0.75 m x 0.62 m) Offset from CL: 2.00 m	Location: Ch 13+050 Trail Pit: (TP/R301/C13050/L)	
Depth (m)		Sample Number	Description of strata	Legend	Depth (m)
Start	End				
0.00	0.055	R301/ 13+050/L/B /WC	Bituminous layer (Stone aggregate mix with bitumen)		0.055
0.055	0.175	R301/ 13+050/L/G /RB	WBM base layer (Brick aggregate only)		0.12
0.175	0.265	-	Scarified and compacted existing bituminous layer		0.09
0.265	0.345	R301/ 13+050/L/G /SB	Sub base layer (Brick aggregate mixed with sand)		0.08
0.345	0.495	R301/ 13+050/L/G/ISG	Local sand (FM < 1.0)		0.15
0.495		R301/ 13+050/L/G /SG	Fine gray soil		

Table A- 7.4: Trial Pit Log sheet of Z3024 (Ch 4+820)

Samples & in situ tests			Pit size: 0.75 m x 0.75 m x 0.60 m) Offset from CL: 0.45 m	Location: Ch 4+ 820 Trail Pit: (TP/Z3024/C4820/R)	
Depth (m)		Sample Number	Description of strata	Legend	Depth (m)
Start	End				
0.00	0.025	Z3024/ 4+820/R/B /WC	Bituminous layer (Stone aggregate mix with bitumen)		0.025
0.025	0.240	Z3024/ 4+820/R/G /RB	WBM base layer (Brick aggregate only)		0.215
0.240	0.315	-	Brick flat soiling (1^{st} layer)		0.075
0.315	0.325	-	Local sand used for levelling between two soling layer)		0.01
0.325	0.400	-	Brick flat soiling (2^{st} layer)		0.075
0.400		Z3024/ 4+820/R/ SG	Gray fine soil		

Table A- 8.1: Strata encountered for soils collected from different boreholes & their classifications

Borehole number	**Depth from EGL**	**Strata encountered**	**Soil classification by USCS**	
BH – 02 (Ch. 6+200)	0.5	Black brownish clay soil	CL	Lean clay
	1	Brownish clayey soil	CL	Lean clay with sand
	1.5	Brownish clayey soil	CL	Lean clay with sand
	2	Gray clayey soil	CH	Fat clay with sand
	2.5	Blackish clayey soil	OH	Organic silt
	3	Grayey clay soil	CL	Lean clay with sand
	3.5	Black organic clay soil	OH	Organic silt with sand
	4	Black organic clay soil	OL	Organic silt
	4.5	Black gray clayey soil	MH	Elastic silt with sand
	5	Black grayey clayey soil	CL	Lean clay with sand
	5.5	Gray clayey soil	CL	Lean clay with sand

Cont…

Borehole number	Depth from EGL	Strata encountered	Soil classification by USCS	
BH – 03 (Ch. 6+480)	0.5	Brownish sandy clay soil	ML	Silt with sand
	1	Blackish grayey clay soil	MH	Elastic silt
	1.5	Gray clay soil	CL	Lean clay
	2	Gray clay soil	CL	Lean clay
	2.5	Blackish organic clay soil	OH	Organic clay
	3	Grayey clay soil	ML	Silt with sand
	3.5	Gray clay soil	ML	Silt with sand
	4	Gray clay soil	CL	Lean clay with sand
	4.5	Black clay soil	OL	Organic silt with sand
	5	Black organic clay soil	Peat	Pt
	5.5	Black organic clay soil	OH	Organic silt
BH – 04 (Ch. 6+600)	0.5	Brownish sandy clay soil	CL	Lean clay with sand
	1	Gray sandy clay soil	CL	Lean clay with sand
	1.5	Brownish gray clay soil	CL	Lean clay with sand
	2	Blackish gray clay soil	CL	Lean clay
	2.5	Blackish organic clay soil	OH	Organic silt with sand
	3	High organic black clay soil	Peat	Pt
	3.5	Gray clay soil	ML	Silt with sand
	4	Gray clay soil	ML	Silt with sand
	4.5	Black clay soil	OH	Organic silt with sand
	5	Black organic clay soil	OH	Organic clay with sand

Table A- 8.2: Compressibility and Permeability properties of soils

BH No.	Sample No.	Pressure (kPa)	Void ratio	Coefficient of volume compressibility (m ^{2} / MN)	Coefficient of Consolidation(from t_{50}) (m^2/yr)	Coefficient of Consolidation(from t_{90}) (m^2/yr)	Permeability (m /s)
2	BH-2/C6200/D-0.5	12.5	1.102	3.047	1.123	1.120	1.06E-09
		25	1.072	1.164	0.462	0.461	1.67E-10
		50	1.006	1.277	0.739	0.777	3.08E-10
		100	0.937	0.689	1.386	1.611	3.44E-10
		200	0.844	0.479	2.034	2.061	3.06E-10
		50	0.864	0.072	2.444	1.884	4.23E-11
		200	0.841	0.081	5.828	11.760	2.96E-10
		400	0.749	0.258	1.370	1.229	9.83E-11
	BH-2/C6200/D-4	12.5	1.23	1.93	26.86	57.34	3.42E-08
		25	1.19	1.30	12.89	39.63	1.6E-08
		50	1.13	1.17	13.85	26.28	9.56E-09
		100	1.07	0.52	11.64	29.56	4.8E-09
		200	1.01	0.30	21.98	23.46	2.19E-09
		50	1.03	0.06	7.75	8.27	1.59E-10
		200	1.01	0.07	9.56	14.69	2.98E-10
		400	0.94	0.18	10.28	9.24	5.09E-10
	BH-2/C6200/D-5	12.5	0.74	3.78	0.44	0.43	5.1E-10
		25	0.70	2.03	0.43	0.42	2.65E-10
		50	0.63	1.53	1.31	1.39	6.6E-10
		100	0.56	0.90	1.20	1.28	3.57E-10
		200	0.49	0.48	2.11	2.25	3.37E-10
		50	0.50	0.08	2.56	2.29	5.39E-11
		200	0.49	0.07	13.40	16.59	3.65E-10
		400	0.42	0.22	0.69	0.65	4.42E-11
	BH 03/ C6480/ D - 2.5	12.5	1.04	2.59	6.64	10.20	8.18E-09
		25	1.02	0.93	0.45	0.42	1.22E-10
		50	0.97	0.92	0.50	0.41	1.17E-10
		100	0.90	0.77	2.07	2.21	5.26E-10
		200	0.83	0.37	3.92	3.11	3.54E-10
		100	0.83	0.03	9.16	14.08	1.35E-10
		50	0.84	0.08	9.23	6.03	1.46E-10

