ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

EVALUATION OF WEATHERED ROCK MATERIAL
FOR EMBANKMENT DAM CONSTRUCTION
(The case of Zarema May Day Dam)

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Notations

ASTM - American society for testing materials

Cc - Compression Index

CH - Inorganic Clays of High Plasticity

CL - Inorganic Clays of Low to Medium Plasticity

Cu - Uniformity Coefficient

D10 - Size of materials for which 10% by weight is finer

D15F - Size of 15% of the filter material is finer

D15B - size of 15% of the Base (protected) material is finer

D30 - Size of materials for which 30% by weight is finer

D60 - Size of materials for which 60% by weight is finer

GC - Clayey gravels, Gravel-Sand-Clay mixtures

Gs - Specific gravity

GW - Well-graded gravels, gravely- sand mixtures, little or no fines

Ip, PI - Plasticity index

LL, WL - Liquid limit

MDD, gdmax - Maximum dry density

OMC, wopt - Optimum moisture content

SC - Clayey sands, Sand-Clay mixtures

SP - Poorly-graded sands, gravely sands, little or no fines

SW - Well-graded sands, gravely sands, little or no fines

U.S.B.R - United States Department of the Interior Bureau of Reclamation

USCS - Unified soil classification system

UU – Undrained unconsolidated

WWDSE - Water works Design and Supervision enterprise.

1D - One dimensional

Abstract

In the construction of many embankment dams rock materials plays a vital role. These rock materials exist at every parts of the world at different stages of weathering which is from residual soil up to fresh intact rock. GW soils (well graded gravels) are the best free draining shell material.

Nevertheless, several problems associated with these materials when they are found at highly weathered condition which creates more fines. The most common problems due to these are pore water development in the dam body, shear strength decrement and failure of the shoulder section of the dam.

In this thesis, evaluation has been conducted on weathered rock materials at different level using several laboratory tests to determine their suitability for use in embankment dam construction. The laboratory test includes grain size analysis, Atterberg limits, permeability, compaction, specific gravity, Los-Angeles abrasion test and direct shear test. Also field Schmidt hammer test had been conducted.

The test result on several samples indicate that as the weathered rock samples produce more fines, the permeability and shear strength of rock decreases. But in recent embankment dam engineering technology there is an approach to design and construct the dam based on the available material on the site without extensive processing. This means accommodating easily accessible materials surrounding the site based on the strength of materials. Also provide technical procedures to following appropriate construction and techniques starting from quarrying up to placement of the material.

From samples collected SN-10 is desirable for core of the dam. And SN-1, SN-2 and SN-3 are best to be inner shell and transition filter from the core to the outer shell. Sample no SN-6 and SN-8 are best to be outer shell because they have high hydraulic conductivity (greater than 10^{-4} cm/s) and helps to stabilize the dam by dissipating pore water pressure. As well as SN-4, SN-7 and SN-9 are better material for inner shell.

CHAPTER 1

INTRODUCTION

The construction of dams ranks with the earliest and most fundamental of civil engineering activities. Most dams constructed in the history of the world are earth fill embankment dam. Now days, with in constraints imposed by local circumstances and economic criteria embankment dams become more preferable choice than concrete dams. The two main advantages of embankment dams are, they can be constructed by locally available materials and the construction of embankment dam doesn't need sound foundation as concrete dams.

1.1 Problem statement

During material investigation for Zarema May Day dam, metavolcanic rock material was selected as shell zone fill. This covers the largest amount of dam body volume. While trial test fills are done using these metavolcanic rock material produce more fine which creates impermeable condition. Meta -Volcanic rocks particularly basalts and andesites often show subtle alteration effects which in some case render them unsuitable for some or all of these purposes. Therefore it cannot be free draining zone. In fact this metavolcanic rock is abundant in the dam area and its weathering effect makes it unfit for the design criteria of shell zone. Shell material of embankment dam has to be pervious in order to decrease the pore water pressure within the dam body and it should have high shear strength. But, when weathered rock material selected as shell zone construction material it has its own problem in case of vibro compaction during construction. This problem comes when the pore spaces are filled with fine shell material during compaction which leads to an increase in pore water pressure. Simultaneously the raise in pore water pressure directly leads to decrease in effective shear strength which decreases the total shear strength. Due to this free draining material constraint become a problem to fulfill the design criteria and to find large amount of material in the vicinity of the site to construct the proposed rock fill asphalt concrete core dam.

1.2 Objective of the study

General Objective

The objective of this thesis is to evaluate the properties of weathered rock materials and verify their suitability for potential use in the construction of embankment dam. It is intended to identify the appropriate type of fill material with economical way of supply during construction. The construction of dam body encompasses the major part of the project budget. Therefore selecting the suitable material at reasonable haul distance has both economical and structural ramification. Hence, this research can provide additional information about fill materials that are weathered and also investigate other methods which can be used as an input for the ongoing project in comparison with their type and degree of weathering.

Specific Objective

The specific objective of this thesis work is to characterize construction material for different zone of embankment dam.

1.3 Methodology

The study starts by selecting weathered volcanic rocks or weathered metamorphic rocks, and makes a comprehensive evaluation by comparing their quality with the requirements as embankment materials. This would mean taking several samples from different sites and then making an evaluation of these materials. Important samples for the study purpose are collected from Zarema May Day dam and irrigation project, which were primarily located by Water Works Design and Supervision Enterprise (WWDSE) and found to be weathered rock at different level. Even if these materials are at economical distance and with required quantity, their weathering effect makes them less suitable for use as shell material. It is necessary to evaluate the potential use of these materials in the construction and design of embankment dams such as Zarema May Day dam. Hence the following steps are undertaken:-

- Field visit and identification of site to collect samples
- Collection of weathered rock samples for laboratory test
- perform laboratory test
- Compare test results to proposed limiting values and engineering use chart
- Interpretation and conclusion.

Desk Study

The first step at this stage was reviewing to the study and design documents about the project from WWDSE library to have the general over view. Then secondary data sources like topographic, soil and geological maps were bought from Ethiopian Mapping Enterprise and

Ethiopian Geological Survey respectively. The main strategic frame work was planned after consulting WWDSE experts (the previous consultants), SUR construction geotechnical engineering staff (main contractor) and SGI STUDIO INGENERING (consultant). Documents were found regarding geotechnical and geological investigation which was very helpful for further investigation. Based on the information gathered from engineering geologists, geotechnical engineers, maps and previous documents about the project potential source were primarily identified within 5 km radius from the dam axis to be economically feasible.

Field Investigation

Under this session the first task was reconnaissance survey of the project around dam site. Basically four locations are selected for the survey. These were upstream of the dam, downstream of the dam, right side and left side abutments along the dam axis out ward. On the right and left side cut of trench along the dam axis shows different stratum of metavolcanic rocks with different degree of weathering. At the top surface residual soil found up to a depth of 1m. Going down in to the valley to which the diversion conduit is constructed the stratum changed to fresh metavolcanic rock having uni-axial compressive strength value within the range of 60-70 MPa. Along the right side of the abutment high amount of soil rock mixtures were observed which is considered as the result of weathering. Along the upstream side the hilly parts of the valley highly dominated by weathered metavolcanic rocks and colluvial deposits with yellowish brown silty gravel with certain amount of parent rock matter. The alluvial deposits found in the upstream and downstream direction have similar feature.

Sampling

Since the dam will be constructed by suitable quarried material from the proposed location, the fill material should be tested in their disturbed condition. For strength parameters like shear strength tests remolded samples are prepared at their optimum moisture content found from modified proctor test.

Samples were collected based on their degree of weathering (soil rock mixture) for laboratory testing. Sampling locations are from right abutment trench along the dam axis, Left abutment trench, upstream and right abutment trench along the dam axis around 1-2 km radius. Sample number for laboratory test was limited to 10 in order to represent different stratum and degree of weathering at small scale level.

***Picture 1.1**- Sampling on the site*

1.4 Organization of the thesis

The thesis consists of six Chapters. The introductory Chapter consists of the background, objective and a brief summary of the work. The second Chapter covers the literature review on weathered rock material and the process of weathering, problems associated with theses material and construction material criteria in embankment dam construction. The third Chapter consists of the sampling area descriptions and soil characteristics of the area of study. The fourth Chapter is devoted to laboratory test results and analysis. The fifth Chapter covers about interpretation and discussion of laboratory test results. The conclusions and recommendations are presented in the six Chapter. References and Appendix are presented at the end of this thesis.

CHAPTER 2

LITERATURE REVIEW

General

The main body of rock fill dams, which should have a structural resistance against failure, consists of rock fill shell, transition zones, core and facing zones have a role to minimize leakage through embankment. Filter zone should be provided in any type of rock fill dams to prevent loss of soil particles by erosion due to seepage flow through embankment. In earth fill dams, on the other hand, the dam body is the only one which should have both structural and seepage resistance against failure with a provided drainage facilities. Shell materials must exhibit adequate shear strength at economical slopes. Upstream shells should be as free draining as possible to ensure stability during rapid reservoir drawdown and under earthquake loadings. Poorer quality shell materials, i.e., materials containing more fines or which may break down on placement or exposure to elements and end up less pervious, may be used in downstream sections if adequately filtered internal drainage systems are provided .[1]

To ensure proper drainage, the USBR recommends the ratio of hydraulic conductivities of permeable zone to impermeable zone be at least 10, and preferably much larger. The hydraulic conductivity of the free-draining zone should be sufficiently high to preclude development of pore pressures during construction. To attain proper drainage the type of fill material is the most prominent factor. The permeability of component rock fill material should be at least 10^{-2} m/s. it is free drained and therefore no problems arise from earth quake when the material is saturated.

The U.S. Bureau of Reclamation's *Earth Manual* (USBR, 1974, 1990) identifies the following criteria for design of pervious earth fill zones:

- The material must be formed into homogeneous mass free from large voids
- The soil mass must be free draining.
- The material must not consolidate excessively under the weight of superimposed fill
- The soil must have a high angle of internal friction (i.e. high shear strength)

Good quality coarse material sources exist in the vicinity of many dam sites. However, specifications for filter and drain zones typically limit the percentage of fines, after compaction, and define gradation boundaries which are rarely met by the materials in their natural state. Processing is generally required, including washing to remove fines, handling to preclude size segregation, and remixing to achieve specified gradation requirements (ICOLD, 1990). Also, the quality of materials should be such that they do not break down into smaller sizes during placement, or weather excessively with time. Gradations should be checked before and after compaction of these materials to ensure that particle breakdown is not a problem. These design objectives are achieved by proper use of equipment and methods of borrow-pit conditioning, excavating, placing, compacting, and foundation shaping. Maximum density is achieved at optimum moisture content for a given compaction energy. However, other factors may influence the specified placement moisture. Weathered materials are used in many civil engineering projects as construction materials such as road pavement fill, embankment dam construction, back fill- - - -etc. Basically knowledge of these earthen materials is vital to use them in the construction of embankment dam according to the standards specified by the design.

2.1 Potential Causes of Pore Water Development in the Dam Body

In the construction of embankment dam the type of material used in the dam body and foundation condition are the main important factors for the stability of the dam as a whole. For zoned type of embankment dam the central impervious core, transition and pervious shell (shoulder) materials play great role in dam body construction. Shell zone material are expected to have high hydraulic conductivity in order to dissipate the pore water pressure created due to superimposed load and water tightness nature of the structure. Weathered rock materials produce fine particles during compaction when we use them as shell zone material which clogs the pore spaces and develop pore water pressure.