Cont…

		100	0.83	0.06	9.23	20.46	4.03E-10
		200	0.82	0.07	1.53	2.90	6.16E-11
		400	0.73	0.26	3.58	3.33	2.66E-10
	BH 03/ C6480/ D - 4	12.5	1.21	4.83	1.03	0.69	1.03E-09
		25	1.15	2.13	1.47	1.34	8.86E-10
		50	1.07	1.56	1.38	1.21	5.83E-10
		100	0.99	0.78	1.66	1.55	3.73E-10
		200	0.89	0.50	1.68	1.70	2.63E-10
		100	0.90	0.04	7.89	6.82	9.19E-11
		50	0.91	0.11	5.34	6.89	2.34E-10
		100	0.90	0.08	6.47	5.22	1.25E-10
		200	0.88	0.10	5.27	4.36	1.31E-10
		400	0.79	0.26	1.86	1.46	1.17E-10
	BH 03/ C6480/ D - 5	12.5	0.83	4.41	1.98	1.84	2.52E-09
		25	0.82	0.61	1.69	1.73	3.27E-10
		50	0.77	0.96	6.55	6.99	2.08E-09
		100	0.73	0.51	11.78	8.05	1.26E-09
		200	0.67	0.32	7.11	13.50	1.36E-09
		50	0.68	0.04	14.12	13.13	1.5E-10
		200	0.67	0.05	19.17	17.44	2.82E-10
		400	0.61	0.17	10.35	12.57	6.48E-10
4	BH 04/ C6600/ D - 1.5	12.5	1.04	3.55	0.59	0.57	6.27E-10
		25	1.00	1.53	0.83	0.67	3.18E-10
		50	0.94	1.18	0.93	1.18	4.35E-10
		100	0.87	0.76	0.87	0.93	2.17E-10
		200	0.78	0.45	1.15	1.02	1.41E-10
		100	0.79	0.04	14.09	5.57	7.65E-11
		50	0.80	0.12	0.68	0.64	2.4E-11
		100	0.79	0.08	4.85	6.76	1.74E-10
		200	0.78	0.10	0.88	0.82	2.42E-11
		400	0.70	0.30	1.29	1.24	1.14E-10
	BH 04/ C6600/ D - 2	12.5	1.015	5.457	0.744	0.710	1.2E-09
		25	0.960	2.197	0.713	0.916	6.24E-10
		50	0.902	1.189	1.608	1.570	5.79E-10
		100	0.800	1.070	1.144	1.221	4.05E-10
		200	0.702	0.546	1.111	1.010	1.71E-10

Cont…

		100	0.708	0.034	3.59	3.552	3.76E-11
		50	0.718	0.118	1.87	1.810	6.6E-11
		100	0.714	0.044	7.43	7.267	9.95E-11
		200	0.696	0.102	2.93	2.879	9.07E-11
		400	0.598	0.304	1.574	1.443	1.36E-10
	BH 04/ C6600/ D - 3	12.5	1.18	2.72	24.74	22.50	1.9E-08
		25	1.17	0.40	7.07	9.14	1.12E-09
		50	1.13	0.69	23.26	24.82	5.27E-09
		100	1.08	0.44	16.43	22.03	2.98E-09
		200	1.01	0.37	21.06	19.15	2.22E-09
		50	1.02	0.04	24.47	15.99	2.05E-10
		200	1.00	0.06	11.43	18.48	3.67E-10
		400	0.92	0.19	27.89	46.51	2.75E-09

Table A - 8.3: Calculation of Cv with Casagrande's method and Taylor's method

Sample description	Pressure	Casagrande's Method (Logarithm of time fitting method.		Taylor Method (Square root of time fitting method)	
Bore hole number/ depth	(kPa)	t_{50} (min)	Cv (m^2/ year)	t_{50} (min)	Cv (m^2/year)
BH -03/D-4 (Organic soil)	12.5	11	0.844	9	1.032
	25	8.5	0.999	5.76	1.474
	50	4.5	1.765	5.76	1.379
	100	4.6	1.595	4.41	1.664
	200	3.5	1.915	4	1.676
	400	2.9	2.083	3.24	1.864
BH – 03/D – 5 (Inorganic soil)	12.5	4.25	2.055	4.41	1.981
	25	5.5	1.488	4.84	1.691
	50	1.6	4.955	1.21	6.552
	100	1.2	6.285	0.64	11.785
	200	1	7.114	1	7.114
	400	2.1	3.154	0.64	10.326

Table A- 9.1: Calculation of CBR from DCP test for Subgrade as well as granular layer

Road No	Location	Layer name	DCP value (mm / blows)	CBR (%) using Kleyn & Van Hearden graph	CBR (%) using Harison's equ. (1987)	CBR (%) Log - log Model	CBR (%) using Riley et al. (1987) equation	Recommended CBR (%)
N4	Ch 2+500 (L)	Total granular layer	3	100	103.20	123.58	65.65	65.65
		Base type -i	7.33	30	37.63	52.42	22.88	22.88
		Base type -ii	2	100	158.70	182.39	105.92	100
		Subbase	2.97	100	104.32	124.78	66.43	66.43
		Imp. Subgrade	3	100	154.30	203.96	65.65	65.65
	Ch 2+500 (R)	Total granular layer	1.16	100	279.27	307.69	201.44	100
		Base type -i	2.14	100	147.81	170.92	97.80	97.8
		Base type -ii	0.671	100	488.47	520.41	384.31	100
		Subbase	2.83	100	109.87	130.70	70.32	70.32
		Imp. Subgrade	1.56	100	312.15	547.50	142.01	100
	Ch 2+700(L)	Total granular layer	0.81	100	403.31	434.36	307.75	100
		Base type -i	2	100	158.70	182.39	105.92	100
		Base type -ii	0.96	100	339.08	368.99	251.84	100
		Subbase	0.55	100	597.65	629.87	485.94	100
		Imp. Subgrade	2.4	100	197.05	285.68	85.42	85.42
	Ch 2+ 700(R)	Total granular layer	2.26	100	139.55	162.20	91.70	91.70
		Base type -i	2.33	100	135.12	157.52	88.46	88.46
		Base type -ii	2.93	100	105.85	126.42	67.50	67.5
		Subbase	1.56	100	205.66	231.53	142.01	100
		Imp. Subgrade	21.47	9	7.19	10.45	6.44	6.44
		Subgrade	36.83	4.2	5.77	5.54	3.40	3.4
	Ch 2+900 (L)	Total granular layer	2.63	100	118.82	140.23	76.68	100
		Base type -i	3.16	95	97.58	117.57	61.74	61.74

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		Base type -ii	2	100	158.70	182.39	105.92	100
		Subbase	3.47	80	88.17	107.47	55.29	55.29
		Imp. Subgrade	5.89	42	70.40	73.64	29.61	29.61
	Ch 2+900(R)	Total granular layer	0.98	100	332.00	361.76	245.79	100
		Base type -i	1.08	100	300.53	329.54	219.17	100
		Base type -ii	0.71	100	461.21	492.94	359.52	100
		Subbase	1.29	100	250.34	277.87	177.71	100
		Imp. Subgrade	1.88	100	256.17	413.07	113.95	100
	Ch 2+920(L)	Total granular layer	0.94	100	346.46	376.53	258.18	100
		Base type -i	0.76	100	430.36	461.76	331.78	100
		Base type -ii	0.88	100	370.61	401.14	279.08	100
		Subbase	1.5	100	214.20	240.41	148.74	100
		Imp. Subgrade	1.9	100	253.30	406.52	112.53	100
	Ch 2+920(R)	Total granular layer	0.96	100	339.08	368.99	251.84	100
		Base type -i	0.79	100	413.72	444.92	316.96	100
		Base type -ii	0.93	100	350.26	380.41	261.46	100
		Subbase	1.5	100	214.20	240.41	148.74	100
		Imp. Subgrade	1.81	100	266.73	437.43	119.17	100
	Ch 2+940(L)	Total granular layer	1	100	325.20	354.81	240.00	100
		Base type -i	1.13	100	286.89	315.53	207.77	100
		Base type -ii	0.56	100	586.84	619.07	475.72	100
		Subbase	1.92	100	165.64	189.68	111.15	100
		Imp. Subgrade	2.8	100	166.51	226.36	71.21	71.21
	Ch 2+940(R)	Total granular layer	0.96	100	339.08	368.99	251.84	100
		Base type -i	0.82	100	398.30	429.28	303.33	100
		Base type -ii	0.76	100	430.36	461.76	331.78	100
		Subbase	1.55	100	207.04	232.96	143.09	100
		Imp. Subgrade	2.08	100	229.93	354.59	101.13	100
N4	Ch 2+960(	Total granular layer	1	100	325.20	354.81	240.00	100