2.2 Weathering of Rocks

Weathering of a rock is its response to the change from the pressure, temperature, moisture and chemical environments in which it was formed, to its new environment at and near the ground surface. Weathering processes are of two fairly distinct types, namely mechanical, and chemical.

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2.2.1 Mechanical weathering

Mechanical weathering includes all of the near- surface physical processes which break rock mass down to progressively smaller rigid blocks or fragments, and cause those blocks to separate. Generally mechanical weathering proceeds chemical weathering. Mechanical weathering takes place due to the following natural processes.

- Gravitational creep
- Earth quake induced displacements
- Growth of tree roots in joints
- Expansion of clays in joints
- Freezing of water in joints etc….

2.2.2 Chemical weathering

Chemical weathering is caused mainly by circulating ground water which gains access to low-porosity rock substance via cleavage micro-cracks, open joints and fracture associated with faults. Most chemical weathering occurs at extremely slow rates, such that the changes to the strength of high strength, non-porous rocks are likely to be insignificant during the operating life of most civil engineering project. Chemical weathering involves the following operation.

- Chemical reaction between the minerals in the rocks
- Removal of the soluble decomposition products by leaching
- Decomposition of some decomposition products in pores or micro-cracks etc…

Factors which are responsible for the development of weathered profile are:-

- Climate and vegetation;
- Rock substance type;
- Defect type and patterns;
- Erosion;
- Time;
- Topography;
- Ground water.

To investigate engineering properties of theses weathered rock material it is recommended to classify based on their degree of weathering. The recommended classification are intended as a general guide lines which can form the basis for site specific or problem specific approaches. Products of rock weathering are often highly complex and variable over short distance. To assist in understanding them, some simplification and classification in to groups with like characteristics has been required. Two main types of classification have been developed:

- Rock substance types, which define fresh substance and up to 5 types of classification by Moyes(1955) and McMahon et al(1975);
- Rock mass types, in which rock masses are classified into zones showing progressively greater weathering effect by Ruxton and Berry (1957), Deere and Patton (1971), International society for Rock Mechanics (1978), and British standards institution (1981).

Mass type classification is best designed specifically for individual sites, because they need to be tailored to the project needs as well as the site conditions. In practice, sites for dams are usually classified in to zones taking into account variations in the overall structure of the rock mass, and the nature and distribution of at least the following;

Table 2.1 *Weathering Of Rock Substance Classification For Granite And Similar Rocks. Modified From Moye (1955) & Hosking (1990). [1]*

Fresh	FR	Rock show no evidence of chemical weathering
Slightly weathered	SW	Rock is slightly discolored but rings when struck by a hammer, not noticeably weaker than fresh rock
Moderately weathered	MW	Rock is discolored produces only a dull thud when struck by a hammer; noticeably weaker than fresh or slightly weathered rock but dry samples about 50 mm across cannot be broken across fabric by unaided hand.
Highly weathered	HW	Rock is discolored, can be broken and crumble by hand but doesn’t readily disintegrate in water
Extremely weathered	XW	Material disintegrate when gently shaken in water i.e. has soil properties

Table 2.2 ***Recommended descriptive terms for strength of rock substance [1]***

Rock strength	symbol	Approximate unconfined compressive strength Qu (Mpa)
Very weak	VW	< 5
Weak	W	5-25
Medium strong	MS	25-50
Strong	S	50-100
Very strong	VS	100-250
Extremely strong	ES	>250

Extremely weathered (or altered) rock is material which was once rock but has been converted by weathering (or alteration), in place, to soil material. Some such seem in this type of situation show slickenside surface, which can indicate either

- The seam was , or has formed next to, a bedding surface fault or
- It is simply a weathered bed within which some displacements have occurred since it becomes weathered.

Extremely weathered seams can be of large or small extent. Their shear strength, shear stiffness, compressibility and erodibilities, will depend largely on the composition and fabrics of the parent rocks. Weathered seams normally occur within or not far below the near-surface weathered zone, but altered seams can occur at any depth. Soil formed by extreme weathering of rocks can inherit any of the fabrics. Also they may contain remnant joints, crushed zones or sheared zones.

The contrast between the strengths of these remnant defects and that of the soil (extremely weathered rock substances) is usually lower than in a comparable mass of less weathered or fresh rock. The economic design and construction of rock fill materials in dams depends on utilizing onsite materials without extensive processing and excessive excavation of unusable material. Many quarries for rock fill materials require considerable excavation of overburden and weathered rock before clean rock fill can be obtained. ***Recent design and construction of large dams has demonstrated that the weathered and weak rock can be used by placing materials where their engineering properties are compatible with design criteria.*** *[6]*

2.3 Construction Material Criteria in Earth and Rock Fill Dams

2.3.1 Earth Fill Material

Core fill: - should have low permeability and ideally be of intermediate to high plasticity to accommodate a limited degree of deformation without risk of cracking. The most suitable soils have clay contents in excess of 25–30%, e.g. glacial tills etc., although clayey sands and silts can also be utilized. The core is the key element in an embankment and the most demanding in terms of material characteristics and uniformity, the properties of the compacted clay core being critical to long-term watertight integrity. [10]

Shoulder fill: - requires sufficiently high shear strength to permit the economic construction of stable slopes of the steepest possible slope angle. It is preferable that the fill has relatively high permeability to assist in dissipating pore water pressures. Suitable materials range across the spectrum from coarse granular material to fills which may differ little from the core materials. The shoulder need not be homogeneous; it is customary to utilize different fills which are available within predetermined zones within the shoulders [10]

Table 2.3 *Indicative engineering properties for compacted earth fill [10]*

fill type	compaction characteristics		shear strength		coefficient of permeability	drainage characteristics
BS 5930	unit weight,pd (kN/m³)	W opt (%)	C (kN/m²)	Friction Ø,(deg)	kh (m/s)	relief of U*w*
Gravel (GW-GC)	18-22	5-10	0	35-40	10^{-3} - 10^{-5}	Excellent
Sands (SW-SP)	16-20	10-20	0	35-40	10^{-4} - 10^{-6}	Good- fair
Silts (ML-MH)	16-20	15-30	<10	25-30	10^{-5} - 10^{-8}	Fair-poor
Clays (CL-CH)	16-21	15-30	<20	20-30	10^{-7} - 10^{-10}	Very poor-impervious
Crushed rock (2-600 mm)	17-21	N/A	0	40-55	10^{-1} - 10^{-2}	Free draining-excellent

Filter material: - must be clean, free draining and not liable to chemical degradation. Processed fine natural gravels, crushed rock and coarse to medium sands are suitable, and are used in sequences and grading determined by the nature of the adjacent core and/or shoulder fills

2.3.2 Rock Fill Material

Rock fill is not related to the mode of obtaining the material, for example through quarrying activity, but is related to its structure (gradation, mechanical characteristics, strength, permeability and so on). Rock fill in current practice includes angular rock fragments as produced by quarry or occurring as talus deposits, and rounded or sub angular fragments such as coarse gravel, cobbles, and boulders occurring in alluvial deposits. These materials are categorized in to clean rock fill material and dirty rock fill based on their gradation and engineering properties. Clean rock fill the rock content is sufficient to have rock to rock contact with the strength of the rock controlling the shear strength rather than the soils or fines, which have 60 to 70 % rock content. Clean rock fill material have hydraulic conductivity greater than 10^{-3} cm/sec.

Dirty rock fill with a hydraulic conductivity less than 10^{-3} cm/sec may be considered as earth fill because the possibility of developing construction pore pressures; more pervious material may be regarded as clean rock fill according to Penman (1976). These materials contain high amount of fines and their compressive strength is less than 15 mpa or 5 mpa. (Lousnov, 1981).

As evidenced by Kutzner [2], rock fill material is suitable for dam shells and for selected zones of all types of dams. Alluvial gravel and cobbles are an excellent “rock fill”, but usually rock fill material has to be quarried. It consists – ideally – of strong and durable rock fragments in the size of gravel and cobbles.

The material must be well graded. An excellent material for dam construction has the following grain size distribution

- not more than 5% below 5 mm,
- not more than 30% below 20 mm,
- maximum particle size 600 to 1000 mm, depending on the rock strength and the tendency towards particle breakage.

2.4 Requirements of Rock as a Construction Material

There are two current trends in rock fill dam design. The first trend is to design rock fill dams with earth fill water barriers using the principle of fill-material zoning. Such dams are economical when all types of locally available rock of different strengths can be used in the structure zoning. The weaker rock is placed in less critical zones under less stress and hard sound rock is used where greater strength is required.

The second trend is to build rock fill dams with man-made water barriers such as diaphragms or facings of reinforced concrete, asphalt concrete or other materials. Such dams are cost-effective by minimizing material volume for rock fill and water barrier features mainly through intense compaction of high strength rock fill materials with heavy vibratory rollers.

The most important properties of rock fill are: - gradation, compacted unit weight, permeability, strength and deformation.

Gradation – Ideally consists of strong and durable rock fragments in the size of gravel and cobbles. The material must be well graded. An excellent material for dam construction has the following grain size distribution. Not more than 5% below 5 mm, not more than 30% below 20 mm, according to USACE, after compaction, the fraction by weight passing ASTM Sieve No 200 shall be less than 5%, maximum particle size 600 to 1000 mm, depending on the rock strength and the tendency towards particle breakage[14]. Modern compacted rock fill contains a much higher percentage of fines. In fact, the more well-graded the material, the higher the unit weight of the placed material with the same compaction effort resulting in a less porous, denser embankment. Embankments of well-graded material have high moduli of deformation and there is less settlement as a result, as well as less crushing of the rock particles.

Compaction - Unit weight of compacted rock fill depends mainly on specific weight of the rock, grain size distribution, compaction effort, lift thickness and compacting machinery. Compaction is achieved from the traffic of loaded trucks and spreading dozers supplemented by passes of a heavy vibratory roller or other compaction equipment [6]. During the design phase, rock fill unit weight can be estimated from published data. During final design, rock fills unit weight and gradation can be confirmed from test fill. Where rock is weak and saturated, and specimens show significant loss of strength, conservative placement specifications can be determined from past experience.

A saturated test fill is advisable for large or high dams because many rock types have lower shear strength when saturated and loaded under very high confining pressures. Compaction of soils is one of methods of soil stabilization. Principal purposes of compaction of fill materials are increasing stiffness, to minimize settlements during and after construction, increasing strength, to prevent sliding shear failure of embankment, and making water tight, to obtain required imperviousness of the core zone. It is generally known that soils compacted in unsaturated states, especially in the dry side of the optimum water content have a certain skeleton strength composed by suction effect between soil particles. This skeleton strength readily disappears by wetting (saturation) during the first filling of the reservoir, which results in large settlement and drugs in the upstream shell of a rock fill dam and also in differential settlement and opening cracks in the core zone.

Field compaction of fill materials is usually carried out by using compaction rollers such as a tamping (sheep foot) roller, a rubber-tired roller and a vibration roller. In general, either a sheep foot roller or a rubber-tired roller is used for impervious or semi-impervious materials. For pervious materials such as sand, gravel and rock, rubber-tired or vibration rollers are usually employed. The usefulness of each roller depends on its compaction characteristics and soil types. The surface of the fill under compaction likely becomes smooth when a rubber-tired or a vibration roller is employed. This is remarkable when comparatively soft rock materials are compacted by a vibration roller or when impervious materials are compacted by a rubber-tired roller. It is readily recognized that the formation of smooth surface is undesirable in the stability of embankment slopes, causing reduction in shear resistance along this plane.

Permeability - Rock fill compacted in layers in a broad embankment is so non-homogeneous that, realistically; seepage through it is highly random and follows no known laws. It is worth to remark that the above expression for permeability (k) was initially given by Hazen (1892) for poorly graded sand and was confirmed by tests on well graded sandy gravel by Beyer (1964). Limitations are expected to apply in the use of such expression to characterize actual rock fill materials. Regardless of the above estimation it is to be remarked that permeability must be evaluated through permeability test suited for this kind of material, after compaction, firstly within the trial embankment and subsequently within the actual embankment during construction. The permeability of compacted soils, on the other hand, has a minimum peak on the wet side of the optimum.

Determination as to whether the rock fill is free draining, semi pervious or impervious is sufficient for design. These values can be determined by comparing gradations with those used in existing dams and field permeability tests in test fills, which is preferable. Rock fills with rock contents (plus 10 millimeters) of 60 to 70 percent can generally be considered free draining shell material.

The permeability of competent rock fill material is 10^{-2} m/s and it is considered as free draining material and, therefore, no problems arise from earthquake when material is saturated.

Strength - A thorough understanding and appraisal of the physical properties of soils and rock fill materials is essential to the use of current methods of design. No stability analysis, regardless of how intricate and theoretically exact it may be, can be useful for design if an incorrect estimation of the shearing strength of the construction materials has been made. In many cases, the errors arising from an improper appraisal of the soil and rock fill material properties can far exceed those resulting from the use of the more approximate methods of analysis USBR (1987). The degree of particle roundedness and the porosity determine the magnitude of the structural component of strength. This component increases shear resistance in much the same way as interlocking asperities on a rough joint surface. The structural component of strength is strongly stress dependent. For many dams, it is the practice to select shear strength values by comparison with test results obtained from published data for dams previously designed and constructed. Current stability analysis techniques allow for a variable shear strength dependent upon the confining pressure. The table below is a sample of shear strength under normal confining pressure for medium strength rock taken from ICOLD (1993). [6].

Table 2.4 *comparison of friction angle between tri-axial deformation & plane strain condition. [6]*

Normal pressure (kPa)	Triaxial Tests	
	Friction angle (Ø´t)* degrees	Friction Angle (Ø´ps)** degrees
14	53	57
35	50.5	54
70	48.5	52
140	46.5	50
350	44	47.5
700	42	45.5
1400	39.5	43
3500	37.5	41

*After Leps (1970)

** ICOLD (1993)

Note: Ø´t = friction angle under tri-axial deformation conditions.

Øps = friction angle under plane strain conditions.

Deformation – Some large scale testing on rocks confirmed the significance of particle breakage on the deformation behavior of rock fill under increasing applied stresses and on saturation. In summary, it was found that the deformation (and modulus) of rock fill was predominantly affected by: The compactive effort in placement of the rock fill, Increased modulus was observed with increased compactive effort (Marsal 1973). The applied stress level increased particle breakage and decreasing modulus were observed with increasing deviatoric stress levels in triaxial compression tests (Marsal 1973; Mariachi et al 1969), In odometer tests relatively high moduli were observed for compacted rock fill samples up to normal stresses in the order of 800 to 1000 kPa, thereafter the modulus was observed to decrease with increasing normal stress (Marshal 1973), the stress path. Significantly higher modulus was observed on un-loading and re-loading at stress levels less than previously experienced by the rock fill (Mori and Pinto 1988), the particle shape and grading of the rock fill. Greater deformation (and lower modulus) was observed for angular (in comparison to rounded), more uniformly graded (i.e., lower coefficient of uniformity, Cu) and coarser (similar Cu but higher maximum particle size) rock fills (Mariachi et al 1969, Marsal 1973, Bowling 1981), Intact rock strength. Reduced modulus, greater deformation and reduced strength were observed for weaker strength rock fills (Marsal 1973). [14].

Data obtained during construction using water-level settlement devices or cross-arms have been used to determine representative moduli of compressibility for a variety of rock types placed with different procedures. Experience indicates moduli range from 27 MPa to 128 MPa, depending on the nature of the rock, the grading of the rock fill, lift thickness, compaction and other factors, Cooke (1990). Where the modulus is of particular concern, such as the upstream rock fill shell of a concrete-face dam, special placing procedures are specified to obtain maximum modulus.

2.5 Proposed Limiting values and Desirability ratings

Gravelly soils are normally preferred construction and foundation materials because of their low compressibility and high shear strength. The GW and GP soils are pervious because they contain little or no fines to fill soil voids ordinarily, good drainage is ensured. Their properties are not affected appreciably by saturation and, if reasonably dense, these soils have good stability and low compressibility. In these respects, GW soils are better than GP soils. The GW and GP soils are virtually unaffected by freezing and thawing.

As the sand, silt, and clay fractions increase, the matrix soils begin to dominate the gravel skeleton structure, and the total material assumes more of the characteristics of the matrix. When properly compacted, GC soils are particularly good material for homogeneous, small earth fill dams or other embankments, or for the impervious sections of high earth dams. Permeability of GC soil is low, shear strength is high, and compressibility is low. An important factor in the behavior of gravelly soils is the gravel content at which interference between the large particles begins to influence total material properties.

Extensive compaction studies demonstrated that particle interference begins to influence compaction at about 30-, 35-, and 45-percent gravel content, respectively, for sandy, silty, and clayey gravel soils tested, when placed using standard comp active effort. Similarly, at about the same gravel content, shear strengths show significant effects of particle interference. Soils having angular gravel particles, as compared to rounded particles, show interference characteristics at lower gravel contents. A mixture's permeability is reduced as gravel content increases because solid particles replace permeable voids until the gravel content reaches an amount at which the matrix soils (i.e., sand, silt, or clay) can no longer fill the voids between the gravel particles. At that point, permeability increases with increase in gravel content. [5] which is also included in Appendix 2.

2.6 Examples from Pre Existing Design and Construction of Embankment Dams

Existing dams should be viewed in light knowledge of studies and reports on similar dams of the same vintage to gain an understanding of probable design and construction methods. The review of existing dams will generally not be as detailed as the procedures involved in the design of new dams. Some critical areas may require detailed review. Primarily, the review is intended to evaluate procedures and methodology of design and analysis to ensure that safe and adequate embankment dams were constructed.

Odelouca dam is a zoned embankment dam, 76 m high, built in Algarve, in south of Portugal. The crest of dam, 11 m wide, is about 415 m long. Odelouca dam creates a reservoir with 7.8 km^2 surface and 157 hm^3 capacities to the maximum water level. Most part of this regularize volume is intended for water supply and a small part will be used for irrigation purposes. The upstream slope is about 1:2.25 (V: H), beneath the berm, and 1:2 (V: H), above it. The downstream slope is about 1:2.25 (V: H), above the rock fill toe and 1:1.5 (V: H) beneath it. The embankment materials are clayey soil, at the core, and weathered schist and greywacke, with a significant fraction of oversized particles, at the shells. The selected shell materials should fulfill the following: maximum fines content 30%, particles passing #10, #4 and ¾" sieves, respectively, between 10 and 50%, 18 and 62%, and 33 and 93%, and average and maximum particles diameters, respectively, between 2 and 50, and between 50 and 400 mm. *[15]*

2.7 Embankment Zoning

For zoned embankments, the zoning geometry and properties of the materials placed in the zones should be reviewed to determine: (1) the structural design, and (2) the types of internal features such as chimney drains, blanket drains, toe drains, etc., that are proposed or were used to provide for and maintain embankment stability. One should keep in mind that embankment zoning is also established for economic reasons according to the availability of materials. The embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells, and seepage control zones. Desirable characteristics that these zones should have or provide are as follows: [2]

In general, the width of the core at the base of cutoff should be equal to, or greater than, 25 percent of the maximum difference between the maximum reservoir and minimum tail water elevations. The minimum top width of the core should not be less than 3.048m. The coefficient of permeability of the core material should preferably be 10^{-4} cm/sec or less. More permeable core material may be acceptable if seepage is still adequately controlled and appropriate factors of safety are still met. Transition zones must meet accepted filter criteria, to protect the adjacent zones from piping. The transition zones should be sufficiently wide to ensure that they are continuous and construct able with a minimum of contamination at the contact. The range of gradation of the transition zones should be limited to avoid segregation of materials during placement. Seepage control features within the embankment should be sized adequately to contain all seepage flows. The features should also be sufficiently pervious to ensure that all seepage will be intercepted and controlled without excessive pressure head losses. Zoning of an embankment that places the more pervious material on each side of the core zone is preferable. This placement improves the stability of the embankment during rapid drawdown conditions and keeps the downstream slope drained for greater effective weight. [4]

CHAPTER 3

DESCRIPTION OF THE STUDY AREA

3.1 Background

The Zarema May Day dam and irrigation project is located in Tekeze river basin in wolkayit woreda of west Tigray zone, Tigray national regional state. The proposed dam site is located in Zarema River at an altitude of around 1000 m a.s.l. Its location is roughly at 1518828 N Latitude and 370667 E Longitude. In geographical terms of spiking the command area lies between 1510000 & 1550000 UTM north and 331500 & 360600 UTM east .The elevation of the irrigation site is predetermined as it is between 800 & 930 m a.s.l. the extent of the surveyed land within the command area is about 45128 hectares. [13] The project area can be accessed in two ways: The first is 1030 km from Addis Ababa in North West direction along Addis Ababa-Gondar-Humera road & the second is 1260 km along Addis Ababa-Mekale-Shiraro road. The irrigation site contains relatively `gently sloppy topography with level and dissected plain, hills/ hillside slope and river terrace. However, the area outside the irrigation site is dissected piedmonts and high Relief mountains, escarpments, valley [13].

A zoned embankment dam is proposed with the height of 147 m to bound Zarema and Dukuko rivers for irrigation purpose. The material needed to construct the dam has an estimated volume of around $15*10^{6}$ m^3.