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	L)	Base type -i	1.2	100	269.70	297.84	193.54	100
		Base type -ii	0.5	100	658.20	690.22	543.78	100
		Subbase	1.64	100	195.25	220.67	133.87	100
		Imp. Subgrade	3	100	154.30	203.96	65.65	65.65
	Ch 2+960(R)	Total granular layer	0.98	100	332.00	361.76	245.79	100
		Base type -i	0.63	100	520.77	552.88	413.99	100
		Base type -ii	1	100	325.20	354.81	240.00	100
		Subbase	1.6	100	200.33	225.97	137.83	100
		Imp. Subgrade	2.3	100	206.34	304.64	89.82	89.82
	Ch 2+980(L)	Total granular layer	1	100	325.20	354.81	240.00	100
		Base type -i	1.35	100	238.87	266.00	168.43	100
		Subbase	1.69	100	189.24	214.40	129.21	100
		Imp. Subgrade	2.67	100	175.43	243.20	75.32	75.32
	Ch 2+980(R)	Total granular layer	0.93	100	350.26	380.41	261.46	100
		Base type -i	0.54	100	608.87	641.06	496.58	100
		Base type -ii	1	100	325.20	354.81	240.00	100
		Subbase	1.67	100	191.60	216.87	131.04	100
		Imp. Subgrade	2.5	100	188.50	268.60	81.40	81.4
N302	Ch 6+000	Base	3.2	90	96.26	116.16	60.83	60.83
		Subbase	6.178	38	46.10	61.77	27.99	27.99
		Imp. Subgrade	12.7	17	23.69	23.08	11.96	11.96
		Subgrade	20.67	9	11.23	10.82	6.73	6.73
	Ch 6+200	Base	3.57	80	85.48	104.58	53.46	53.46
		Subbase	6.45	37	43.83	59.27	26.60	26.6
		Imp. Subgrade	20	9	8.95	11.63	7.00	7
		Subgrade	21.5	8.5	10.75	10.34	6.43	6.43
	Ch 6+400	Base	3.61	78	84.44	103.46	52.77	52.77
		Subbase	6.73	35	41.68	56.90	25.30	25.3
		Imp. Subgrade	15	14.5	17.50	17.95	9.83	9.83
		Subgrade	20.33	8.4	11.43	11.03	6.86	6.86
	Ch 6+440	Base	3.04	98	101.74	122.02	64.63	64.63
		Subbase	5.75	42	50.11	66.18	30.47	30.47
		Imp. Subgrade	9.23	25	38.88	37.37	17.43	17.43

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		Subgrade	20.5	9	11.33	10.92	6.80	6.8
	Ch 6+520	Base	1.85	100	172.20	196.57	116.13	100
		Subbase	4.06	70	74.22	92.43	45.94	45.94
		Imp. Subgrade	10	22	34.60	33.11	15.86	15.86
		Subgrade	18	11	13.07	12.70	7.92	7.92
	Ch 6+600	Base	2.05	100	154.64	178.12	102.88	100
		Subbase	5.3	50	55.03	71.56	33.54	33.54
		Imp. Subgrade	12.5	18	24.34	23.64	12.19	12.19
		Subgrade	19.67	9.8	11.86	11.46	7.14	7.14
R30 1	Ch 12+950(L)	Base -1	1.37	100	235.27	262.27	165.53	100
		Base -II	1.16	100	279.27	307.69	201.44	100
		Subbase	1.83	100	174.17	198.63	117.63	100
		Imp. Subgrade	2.43	100	194.41	280.37	84.18	84.18
		Subgrade	3.5	85	72.22	84.89	54.73	54.73
	Ch 12+950(R)	Base - I	1.1	100	294.93	323.79	214.47	100
		Base - II	0.75	100	436.20	467.67	337.01	100
		Subbase	2.6	100	120.28	141.78	77.72	77.72
		Imp. Subgrade	8.32	27	44.96	43.71	19.70	19.7
		Subgrade	26.63	6	8.44	8.06	4.99	4.99
	Ch 12+970(L)	Base -I	1.04	100	312.39	341.70	229.15	100
		Base -II	1.16	100	279.27	307.69	201.44	100
		Subbase	3.11	99	99.27	119.39	62.91	62.91
		Imp. Subgrade	18.5	11	11.03	13.08	7.67	7.67
		Subgrade	9	25	27.35	28.38	17.96	17.96
	Ch 12+970(R)	Base -I	1.21	100	267.41	295.48	191.66	100
		Base -II	1.17	100	276.82	305.17	199.41	100
		Subbase	3.43	98	89.28	108.67	56.05	56.05
		Imp. Subgrade	12.33	18	24.91	24.13	12.38	12.38
		Subgrade	7.5	29	33.06	35.07	22.27	22.27
	Ch 12+990(L)	Base -I	1.37	100	235.27	262.27	165.53	100
		Base - II	0.87	100	374.96	405.57	282.86	100
		Subbase	2.34	100	134.51	156.87	88.01	88.01
		Imp. Subgrade	14.5	15	18.68	18.89	10.23	10.23
		Subgrade	17.47	12	13.50	13.15	8.21	8.21
	Ch 12+990(	Base - I	0.84	100	388.63	419.46	294.82	100
		Base - II	0.675	100	485.53	517.45	381.62	100

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	R)	Subbase	3	100	103.20	123.58	65.65	65.65
		Imp. Subgrade	17.5	12	12.61	14.22	8.19	8.19
		Subgrade	11.13	20	21.88	22.19	13.97	13.97
Z3024	Ch 4+800 (L)	Base	0.81	100	403.31	434.36	307.75	100
		Subgrade	16.33	13	14.53	14.22	8.89	8.89
	Ch 4+820 (R)	Base	0.846	100	385.82	416.60	292.36	100
		Subgrade	18.8	10	12.46	12.08	7.53	7.53
	Ch 4+840 (L)	Base	1.71	100	186.94	211.99	127.43	100
		Subgrade	17.6	11	13.39	13.04	8.14	8.14
	Ch 4+860 (R)	Base	0.986	100	329.93	359.65	244.03	100
		Subgrade	22.25	8	10.34	9.93	6.17	6.17
	Ch 4+880 (L)	Base	2.19	100	144.25	167.18	95.17	100
		Subgrade	33.31	4.8	6.51	6.22	3.83	3.83
	Ch 4+ 900 (R)	Base	0.832	100	392.44	423.33	298.17	100
		Subgrade	20.68	8.5	11.22	10.81	6.73	6.73
	Ch 4+ 920 (L)	Base	1.4	100	230.1	256.87	161.35	100
		Subgrade	24	7.25	9.50	9.10	5.64	5.64
	Ch 4+ 940 (R)	Base	1.31	100	246.40	273.79	174.51	100
		Subgrade	33.47	4.7	6.47	6.19	3.81	3.81
	Ch 4+ 960 (L)	Base	0.976	100	333.38	363.19	246.98	100
		Subgrade	16.34	13	14.52	14.21	8.88	8.88
	Ch 4+ 980 (R)	Base	0.89	100	366.36	396.81	275.38	100
		Subgrade	15.2	14	15.70	15.45	9.67	9.67
	Ch 5+ 000 (L)	Base	1.65	100	194.02	219.39	132.92	100
		Subgrade	19.5	9.5	11.97	11.58	7.21	7.21

Table A - 9.2: Resilient Modulus test results for subgrade soils

Subgrade sample No.	Confining pressure (kPa)	Contract stress (kPa)	Deviator stress (kPa)	Bulk stress, (kPa)	Permanent strain (%)	Recoverable strain (%)	Resilient Modulus (MPa)
01	41.4	1.4	13.8	138.0	0.6	0.024	50.907
	41.4	2.8	27.5	151.7	0.6	0.051	48.180
	41.4	4.1	41.4	165.6	0.7	0.097	38.313
	41.4	5.5	55.2	179.4	0.8	0.160	30.976
	41.4	6.9	68.8	193.0	1	0.222	27.914
	27.6	1.4	13.8	96.6	0.8	0.022	57.546
	27.6	2.8	27.5	110.3	0.8	0.054	46.056
	27.6	4.1	41.4	124.2	0.9	0.115	32.351
	27.6	5.5	55.2	138.0	0.9	0.185	26.923
	27.6	6.9	68.8	151.6	1.1	0.246	25.209
	14.4	1.4	13.8	57.0	0.9	0.025	51.141
	14.4	2.8	27.5	70.7	0.9	0.065	38.416
	14.4	4.1	41.4	84.6	1	0.137	27.183
	14.4	5.5	55.2	98.4	1.1	0.212	23.411
	14.4	6.9	68.8	112.0	1.3	0.268	23.161
02	41.4	1.4	13.8	138.0	0.7	0.022	57.107
	41.4	2.8	27.5	151.7	0.8	0.046	54.360
	41.4	4.1	41.4	165.6	0.9	0.091	40.786
	41.4	5.5	55.2	179.4	1	0.156	31.801
	41.4	6.9	68.8	193.0	1.3	0.220	28.239
	27.6	1.4	13.8	96.6	1.2	0.024	50.838
	27.6	2.8	27.5	110.3	1.2	0.058	42.567
	27.6	4.1	41.4	124.2	1.3	0.116	32.243
	27.6	5.5	55.2	138.0	1.4	0.179	27.786
03	41.4	1.4	13.8	138.0	2.6	0.023	53.534
	41.4	2.8	27.5	151.7	2.7	0.043	57.968
	41.4	4.1	41.4	165.6	3	0.077	48.500
	41.4	5.5	55.2	179.4	3.6	0.127	39.018
	41.4	6.9	68.8	193.0	4.7	0.200	30.949
	27.6	1.4	13.8	96.6	4.6	0.026	48.497
	27.6	2.8	27.5	110.3	4.6	0.058	42.732
	27.6	4.1	41.4	124.2	4.7	0.114	32.581

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Subgrade sample No.	Confining pressure (kPa)	Contract stress (kPa)	Deviator stress (kPa)	Bulk stress, (kPa)	Permanent strain (%)	Recoverable strain (%)	Resilient Modulus (MPa)
	27.6	5.5	55.2	138.0	4.8	0.172	28.906
Soaked sample	41.4	1.4	13.8	138.0	1.8	0.036	34.308
	41.4	2.8	27.6	151.8	1.8	0.089	27.881
	41.4	4.1	41.38	165.6	2.7	0.177	21.104
	41.4	5.5	55.22	179.4	4.6	0.249	19.998
4	41.4	5.5	55.2	179.4	4.5	0.1842	27.07

Table A- 10.1: Resilient modulus test results for Brick Subbase samples

Sample No.	Confining pressure (kPa)	Deviator stress (kPa)	Bulk stress, (kPa)	Permanent strain (%)	Recoverable strain (%)	Resilient Modulus (MPa)
01	20.9	20.68	41.58	1.4	0.054	34.469
	20.9	41.36	62.26	1.5	0.077	48.440
	20.9	62.10	83.00	1.5	0.088	62.998
	35.1	34.52	69.62	1.6	0.0699	44.349
	35.1	68.92	104.02	1.6	0.090	68.844
	35.1	103.40	138.50	1.7	0.10	89.721
	69.2	68.92	138.12	1.7	0.08	69.052
	69.2	137.86	207.06	1.9	0.112	109.861
02	20.9	20.7	83.4	1.3	0.061	30.450
	20.9	41.4	104.1	1.4	0.087	42.884
	20.9	62.2	124.9	1.4	0.098	57.242
	34.7	34.4	138.5	1.4	0.078	39.537
	34.7	68.9	173	1.4	0.097	63.908
	34.7	103.4	207.5	1.5	0.110	84.440
	69.5	68.9	277.4	1.5	0.097	63.848
	69.5	137.9	346.4	1.7	0.118	104.844
	69.5	206.8	415.3	2.1	0.142	131.321
	103.4	68.7	378.9	2	0.101	61.728
	103.4	103.4	413.6	2.1	0.112	83.484
	103.4	206.9	517.1	2.9	0.143	129.830
	137.8	103.4	516.8	3.2	0.110	84.463
03	20.9	20.68	83.38	1.7	0.049	37.597
	20.9	41.36	104.06	1.8	0.071	52.426
	20.9	62.1	124.8	1.8	0.082	68.346

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Sample No.	**Confining pressure (kPa)**	**Deviator stress (kPa)**	**Bulk stress, (kPa)**	**Permanent strain (%)**	**Recoverable strain (%)**	**Resilient Modulus (Mpa)**
03	35.1	34.52	139.82	1.8	0.064	48.234
	35.1	68.92	174.22	1.8	0.082	75.336
	35.1	103.4	208.7	1.9	0.095	98.220
	69.2	68.92	276.52	1.9	0.083	75.023
	69.2	137.86	345.46	2	0.104	119.291
	69.2	206.72	414.32	2.4	0.127	146.085
	103.8	68.9	380.3	2.3	0.089	69.613
	103.8	103.42	414.82	2.3	0.101	92.394
	103.8	206.86	518.26	2.6	0.126	147.152
	138.1	103.42	517.72	2.5	0.104	89.715
	138.1	137.88	552.18	2.6	0.111	112.204
	138.1	275.82	690.12	3.1	0.152	163.511
04	20.9	20.68	83.38	0.9	0.056	32.995
	20.9	41.36	104.06	1	0.078	47.946
	20.9	62.1	124.8	1	0.088	63.584
	35.1	34.52	139.82	1	0.070	44.510
	35.1	68.92	174.22	1	0.087	71.210
	35.1	103.4	208.7	1.1	0.099	93.846
	69.2	68.92	276.52	1.1	0.086	71.806
	69.2	137.86	345.46	1.2	0.107	115.773
	69.2	206.72	414.32	1.5	0.126	147.508
	103.8	68.9	380.3	1.4	0.091	68.510
	103.8	103.42	414.82	1.4	0.102	91.277
	103.8	206.86	518.26	1.6	0.124	149.662
	138.1	103.42	517.72	1.5	0.103	90.318
	138.1	137.88	552.18	1.6	0.110	112.623
	138.1	275.82	690.12	1.9	0.138	179.572

Table A - 10.2: Resilient modulus test results for Brick Base samples

Sample No.	Confining pressure (kPa)	Deviator stress (kPa)	Bulk stress, (kPa)	Permanent strain (%)	Recoverable strain (%)	Resilient Modulus (MPa)
01	20.9	20.68	83.38	1.4	0.071	26.341
	20.9	41.36	104.06	1.4	0.101	36.904
	20.9	62.10	124.80	1.5	0.116	48.169
	35.1	34.52	139.82	1.5	0.093	33.421
	35.1	68.92	174.22	1.5	0.118	52.671
	35.1	103.40	208.70	1.6	0.130	71.860
	69.2	68.92	276.52	1.6	0.119	52.330