Picture – 3.1 *Trench along the dam axis of study area*

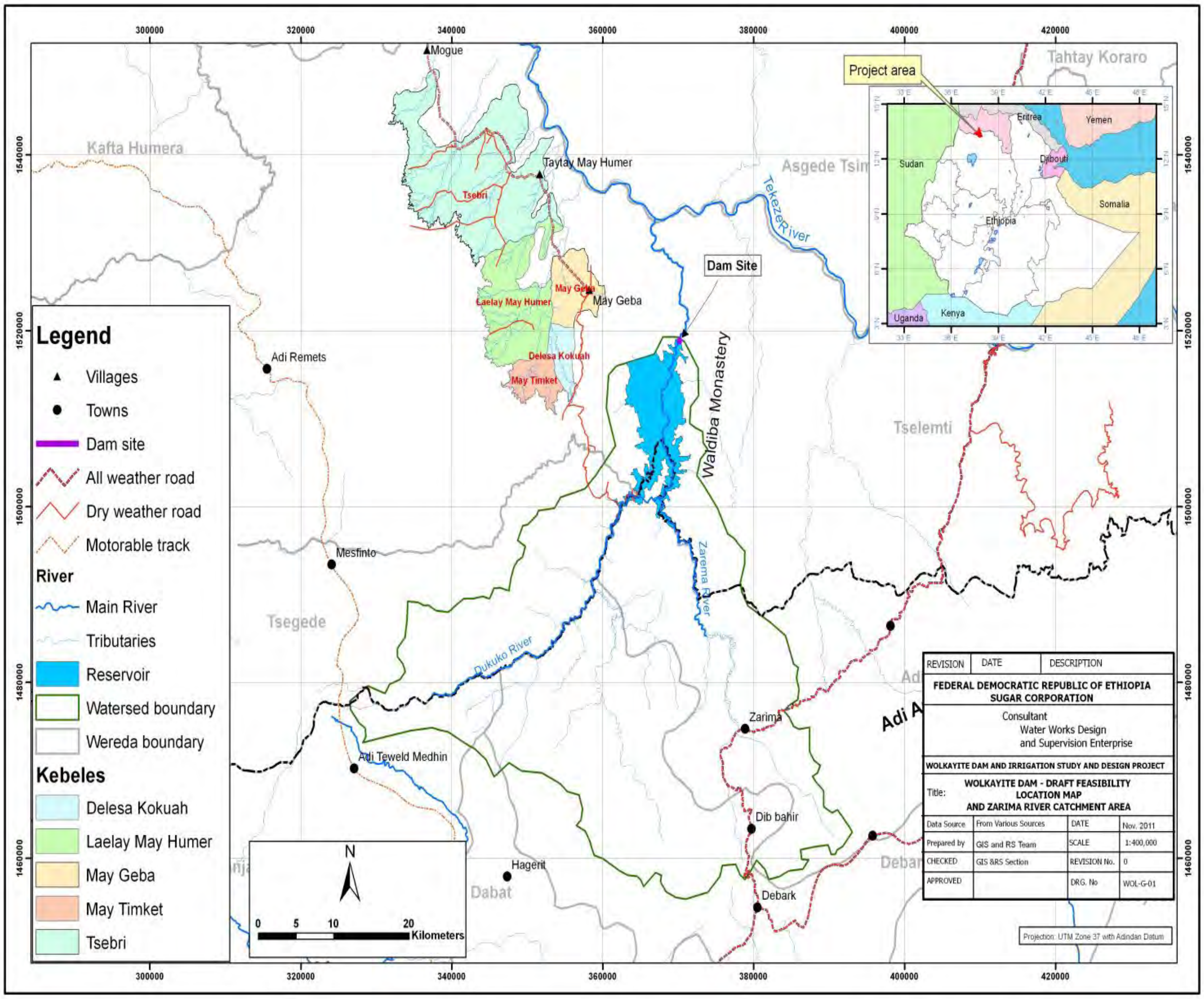

Figure 3.2 *Map of the study area [geology geotechnique final feasibility report by WWDSE, 2012]*

3.2 Dam type

A zoned embankment dam is proposed with the height of 143 m, crest length of 717.0m and it is laid on about 36.50ha of land. The reservoir inundates about 9,650ha of land at the normal flood level to bound Zarema and Dukuko rivers for irrigation purpose. The material needed to construct the dam has an estimated volume of around $15*10^{6}$ m^3. The main components of the project are: Main dam, cofferdam (which is part of main dam), one diversion conduit with a size of 4.0m having a length of 620.0m, side channel spillway at left side of the dam, separate outlet at the rim of the reservoir at about 6.2km from the dam.

The impervious material required is in huge quantity, but not available in close proximity to the dam site. Fatty clay material found near the dam is abundant, but not suitable to use as construction material. Hence at this stage of the project study regarding construction material is still ongoing and close analysis of the various dam option will continue through the next phase of the project. It is proposed that the dam type to be asphalt concrete core embankment dam having an embankment volume of 15MMC.The total command area of the proposed development at Wolkayite is in the order of 40,000 ha. The mean annual flow (water resource potential) of the Zarima river is 0.64BMC.The available water is not sufficient to develop 40,000 ha sugar cane only, so 25,000 ha sugar cane and 15,000 ha other crops are proposed to be developed. Based on this the reservoir capacity is determined. As a result the live storage is found 0.64BMC and dead storage capacity of 2.86BMC. Assessment of availability of various construction materials for different type of dams has been carried out for embankment and gravity dams.

Materials explored include impervious material for core, rock fill, rock for embankment, aggregate and riprap and filter and transition materials. [14] The design specification of materials in the dam section is listed below. Also refer fig 3.1 for the embankment cross section.

Asphaltic Concrete Mix Design and Properties

To be defined in detail in a further stage even if it can be assumed as a fuller type grain size distribution as for the cement concrete. The selected material will be the crushed Dolerite rock.

Filter-Transition zone

This material, at direct contact with the core for a width of 1.5 m, obtained as natural gravel or crushed rock like Dolerite, is a well graded sand and gravel to be compacted simultaneously with the asphalt core in 0.20 m thick layers. The material has to be compacted in 0.20 m thick layers (after compaction) with vibratory roller of minimum weight of 1.5 tons, with a number of passes ranging from 3 to 6 depending on the results gathered from trial embankment operations in order to achieve the specified dry density.

Table 3.1– Filter-transition zone material – average grain size distribution

Grain size (mm)	0.1	0.42	1	3.5	10	25	50
Percent passing (%)	0-2	4-8	6-14	20-30	50-60	80-90	96-100

Transition

This material, at direct contact with the transition-filter for a width of 4 m, is a well graded mix of sand, gravel, cobbles and boulders up to 200 mm size, to be compacted in 0.40 m thick layers. It operates as a transition between the filter-transition zone and the inner shell material zone and exerts the effective transfer of load between the two zones. The material has to be compacted in 0.40 m thick layers (after compaction) with vibratory roller of minimum weight of 15 tons, with a number of 4 passes if confirmed by the results gathered from trial embankment operations in order to achieve the specified dry density.

Table 3.2– Transition zone material – average grain size distribution

Grain size (mm)	0.85	3.3	6.3	12.5	25	50	100	150	200
Percent passing (%)	0-6	6-18	14-26	24-40	42-58	62-78	80-94	92-100	100

Free Draining inner Shell material

The material of this zone is to be especially well compacted, in order for its Deformation Modulus to be as high as possible and limit the decrease of its Strength parameters with strains negligible. The Material obtained by quarries and must be free-draining. The material has to be compacted in 0,80 m thick layers (after compaction) with vibratory roller of minimum weight of 15 tons, a number of 8 passes, if confirmed by the results gathered from trial embankment operations, in order to achieve the specified dry density.

Table 3.3 – Free draining inner shell zone material – average grain size distribution

Grain size (mm)	4.75	25	50	150	230	304	350
Percent passing (%)	0-10	16-30	30-44	58-72	74-86	86-100	100

Free Draining outer Shell material

The material obtained by quarries and must be free-draining. The material has to be compacted in 1,20 m thick layers (after compaction) with vibratory roller of minimum weight of 15 tons, a number of 6 passes if confirmed by the results gathered from trial embankment operations in order to achieve the specified dry density.

Table 3.4– Free draining outer shell zone material – average grain size distribution

Grain size (mm)	40	76	150	300	450	600	
Percent passing (%)	10	5-25	34-42	54-64	74-84	1	

3.3 Background of the problem

During detail site investigation; the proposed rock fill material prevails high degree of fines which doesn't fulfill the design specification. Shell material of the dam has to be pervious in order to decrease the pore water pressure within the dam body and it should have high shear strength. But, when weathered rock material selected as shell zone construction material it has its own problem in case of vibro-compaction during construction. This problem comes when the pore spaces are filled with fine shell material during compaction which leads to an increase in pore water pressure. Simultaneously the raise in pore water pressure directly leads to decrease in effective shear strength which decreases the total shear strength. However there are researches which show the suitability of weathered rock material in the construction of embankment dam using different approach and cross-section. Due to this the research conducted on these weathered material investigate their potential use in the construction of embankment Dam.

3.4 Climate

In terms of geographic co-ordinates, the watershed is located within the *Tekeze* River Basin described by 1453636 and 1532273 N Latitude and 327273 and 394545 E Longitude. The altitudes of the watershed range from 1000 to 3200 meters above sea level. Generally, the elevation increased from North to South. Mean annual rainfall varies between 590 and 1740 mm. Agro-ecologically the watershed classified as Hot to warm moist mountains, Tepid to Cool Moist Mountains and Plateau, Hot to warm semi-arid plains and Hot to warm sub-moist river gorges (SCRP database, 2000).

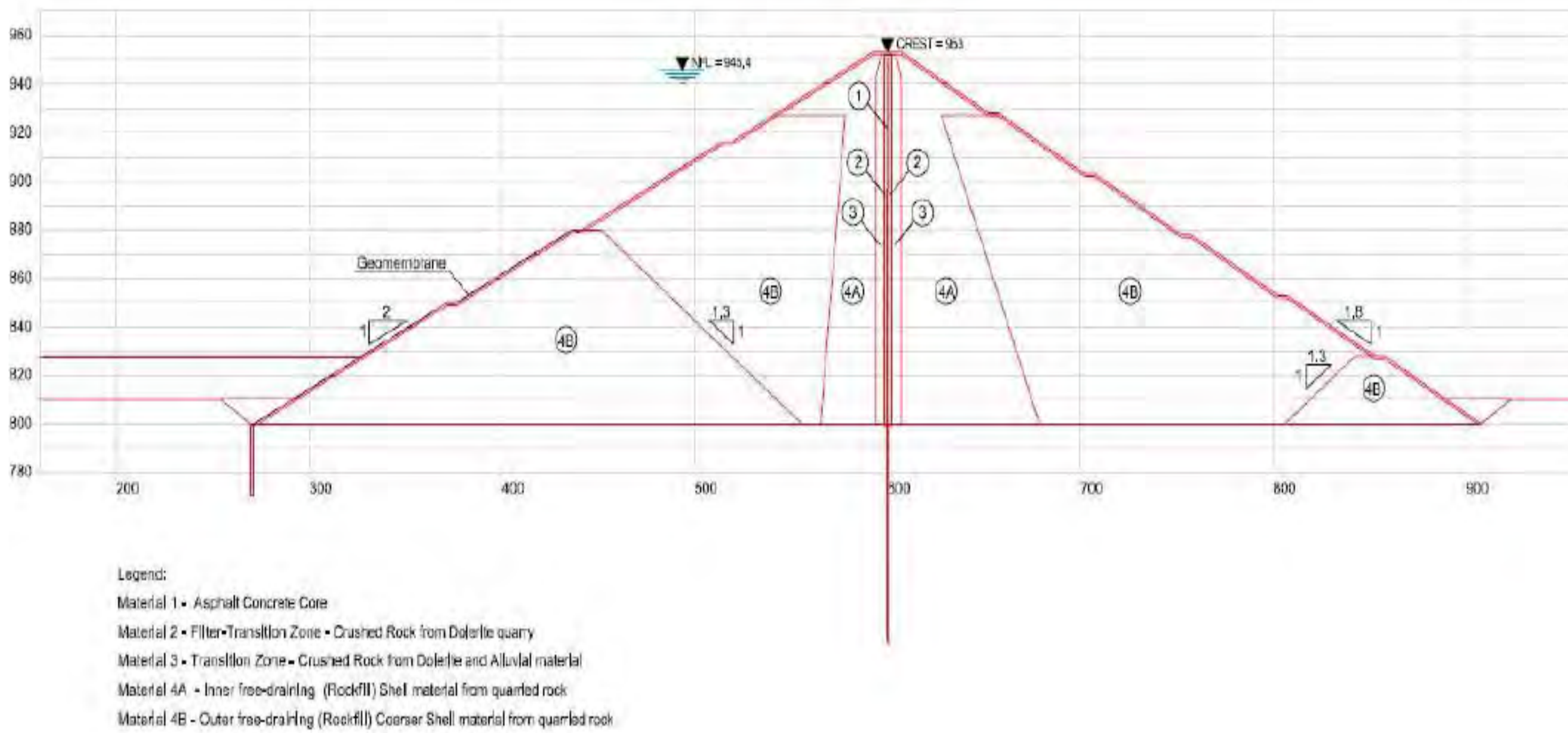

Figure 3.1 proposed section of Zarema May Day Dam [14]

3.4 Geology

Broadly the northern and northwestern Ethiopia is underlain by low-grade Precambrian basement assemblages, Phanerozoic sedimentary rock successions, and various Tertiary volcanic rocks (e.g. Hailu, 1979; Aguma and Kebede, 1982; Tefera et al. 1996; Tadesse, 1997).[13]

The Zarema May Day Dam site is located to the south of the Adi Hageray and Adi Nebrid tectono stratigraphic blocks to the south of Tekeze River. As such the metamafic-ultramafic rocks and associated meta-volcanic encountered in the dam site and surrounding areas can be interpreted as the southern extension of the Zager mafic-ultramafic belt.

Tectonically the area is located in southern part of the Arabian Nubian Shield in the East African Orogen; and as such the Precambrian basement rocks, which include the low-grade metavolcano-sedimentary assemblage. The Precambrian structures particularly the shear zones and thrust faults were reactivated during the Phanerozoic upliftment of east Africa as a result of which considerable thicknesses of the Paleozoic-Mesozoic sedimentary rock successions and Tertiary volcanic sequences with rare inter-volcanic sedimentary deposits were formed. The sedimentary successions and volcanic sequences then were affected by the reactivated older Precambrian structures as well as Phanerozoic structures. As such the area is located out of the Main Ethiopian rift system; however, the Atbara rift extends SE into northwestern Ethiopia where the project is located.

The reservoir area is underlain by the Precambrian basement rocks in the narrow valley portions of Zarema, and followed by Mesozoic sedimentary succession and Tertiary volcanic rocks Moreover, dolerite and basaltic dykes intrude the ridges (Tertiary volcanic rocks) that separate Zarema and Kalema catchments. Litho logically speaking Superficial deposits including residual soils, alluvial deposits adjacent to Zarema river and colluvial deposits are common. The residual soils are mainly black cotton soil and reddish brown lean clay. Moreover, dark brown and brick red lateritic soils are commonly exposed blanketing the sedimentary rock succession particularly around left abutment. [13]

CHAPTER 4

LABORATORY TEST RESULTS

4.1 General

Fill material is the key element in an embankment dam and the most demanding in terms of material quality and uniformity. Therefore the properties of the compacted embankment fill material should be investigated with utmost care and its parameters determined carefully to ensure long term water tight integrity.

A number of laboratory tests are conducted in SUR construction material laboratory at the project and the remaining shear and permeability test are conducted in AAiT Geotechnical laboratory on soil samples collected from dam site borrow areas identified in the sampling report SN-1 up to SN-10.

4.2 Specific gravity of Solids

Specific gravity of solids is determined by Pycnometer method. The value of Gs is required in subsequent analysis such as hydrometer analysis, consolidation and void ratio computations. The values are corrected for temperature to standard 20^{o}c.

Specific Gravity Laboratory test data and computations are given in Appendix3 – Table A-J

Table 4.1 *specific gravity results*

Soil type	Specific gravity
SN-1	2.70
SN-2	2.99
SN-3	2.77
SN-4	2.75
SN-5	2.75
SN-6	2.85
SN-7	2.65
SN-8	2.58
SN-9	2.80
SN-10	2.70

4.3 Particle size distribution

The particle size analysis is done through both sieve analysis and hydrometer analysis. Since samples are collected from weathered rock, particles within each sample are analyzed using dry sieve. Hydrometer analysis is conducted as per ASTM D2217-85[2] for which the percent finer than 75µm [sieve No 200] is greater than 10%.Sodium hexa met phosphate [40g/l] is used to disperse the soil suspension and appropriate temperature, meniscus and dispersing agent correction were applied to hydrometer readings.

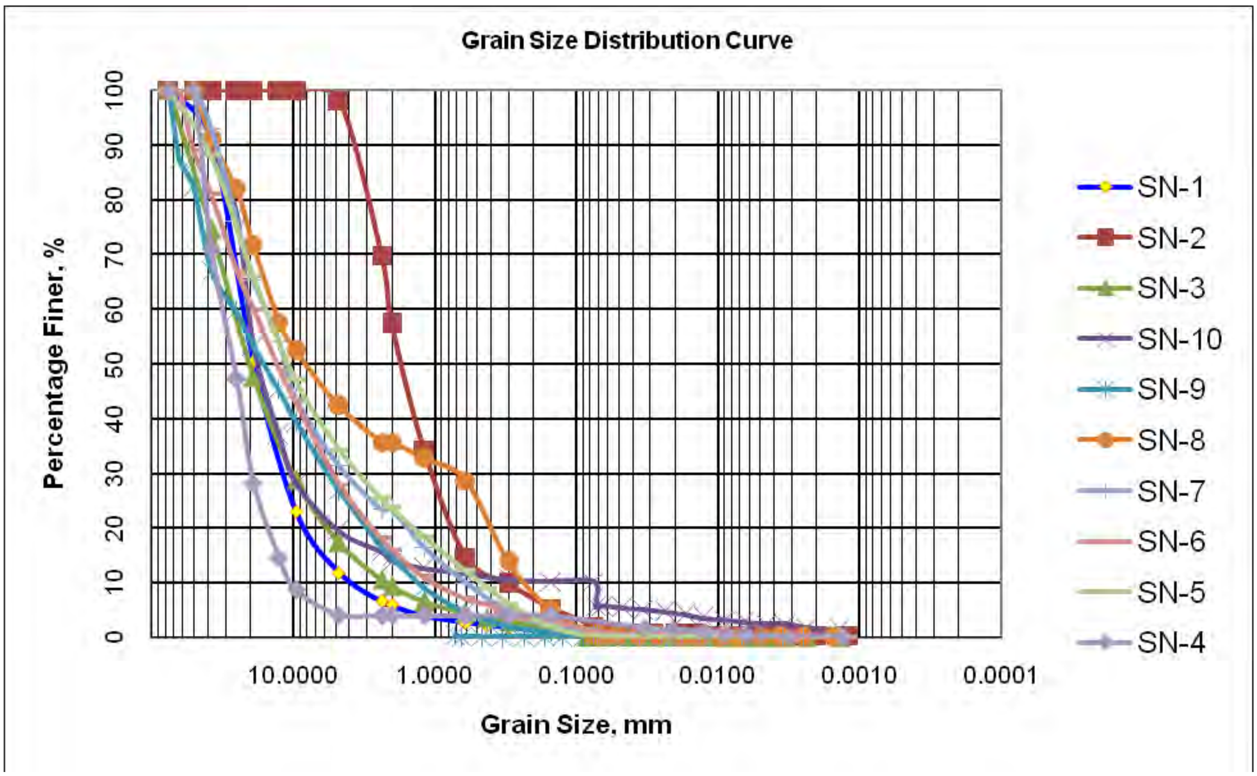

Figure-4.1Grain *size distribution curve*

4.4 Atterberg Limits

This laboratory test is performed to determine the plastic and liquid limits of a fine grained soil. They also help to understand the properties of these soils which are not attributed to their particle sizes. The Casagrande's apparatus was used to determine the liquid limit on air dried soil passing 425µm sieve. The plastic limit is also determined on the same sample. The values of liquid limit and plastic limit, plasticity index activity and USCS group are given table below.

Table - 4.2. *Atterberg limit test results*

Sample no.	Liquid limit	Plastic limit	Plasticity index
SN-1	36.2	27.5	8.7
SN-2	28.8	15.6	13.1
SN-3	32.3	21.2	11.1
SN-4	34.2	20.2	14.0
SN-5	26.1	18.2	7.9
SN-6	34.8	-	N.P
SN-7	34.3	21.1	13.2
SN-8	33.7	29.5	4.2
SN-9	28.0	26.3	1.7
SN-10	28.3	21.1	7.2

4.5 Compaction Test

Air dried soil sample is sieved through 4.75mm and 19mm ASTM sieves to determine which mold should be used for the compaction test. Since the material tested are weathered rock material with different degrees they have considerable amount of large proportion of gravel [>7%] therefore the modified proctor mold have been used. The values of maximum dry density [MDD] and optimum moisture content [OMC] are given below in table 4.3.

Table 4.3*- Compaction test results.*

Sample no.	Maximum dry density, MDD in g/cc	Optimum moisture content, OPM in %
SN-1	2.01	11.5
SN-2	2.82	7.3
SN-3	2.23	8.26
SN-4	1.89	14.0
SN-5	2.22	8.10
SN-6	2.4	5.5
SN-7	2.01	11.2
SN-8	1.97	9.70
SN9	2.19	7.40
SN-10	2.07	9.80

4.6 Permeability

Permeability of soils can be determined either directly or indirectly. The direct methods include constant head, falling head and consolidometer tests. The indirect methods are correlation equation particle size, and from consolidation data. The constant head test is used principally for course grained soils (clean sand and gravel) with K value greater than 10^{-5} m/sec. The falling head test is generally used for less pervious soils (fine sand and clays) with K values less than 10^{-5} m/sec. For the constant head test the soil sample is compacted in standard proctor mold at optimum moisture content and sample was soaked in water for one day before the test was conducted. The values of permeability coefficients are given in Table below.

Table 4.4. *Constant head permeability test results.*

Sample no	SN-1	SN-2	SN-3	SN-4	SN-5	SN-6	SN-7	SN-8	SN-9	SN-10
K in cm/s	$2.95*10^{-5}$	$1.43*10^{-4}$	$2.7*10^{-5}$	$2.2*10^{-5}$	$8.6*10^{-5}$	$2.15*10^{-3}$	$1.3*10^{-5}$	$1*10^{-4}$	$7.2*10^{-5}$	$2.9*10^{-5}$

4.7 Shear strength

A granular soil develops its shear strength as a result of the frictional and interlocking resistance between the individual soil particles. Granular soils, also known as cohesionless soils, can only be held together by confining pressures and will fall apart when the confining pressure is released. The drained shear strength is of most importance for granular soils. The shear strength of granular soils is often measured in the direct shear apparatus. By plotting the vertical pressure (σ'_n) versus shear stress at failure (τ_f), the effective friction angle (ϕ') and cohesion (C') can be obtained. Because the test specifications typically require the direct shear testing of soil in a saturated and drained state, the shear strength of the soil is expressed in terms of the effective parameters. Granular soils can also be tested in a dry state, and the shear strength of the soil is then expressed in terms of the friction angle (ϕ) and cohesion (C). Usually, the specimen is tested under consolidated-drained conditions. The specimen is allowed to fully consolidate under each applied normal pressure and is sheared at a rate slow enough to allow pore pressures to remain at equilibrium during shear. Therefore, test results are reported in terms of effective stress.