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Sample No.	Confining pressure (kPa)	Deviator stress (kPa)	Bulk stress, (kPa)	Permanent strain (%)	Recoverable strain (%)	Resilient Modulus (MPa)
01	69.2	137.86	345.46	1.8	0.137	90.554
	69.2	206.72	414.32	2.2	0.154	121.096
	103.8	68.9	380.30	2.1	0.126	49.316
	103.8	103.42	414.82	2.2	0.134	69.601
	103.8	206.86	518.26	2.7	0.144	129.196
	138.1	103.42	517.72	2.6	0.128	72.490
	138.1	137.88	552.18	2.7	0.130	95.840
	138.1	275.82	690.12	3.1	0.140	177.287
02	20.9	20.7	83.4	0.1	0.075	24.667
	20.9	41.4	104.1	0.1	0.075	36.015
	20.9	62.1	124.8	0.1	0.104	49.145
	35.1	34.6	139.9	0.2	0.114	32.636
	35.1	68.9	174.2	0.1	0.095	55.052
	35.1	103.5	208.8	0.2	0.113	78.749
	69.2	68.9	276.5	0.2	0.118	55.530
	69.2	137.9	345.5	0.2	0.112	105.210
	69.2	206.5	414.1	0.3	0.118	151.343
	103.8	68.9	380.3	0.4	0.123	54.877
	103.8	103.5	414.9	0.2	0.113	78.519
	103.8	206.5	517.9	0.3	0.119	153.357
	138.1	103.5	517.8	0.4	0.121	78.337
	138.1	137.9	552.2	0.3	0.119	103.668
	138.1	275.8	690.1	0.3	0.120	186.364
03	20.9	20.68	83.38	2.2	0.060	31.307
	20.9	41.36	104.06	2.2	0.086	43.293
	20.9	62.1	124.8	2.3	0.099	56.238
	35.1	34.52	139.82	2.2	0.077	40.125
	35.1	68.92	174.22	2.3	0.101	61.587
	35.1	103.4	208.7	2.4	0.115	81.209
	69.2	68.92	276.52	2.4	0.100	62.114
	69.2	137.86	345.46	2.5	0.119	104.659
	69.2	206.72	414.32	2.7	0.129	143.853
	103.8	68.9	380.3	2.6	0.099	62.863
	103.8	103.42	414.82	2.7	0.110	84.665
	103.8	206.86	518.26	2.8	0.126	148.017
	138.1	103.42	517.72	2.8	0.110	84.991
	138.1	137.88	552.18	2.8	0.115	107.460
	138.1	275.82	690.12	3	0.132	187.362
04	20.9	20.68	83.38	1	0.051	36.365
	20.9	41.36	104.06	1.1	0.073	51.331

Cont…

Sample No.	Confining pressure (kPa)	Deviator stress (kPa)	Bulk stress, (kPa)	Permanent strain (%)	Recoverable strain (%)	Resilient Modulus (MPa)
04	20.9	62.1	124.8	1.1	0.083	67.131
	35.1	34.52	139.82	1.1	0.066	47.237
	35.1	68.92	174.22	1.1	0.084	73.915
	35.1	103.4	208.7	1.2	0.097	96.393
	69.2	68.92	276.52	1.2	0.083	74.316
	69.2	137.86	345.46	1.3	0.105	118.277
	69.2	206.72	414.32	1.5	0.130	143.859
	103.8	68.9	380.3	1.5	0.074	83.856
	103.8	103.42	414.82	1.5	0.095	98.035
	103.8	206.86	518.26	1.6	0.129	143.767
	138.1	103.42	517.72	1.6	0.091	101.705
	138.1	137.88	552.18	1.6	0.110	113.264
	138.1	275.82	690.12	1.8	0.143	195.235
5*	138.1	275.82	690.12	0.6	0.137	180.25

***** *The 5th test is done by user defined test condition.*

Table A- 11.1: Summary of testing methods and evaluated design input parameters

Name of input parameter for design	Name of the test	Appropriate standard (BS /ASTM/ others)	Number of tests done	Test Results used as design input parameters (upper & lower range of values)
CBR value	Lab. Soaked CBR.	ASTM D 1883 – 92.	Subbase = 10 Nos Base = 15 Nos	For subbase layer CBR = 32.25 % to 73.79% For base layer CBR = 29% to 119%
Resilient Modulus (M_r)	1.Cyclic triaxial test	AASHTO T-307	4 nos brick - sand mix, 4 nos brick aggregates	Subbase layer M_r = 35 to 179 MPa Base layer (brick aggregate) M_r = 52 to 195 MPa
Gradation of aggregate	Sieve analysis	BS1377- 2:1990,	For subbase = 10 nos For base = 13 nos	Enclosed in chapter ten
Elastic modulus of bituminous layer	1.Witczak model	(Using correlation)	4 Nos	E = 1394.46 to 1904.4 MPa
Bitumen content of the bituminous mixtures	Bitumen extraction test	STP 10.4 of RHD specification	For wearing course = 5 nos For binder course = 4 nos	For Wearing course = 4.13 to 6.15% For Base course = 5.1% to 5.63%
Stability & flow of bituminous materials	Stability & flow test	ASTM D 1559	11	Stability = 618 to 1844.8 kg, Flow = 9 to 19.5 mm
Viscosity of bitumen	Viscosity test of bitumen	ASTM D 2170	4	1066 to 16965 cSt
Bitumen properties test like ductility, softening point, loss of heating etc	Ductility test, Ring & ball test, Loss of heating test.	ASTM D 113, D 36, D 6-95	4	Ductility = 3.25 to 43.25 Softening point=63^o to 97^oC Loss of heating = 0%
Particle size distribution of aggregates	Sieving test	BS1377- 2:1990,	9	Enclosed in chapter ten
ACV & TFV of aggregates	ACV & TFV test	BS812-110:1990, BS812-111:1990	4	ACV = 20 to 23% TFV = 120 to 150 kN
Elongation & Flakiness index	Elongation & flakiness index test	BS 812 – 105.2:1990 & BS 812-105.1:1989	4	Elongation index = 20 to 34% Flakiness index = 14 to 26%

Remarks: These results are derived for limited number of granular and bituminous samples

APPENDIX – H: Appendix Figures

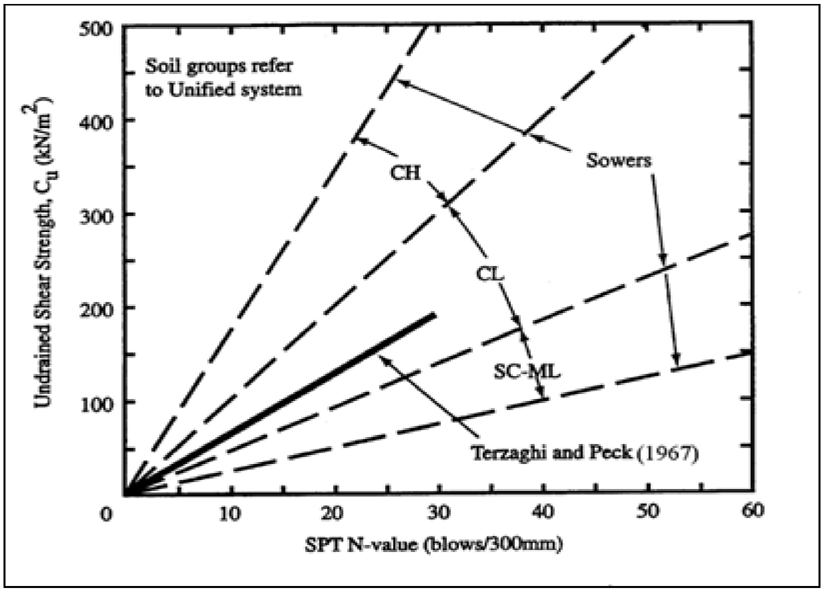


Figure B- 4.1: Approximate correlation between undrained shear strength and SPT – N value (After Sowers, 1979)