Sample preparation*:* Specimens shall be prepared using the compaction method, water content, and unit weight prescribed by the individual compaction test. Assemble and secure the shear box. Place a moist porous insert in the bottom of the shear box. Specimens may be molded by either kneading or tamping each layer until the accumulative mass of the soil placed in the shear box is compacted to a known volume, or by adjusting the number of layers, the number of tamps per layer and the force per tamp. The top of each layer shall be scarified prior to the addition of material for the next layer. The tamper used to compact the material shall have an area in contact with the soil equal to or less than 1⁄2 the area of the mold. Determine the mass of wet soil required for a single compacted lift and place it in the shear box. Compact the soil until the desired unit weight is obtained. Continue placing and compacting soil until the entire specimen is compacted.

Table 4.5- *Shear strength test results [5]*

Sample Number	SN-1	SN-2	SN-3	SN-5	SN-6	SN-7	SN-8	SN-9	SN-10
C in kN/m^2	21.4	16	12.3	45.8	2.5	16.7	26.7	9.2	40.6
Φ in deg	28.1	27.8	30.2	16.1	34.4	25.6	17.8	27.9	24.5

4.8 Abrasion test

The objective of this test is to evaluate how the aggregate is sufficiently hard to resist the abrasion effect. This test give a measure of the resistance of aggregate to surface wears by abrasion. The most widely used abrasion test is the Los Angeles abrasion test. Where aggregate sample is placed in steel drum with a number of steel balls of 4.8 mm diameter and the drum is set to rotate a specified number of times at a specified speed. The Los Angeles abrasion value is the percentage of fines passing the 1.7 mm (ASTM) sieve that gives the abrasion resistance of the aggregate. Soft aggregates are quickly ground to dust while hard aggregates lose little mass.

From the samples collected sample number 2 and 6 have relatively high amount of gravel in between 37.5 mm and 9.5 mm.

Table 4.6 - *Los Angeles abrasion test results[5]*

Sieve size (square openings)		Mass of indicated size, (g)		Wt. of sample to be tested (g)
passing	Retained on	Grading		
		A	B	
37.5 mm	25.0 mm	1250 ± 25		1250
25.0 mm	19.0 mm	1250 ± 25		1250
19.0 mm	12.5 mm	1250 ± 10	1250 ± 10	1250
12.5 mm	9.5 mm	1250 ± 10	1250 ± 10	1250

Test result analysis

Grading type used	SN-2		SN-6	
No. of trials	Trial $_1$	Trial $_2$	Trial $_1$	Trial $_2$
No. of revolution	500	500	500	500
Total Wt. of sample tested (W) gm	5000	5000	5000	5000
Wt. of sample retained on 1.70 mm sieve (X)	2806	2698	3667	3724
Loss in grams Y = (W-X)	2194	2302	1333	1276
Percent loss Z = (Y/W)*100	43.88	46.04	26.66	25.52
Average percent loss (trial $_1$ + trial $_2$) / 2	44.96 %		26.09 %	

CHAPTER 5

INTERPRETATION AND DISCUSSION

The laboratory test of particle size distribution curve based on ASTM D 422 and classification according to USCS indicates that the soil is in the range of well graded gravel to poorly graded gravel with little amount of silty clay. The first five samples collected from right abutment side of the dam have high amount gravely material depending on their degree of weathering. The materials shows high amount of fines at the upper layers of sampling trench. In general samples collected on the upstream and downstream of the dam on both sides of the abutment exhibit high amount of gravelly character at the upper most layers of the valley. As we go deeper into metavolcanic rock layers the grain size increases in to cobbles and boulders.

Table 5.1: *Summery outputs of sieve, Atterberg limits & shear parameters with engineering classification*

Sample no.	Specific gravity	Sieve analysis in %				c_c	c_u	Atterberg limits			Direct shear		Eng'g clasfn.
		cobble	gravel	sand	Silt/ clay			LL	PL	PI	C (KN/m^2)	Φ (deg)	
SN-1	2.70	nil	88.24	11.2	0.56	1.54	5.97	36.2	27.5	8.7	11.4	35.1	GW
SN-2	2.99	nil	88.59	9.95	1.45	1.80	6.51	28.8	15.6	13.1	6	34.8	GW
SN-3	2.77	nil	82.66	17.2	0.15	1.90	12.4	32.3	21.2	11.1	12.3	30.2	GC
SN-4	2.75	nil	81.75	16.55	1.70	1.19	3.11	34.2	14.0	20.2			GP
SN-5	2.75	nil	65.70	34.11	0.19	1.69	33.3	26.1	18.2	7.9	15.8	26.1	GM
SN-6	2.85	nil	72.18	26.92	0.90	1.32	16.5	34.8		N.P	0	38.4	GW
SN-7	2.65	nil	76.6	22.96	0.44	1.74	23.5	34.3	21.1	13.2	6.7	32.6	GM
SN-8	2.58	nil	64.35	33.85	1.8	0.19	58.6	33.7	29.5	4.2	16.7	25.8	GW
SN-9	2.80	nil	73.2	26.45	0.35	1.03	19.7	28.0	26.3	1.7	9.2	34.9	GW
SN-10	2.70	nil	64.3	25.49	10.14	-		28.3	21.1	7.2	20.6	24.5	GC

5.1 Permeability

Permeable materials are used in rolled embankment dams in internal filters , drains and transition zones to control seepage and piping. In the outer shells it also provide high strength to support the impervious core. Usually, pervious fill is free-draining cohesion less sand and gravel containing less than about 5 percent fines (material passing the 75-µm (No. 200) sieve).United states biro of reclamations (USBR) [5] describes soils with permeability less than 1× 10^{-6}cm/s

(1 ft/yr) considered as impervious, those with permeability between 1x10^{-6} cm/s and 1x10^{-4} cm/s (1 and 100 ft/yr) as semi pervious, and soils with permeability greater than 1× 10^{-4} cm/s (100 ft/yr) as pervious. According to USBR, the laboratory constant head permeability tests of sample SN-6 and SN-8 indicates they are under previous range and the rest are under semi-pervious range. These types of materials can be used as shell material and transition filter material according to the design recommendation.

5.2 Shear Strength

A granular soil develops its shear strength as a result of the frictional and interlocking resistance between the individual soil particles. Granular soils, also known as cohesionless soils, can only be held together by confining pressures and will fall apart when the confining pressure is released. According to USBR standards the material categorized under GW, GP, GM and GC should have friction angle greater than 33, 37, 34 and 31 degrees respectively [5]. These materials should be cohesionless or should have negligible cohesion. The direct shear test done on samples remolded at their optimum moisture content and maximum dry density shows certain deviation from the expected numerical values. Due to the mixing of certain residual soil from the upper part of the valley the samples show certain amount of cohesiveness, which also reduce the friction angle to some level. During the excavation of quarry, it is advisable to remove dirty rock fill from the upper layer.

5.3 Compaction

The compaction moisture content is very important in embankment fills to attain maximum dry density with appropriate compaction effort. The results of the test are within typical values associated with proctor mold compaction results of USCS soil groups. Table 5.2 compares test results and typical values.

Table 5.2 *Comparisons between USCS values and actual values.*

Sample no.	USCS group	MDD range in gm/cc	OMC range in %	Actual MDD in g/cc	Actual OMC In %
SN-1	GW	2.0-2.2	11-8	2.01	11.5
SN-2	GW	2.0-2.2	11-8	2.82	7.3
SN-3	GW	2.0-2.2	11-8	2.23	8.26
SN-4	GP	1.84-2.0	14-11	1.89	11.4

SN-5	GM	1.92-2.22	12-8	2.22	8.10
SN-6	GW	2.0-2.2	11-8	2.4	5.5
SN-7	GW	2.0-2.2	11-8	2.01	11.2
SN-8	GM	1.92-2.22	12-8	1.97	9.70
SN-9	GW	2.0-2.2	11-8	2.19	7.40
SN-10	GC	1.84-2.08	14-9	2.07	9.80

5.4 In-situ rock compressive strength

Table 5.3 *Unconfined compressive strength test result by Schmidt hammer*

Sample No.	Angle of measure	Hammer reading	U.C.S in Mpa
SN-1	0º	18	8
SN-2	0º	38	37
SN-3	0º	12	5
SN-4	45º	32	28.5
SN-5	0º	24	20
SN-6	-90º	66	69
SN-7	-90º	0	19
SN-8	45º	5	8
SN-9	0º	11	1
SN-10	0	20.5	0

In-situ rock compressive strength test using Schmidt hammer test indicates the range of the rock samples are from 0 up to 70 MPa. This gives information about rocks strength and durability which is important to zone the material depending on the design specification. Shell zone at upstream side of the dam needs high strength to resist the stress from reservoir. Design rock fill dams with earth fill water barriers using the principle of fill-material zoning. Such dams are economical when all type of locally available rock of different strength can be used in the structure zoning. The weaker rock is placed in less critical zones under less stress and hard sound rock is used where greater strength is required. Rocks with compressive strength of 70 MPa-200MPa are categorized as high compressive strength, 17MPa to 70MPa medium compressive strength and 3.5MPa to 17MPa are categorized as low compressive strength.

CHAPTER 6

CONCLUSION AND RECOMMENDATION

Through the evaluation of 10 selected weathered metavolcanic rock samples the following conclusions have been drawn. The rocks exhibit certain amount of fines depending on their degree of weathering.

Samples SN-1, SN-2 and SN-3 are GW soil group which are highly recommended for the inner shell of zoned rock fill dam. According to USBR their permeability range lies in semi-pervious category. Even if their particle size distribution curve indicates that they have high amount of gravel, the amount of fines increase during quarry and placement compaction. Refer table 4.4

Samples SN-4, SN-5 and SN-7 are GP, GM and GM respectively. These materials contain considerable amount of sand and silt material, which can be considered as dirty rock fill. Due to considerable amount of silt and sand these materials may be used as a transition filter according to their considerable amount of silt and sand. Refer figure 4.2

Samples SN-6 and SN-8 are GW soil group are recommended for outer shell because they are free draining material with the permeability greater than 10^{-4} cm/s. Gradation of theses material can be processed based on the design specification. Field unconfined compression test and Los-Angeles abrasion test indicated that they are medium to strong range and durable rocks if they are used as outer shell zone. Refer table 4.4

Samples SN-10 is GC soil group. Such type of materials is recommended for homogeneous type of embankment dams. Also according to USBR desirability ratings, SN- 4 as core of zoned embankment dam

Recommendations

During material investigation for embankment dam it a right hand rule to ensure 200% of fill material proposed by the design. Therefore zoned type rock fill dam is recommended in areas where varieties of materials exist. The embankment should be zoned to use as much material as possible from required excavation and borrow areas with the shortest haul distance and least waste. Zoning should provide adequate impervious zone, transition zone between core and shell, seepage control and stability.

Staged construction is other measure required to dissipate high pore-water pressure more rapidly.

In general, placement water contents for most projects will fall within the range of 2% dry to 3% wet of OMC determined by standard compaction test. Alternatively, soils that are compacted dry of OPM exhibit a more rigid stress-strain behavior develop high construction strength and low pore-water pressure during construction and consolidate less than soils compacted wet of OMC. Selection of design densities while a matter of judgment should be based on the result of test fills or past experience with similar construction material and field compaction equipment.

Rock fill should not be placed in layers thicker than 60.69cm. As the maximum particle size of rock fill material decrease, the lift thickness should be decreased. Quarry run rock having an excess of fines can be passed over grizzly and fines placed next to the core. Fine rock zones should be placed in 30.48 cm to 45.72cm lift thickness. In the compaction of pervious material, allowable lift thickness of layer will be specified, either in the form of compacted thickness or placed thickness. Vibratory steel drum rollers in the mass range of 4500 kg to 9000kg are the best equipment for compacting pervious clean coarse grained soil.

Weathered rock materials can be used only in the downstream shell, which is not saturated. Here the boundary is set by the shear strength and cohesion which is required for slop stability. The downstream shell is often formed of materials of reduced quality is reflected by increased deformability or decreased permeability , both in comparison to the property of high quality rock fill material.

Depending on the grain size, hydraulic conductivity values and criteria needed in the respective zones of the dam; the following type of zoned type of rolled earth fill dam is chosen to accommodate the material found at different level of weathering. Fig 6.1 below shows only the possible layout of material.

Alluvial deposit can be placed in the outer shell since they are free draining and durable material.

Based on engineering property of the material and the criteria needed at each section of the dam, the materials are allocated in all sections. SN-10 and SN-5 are good materials for core of the dam. SN-6 is durable and free draining material can be placed in the outer shell. GM and GP materials can be used as a transition filters. The rest of GW materials can be placed in the downstream section as inner and outer shell.

Limitations

1. Lack of large scale tri-axial apparatus to include large grain size of parent rock fragments in a certain amount of sample. Also, the shear strength parameters determined using direct shear test apparatus doesn't simulate the saturated condition during reservoir full condition.
2. A compaction test result doesn't include the effect of compaction effort on the weathering process of embankment fill.
3. Lack of test apparatus for transitional material like material in between soil and rock to check their engineering property

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APPINDEX 1

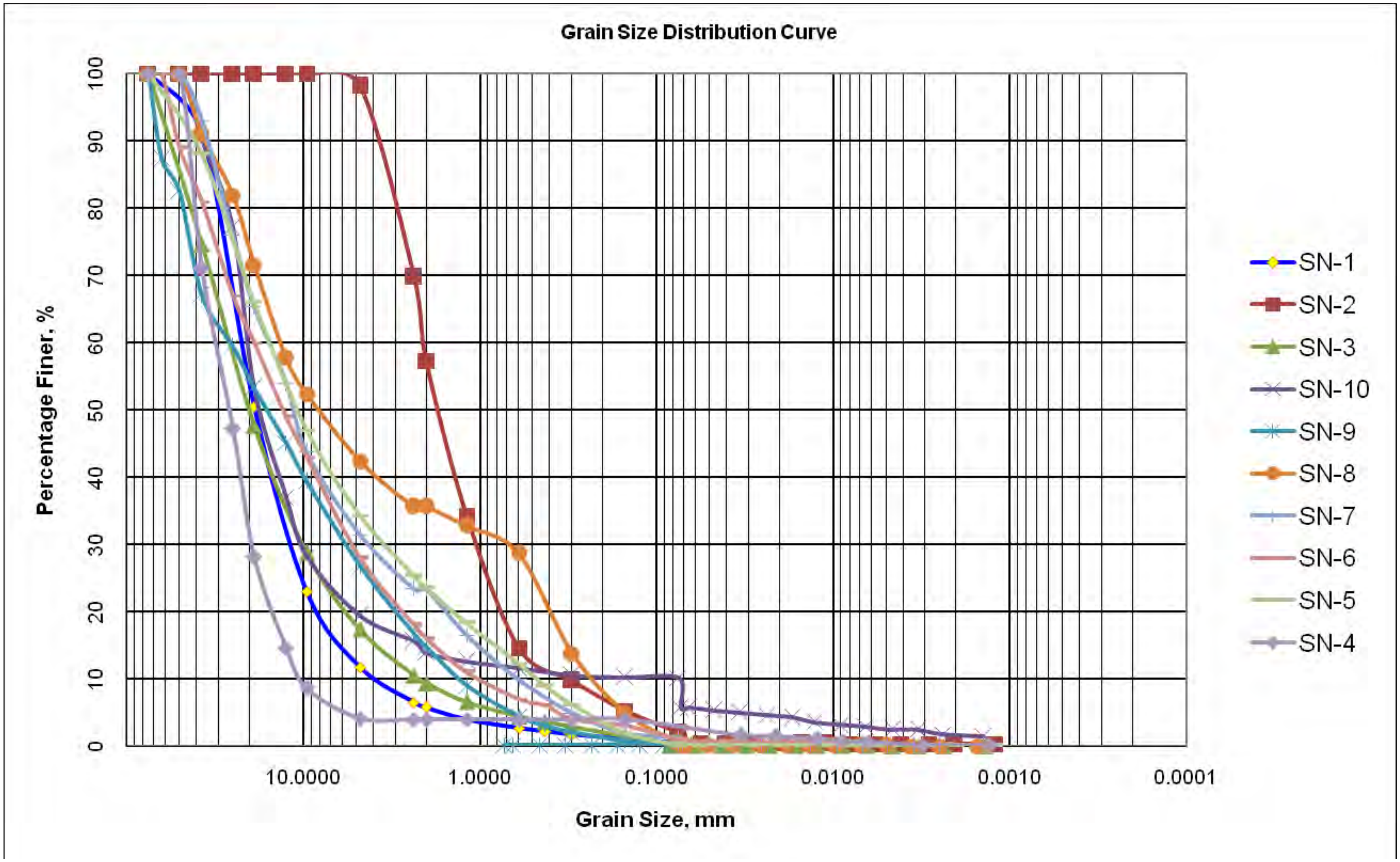
GRAIN SIZE ANALYSIS

PARTICLE SIZE DISTRIBUTION													
SN-1			SN-2			SN-3			SN-4			SN-5	
sieve size(mm)	percent finer		sieve size(mm)	percent finer		sieve size(mm)	percent finer		sieve size(mm)	percent finer		sieve size(mm)	percent finer
75	100		75	100		75	100		75	100.00		75	100
37.5	91	SEIVE ANALYSI	37.5	89	SEIVE ANALYSI	37.5	75	SEIVE ANALYSI	50	100.00		37.5	88
19	50		19	45		19	48		37.5	71.01		19	66
9.5	23		9.5	20		9.5	29		25	47.27		9.5	47
4.75	11.8		4.75	11.4		4.75	17.3		19	28.20		4.75	34.3
2.36	6.5		2.36	7.5		2.36	10.5		12.5	14.63		2.36	25.3
2	5.8		2	6.9		2	9.4		9.5	8.64		2	23.6
1.18	3.9		1.18	5.5		1.18	6.6		4.75	4.11		1.18	18.3
0.6	2.7		0.6	4.3		0.6	4.4		2.36	4.00		0.6	12.2
0.425	2.2		0.425	3.8		0.425	3.6		2	3.97		0.425	9.0
0.3	1.7		0.3	3.3		0.3	2.9		1.18	3.97		0.3	6.2
0.15	1.1		0.15	2.2		0.15	1.5		0.6	3.97		0.15	1.9
0.075	0.6		0.075	1.4		0.075	0.2		0.3	3.97		0.075	0.2
0.0714	0.39	HYDROMETER ANALYSI S	0.0725	0.27		0.0706	0.11		0.15	3.97		0.0737	0.17
0.0615	0.37		0.0615	0.26	HYDROMETER ANALYSI S	0.0599	0.11	HYDROMETER ANALYSI S	0.075	3.97		0.0564	0.14
0.0435	0.37		0.0435	0.26		0.0423	0.11		0.0329	1.70		0.0402	0.14
0.0311	0.35		0.0311	0.24		0.0307	0.10		0.0210	1.47		0.0283	0.13
0.0226	0.32		0.0226	0.22		0.0228	0.08		0.0123	1.08		0.0201	0.13
0.0165	0.27		0.0165	0.18		0.0163	0.07		0.0088	0.77		0.0150	0.11
0.0124	0.23		0.0124	0.16		0.0122	0.06		0.0063	0.62		0.0112	0.10
0.0090	0.18		0.0090	0.12		0.0089	0.05		0.0031	0.15		0.0080	0.09
0.0065	0.14		0.0065	0.10		0.0064	0.04		0.0013	0.08		0.0057	0.09
0.0046	0.12		0.0046	0.08		0.0046	0.03					0.0041	0.09
0.0033	0.09		0.0033	0.06		0.0033	0.02					0.0029	0.09
0.0024	0.07		0.0024	0.05		0.0024	0.02					0.0020	0.09
0.0014	0.05		0.0014	0.04		0.0014	0.01					0.0012	0.08

APPINDEX 1 – TABLE A

PARTICLE SIZE DISTRIBUTION CURVE									
SN-6		SN-7		SN-8		SN-9		SN-10	
sieve size(mm)	percent finer	sieve size(mm)	percent finer	sieve size(mm)	percent finer	sieve size(mm)	percent finer	sieve size(mm)	percent finer
75	100	75	100	75	100.00	75	100.0	75	100.00
63	100	50	100	50	100.00	63	87.5	50	100.00
50	89	37.5	93	37.5	91.07	50	82.5	37.5	90.14
37.5	81	25	76	25	81.91	37.5	67.1	25	77.05
25	67	19	65.3	19	71.62	25	59.4	19	53.25
19	60	12.5	54.0	12.5	57.70	19	53.2	12.5	37.01
12.5	49	9.5	43.7	9.5	52.40	12.5	45.1	9.5	28.24
9.5	43	4.75	31.4	4.75	42.36	9.5	39.5	4.75	19.41
4.75	28	2.36	23.4	2.36	35.65	4.75	26.8	2.36	15.63
2.36	18	2	23.4	2	35.65	2.36	16.6	2	13.88
2	16	1.18	16.5	1.18	32.75	2	14.8	1.18	12.61
1.18	11	0.6	9.9	0.6	28.68	1.18	9.0	0.6	11.61
0.6	7	0.3	4.9	0.3	13.79	0.6	4.4	0.3	10.53
0.425	6	0.15	2	0.15	4.91	0.425	3.1	0.15	10.21
0.3	4	0.075	1.3	0.075	0.56	0.3	2.1	0.075	10.14
0.15	3	0.0748	0.41	0.0724	0.18	0.15	0.9	0.0727	5.68
0.075	1	0.0711	0.41	0.0674	0.18	0.075	0.3	0.0651	5.68
0.0726	0.67	0.0504	0.39	0.0512	0.18	0.7192	0.16	0.0465	5.32
0.0649	0.67	0.0358	0.38	0.0366	0.16	0.6506	0.15	0.0333	4.96
0.0464	0.62	0.0253	0.37	0.0259	0.16	0.4600	0.15	0.0238	4.61
0.0328	0.62	0.0179	0.37	0.0185	0.14	0.3289	0.14	0.0170	4.25
0.0234	0.57	0.0133	0.29	0.0136	0.13	0.2326	0.14	0.0127	3.53
0.0167	0.52	0.0095	0.25	0.0097	0.11	0.1662	0.13	0.0091	3.18
0.0124	0.46	0.0068	0.20	0.0070	0.07	0.1240	0.11	0.0065	2.82
0.0088	0.41	0.0049	0.16	0.0050	0.05	0.0895	0.09	0.0046	2.46
0.0063	0.33	0.0035	0.12	0.0036	0.03	0.0639	0.07	0.0033	2.46
0.0045	0.26	0.0025	0.08	0.0025	0.03	0.0456	0.06	0.0024	1.75
0.0032	0.26	0.0014	0.04	0.0015	0.02	0.0329	0.04	0.0014	1.39
0.0023	0.20					0.0232	0.04		
0.0013	0.15					0.0135	0.03		