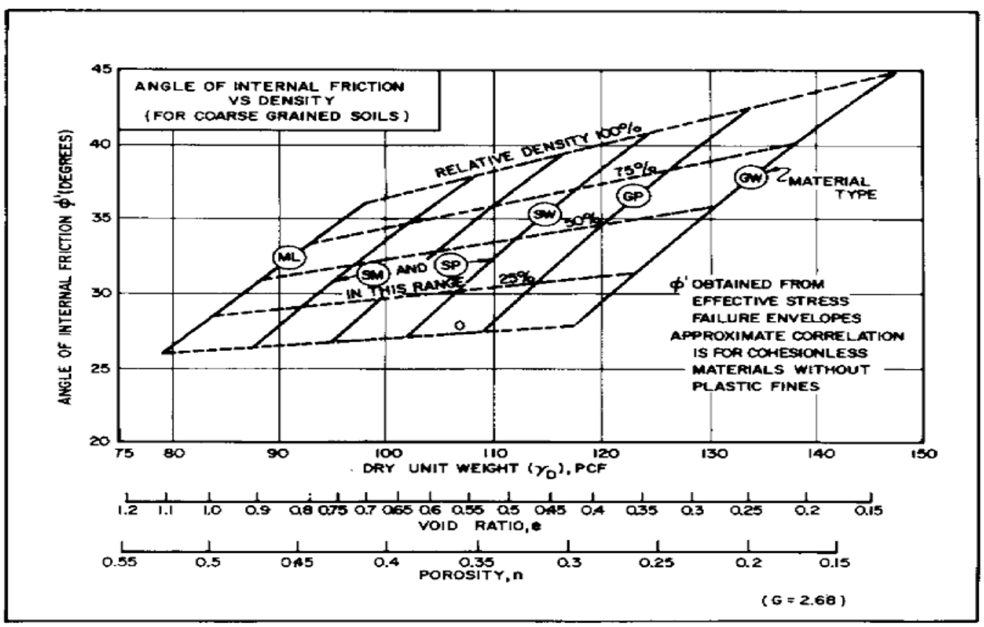


Figure B- 4.2: Relationship among Angle of Internal Friction, Dry Unit Weight, Void Ratio and Porosity of the granular materials (U.S. Navy, 1982).

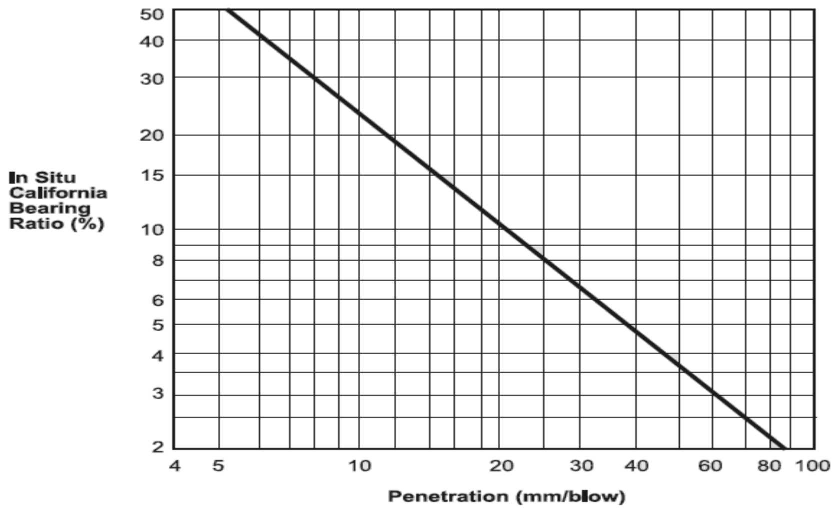


Figure B- 4.3: Relationship between in situ CBR (%) and Penetration (mm /blow) for fine grained cohesive soils (Ausroad pavement design guide)

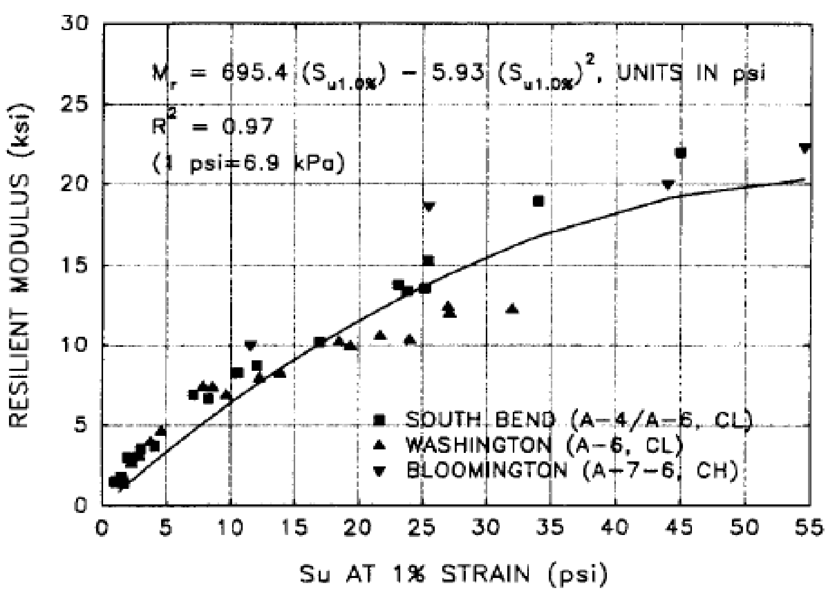


Figure B - 4.4: Relationship between Resilient Modulus and Unconfined Compression Strength for Washington, South Bend, and Bloomington soils. (Lee, 1997)

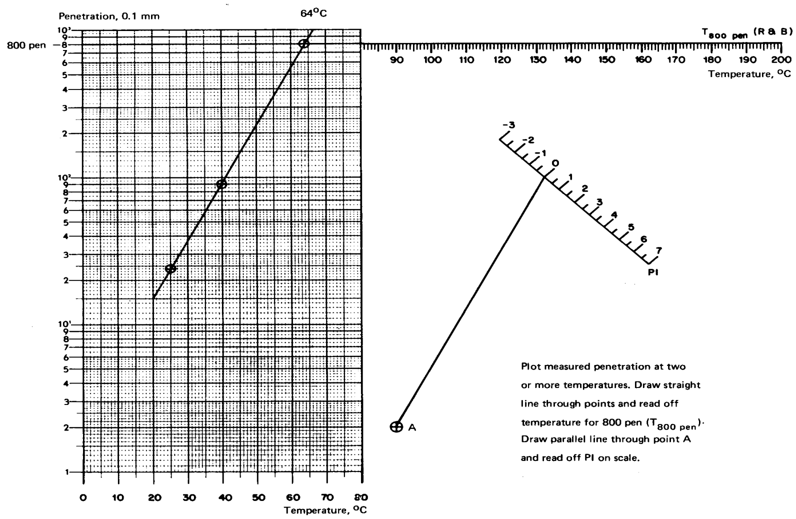


Figure B -4.5: Nomograph for determination of Penetration Index (PI) (Shell, 1978)

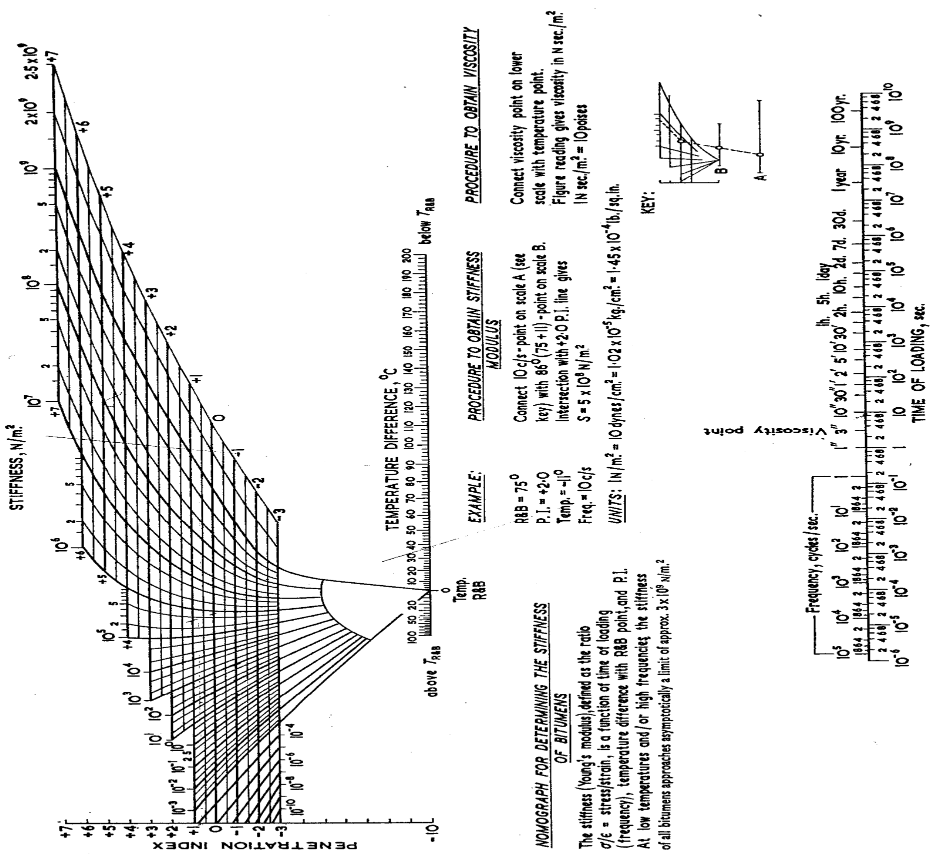


Figure B- 4.6: Nomograph for Stiffness Modulus determination (Van der Poel, 1954)

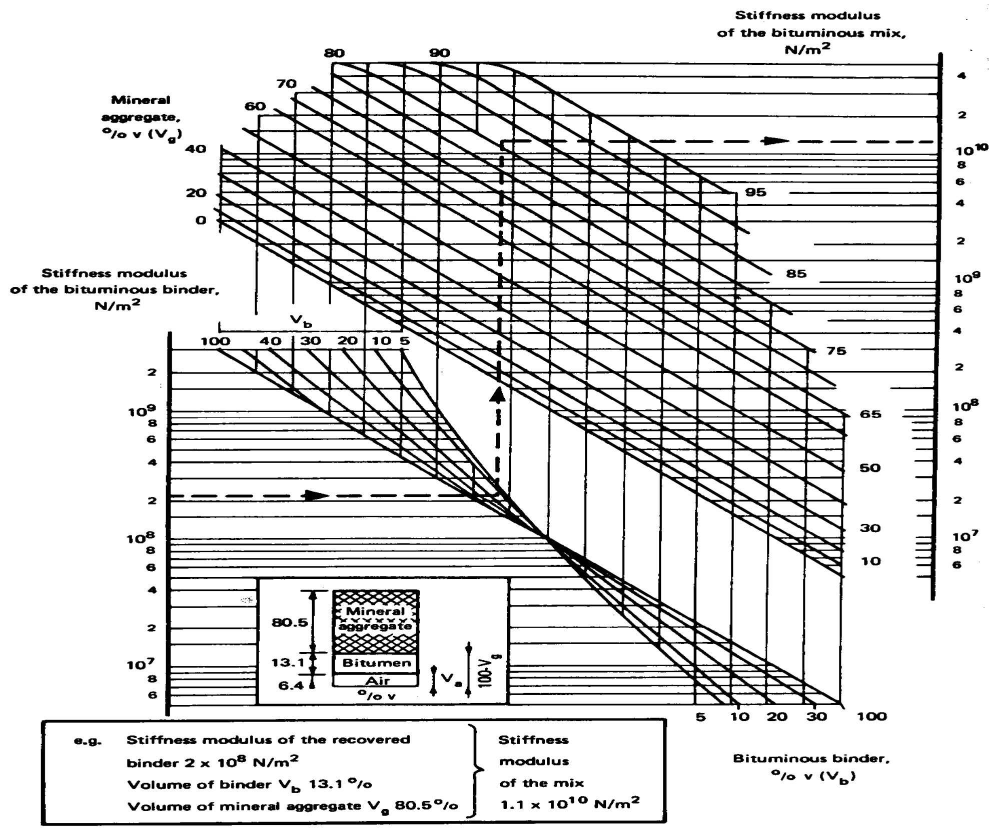
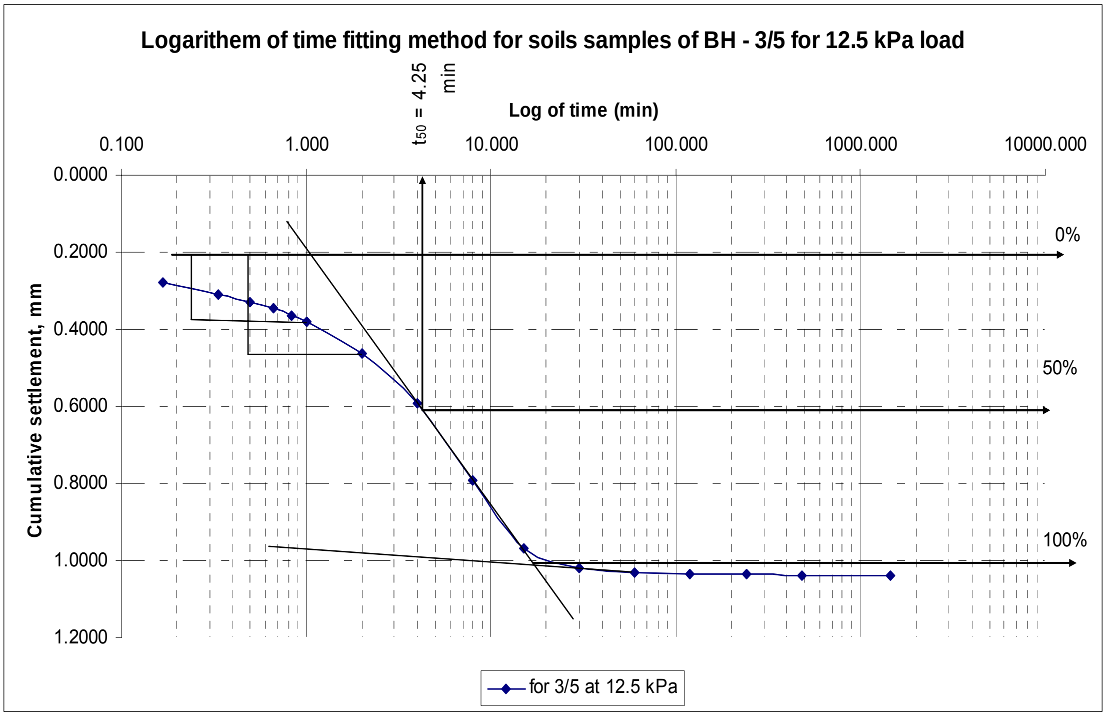


Figure B - 4.7: Nomograph for stiffness modulus of bitumen- Aggregate mixes (Shell, 1978)

Figure B- 7.1: A new DCP soil Sampler.



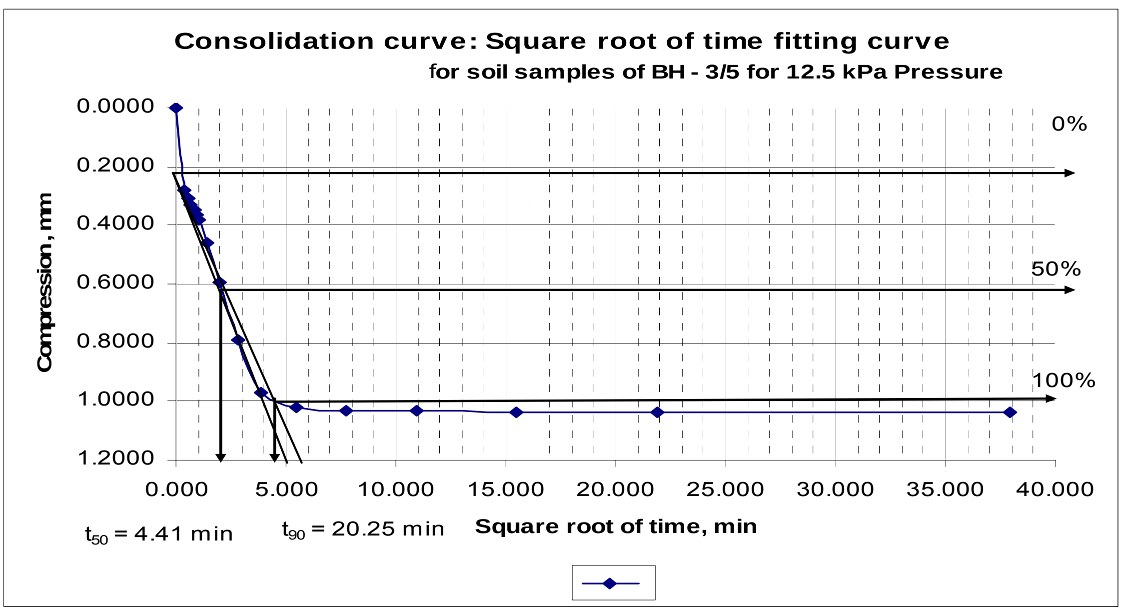


Figure B -8.1: Calculation of t_{50} & t_{90} for both logarithem of time fitting and square root of time method.

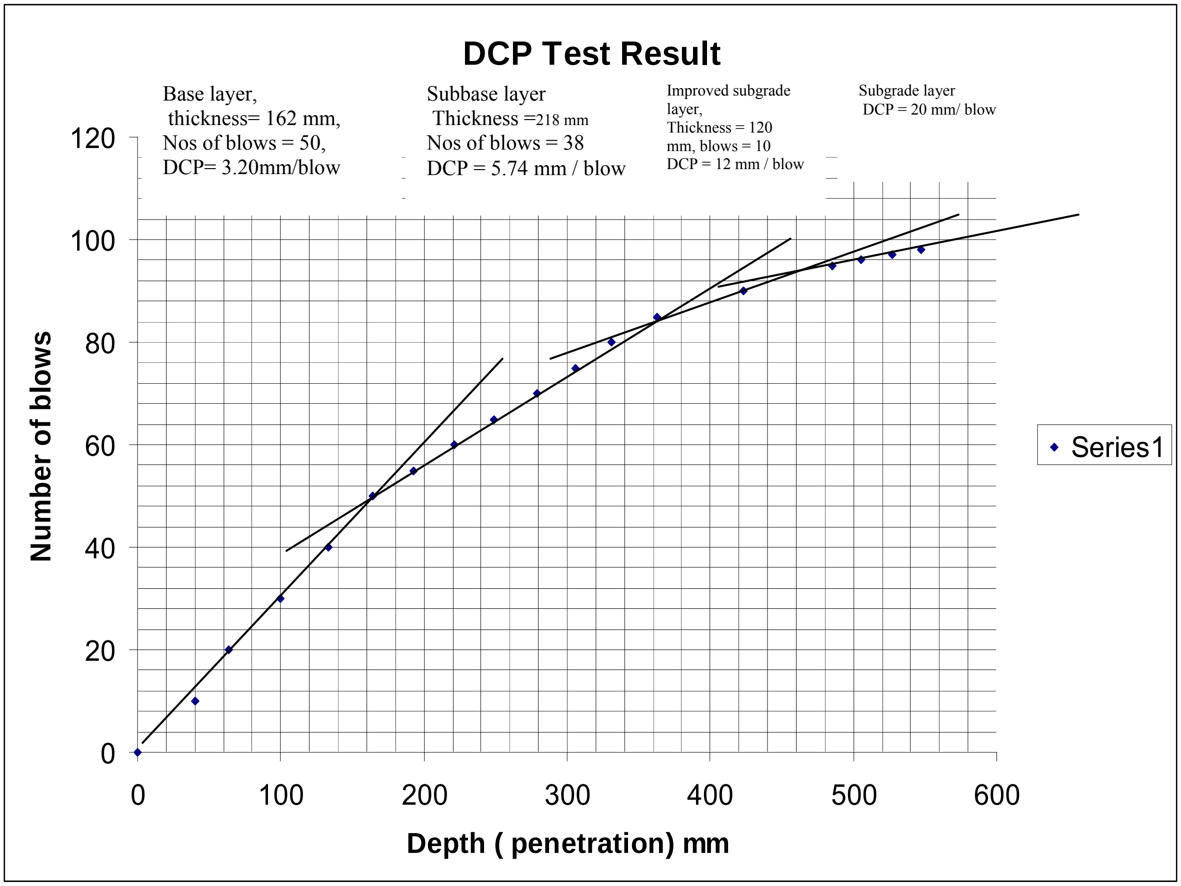


Figure B -9.1: A typical calculation of DCP value as well as layer thickness from DCP test at Ch 6+000 (L/S) of N302.

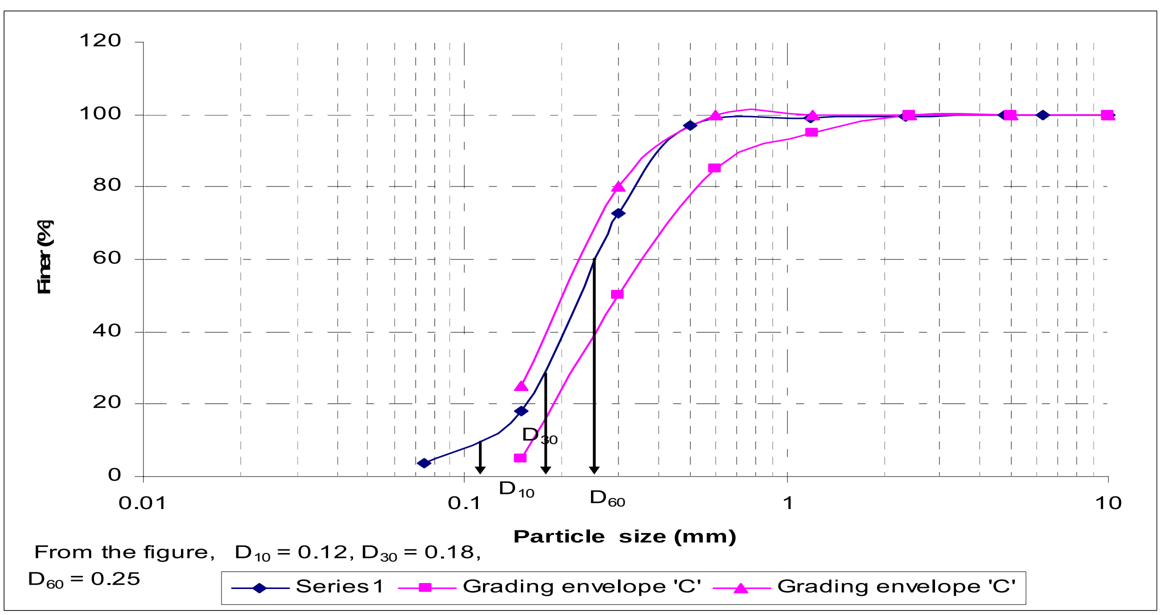


Figure B -9.2: Particle size distribution of sand used in cyclic triaxial test.