APPINDEX 1-TABLE B

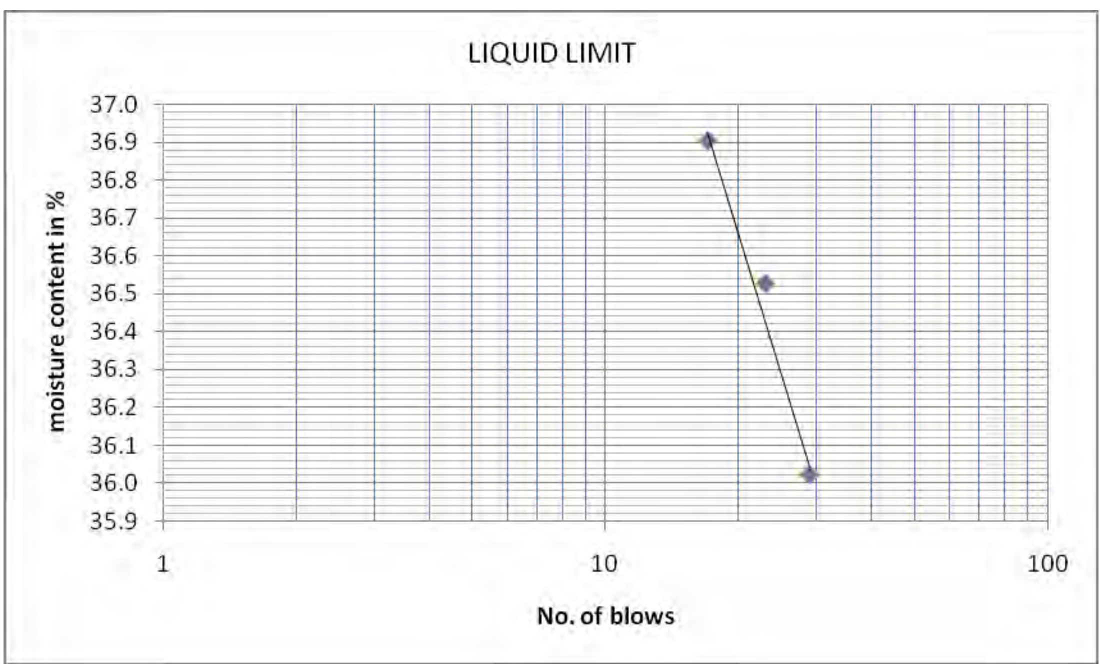


APPINDEX 1- FIGURE A

APPINDEX 2

ATTERBERG LIMITS

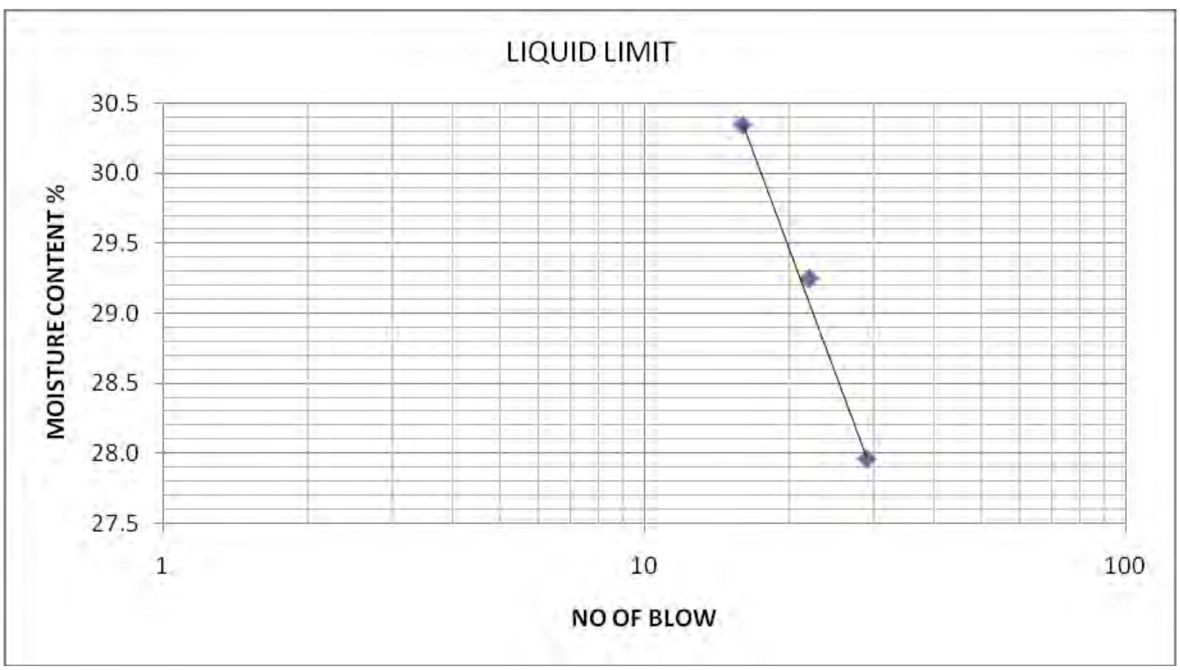
Atterberg limits					
sample code SN-1					
LIQUID LIMIT					Remarks
No. of blows		29	23	17	
Container No.		A1	A6	BH-4	
Mass of wet soil + container (a)	g	44.4	41.7	40.3	
Mass of dry soil + container (b)	g	37.7	35.6	34.1	
Mass of container (c)	g	19.1	18.9	17.3	
Mass of moisture (a-b)	g	6.7	6	6.2	
Mass of dry soil (b-c)	g	18.6	16.7	16.8	
Moisture content (w=a-b/b-c x 100)	%	36.0	36.5	36.9	
Liquid limit		36.2			
PLASTIC LIMIT					
Container No.		X	12		
Mass of wet soil + container (a)	g	12.8	12.8		
Mass of dry soil + container (b)	g	12.1	12.10		
Mass of container (c)	g	9.6	9.5		
Mass of moisture (a-b)	g	0.7	0.70		
Mass of dry soil (b-c)	g	2.5	2.60		
Moisture content (w=a-b/b-c x 100)	g	28.0	26.9		
plastic limit		27.5			
plasticity index		8.7			



APPINDEX 2- FIGURE A

Atterberg limits					
sample code		SN-2			
LIQUID LIMIT					Remarks
No. of blows		29	22	16	
Container No.		PI-1	PI-2	A6	
Mass of wet soil + container (a)	g	43.7	40.2	42.1	
Mass of dry soil + container (b)	g	38.5	35.5	36.7	
Mass of container (c)	g	19.9	19.6	18.9	
Mass of moisture (a-b)	g	5.2	4.7	5.4	
Mass of dry soil (b-c)	g	18.6	15.9	17.8	
Moisture content (w=a-b/b-c x 100)	%	28.0	29.2	30.3	
Liquid limit		28.8			
PLASTIC LIMIT					
Container No.		12	N		
Mass of wet soil + container (a)	g	13.1	13.3		
Mass of dry soil + container (b)	g	12.6	12.80		
Mass of container (c)	g	9.5	9.5		
Mass of moisture (a-b)	g	0.5	0.50		
Mass of dry soil (b-c)	g	3.1	3.30		
Moisture content (w=a-b/b-c x 100)	g	16.13	15.15		
plastic limit		15.6			
plasticity index		13.1			

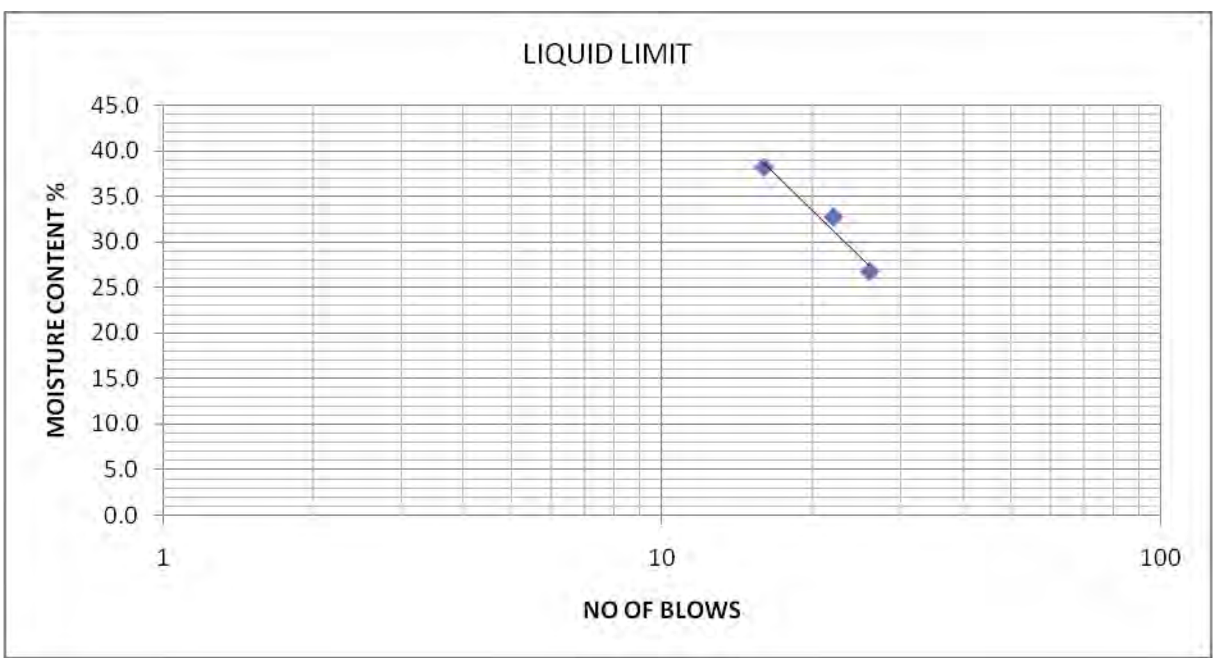
APPINDEX 2 – TABLE B



APPINDEX 2- FIGURE B

Atterberg limits					
sample code	SN-3				
LIQUID LIMIT					Remarks
No. of blows		26	22	16	
Container No.		BH-2	H8	W1	
Mass of wet soil + container (a)	g	43.5	40.5	41.9	
Mass of dry soil + container (b)	g	37.3	34.6	36	
Mass of container (c)	g	17.9	16.6	17.7	
Mass of moisture (a-b)	g	6.2	5.9	5.9	
Mass of dry soil (b-c)	g	19.4	18	18.3	
Moisture content (w=a-b/b-c x 100)	%	26.8	32.8	38.3	
Liquid limit		32.3			
PLASTIC LIMIT					
Container No.		5	A2		
Mass of wet soil + container (a)	g	14.7	14.5		
Mass of dry soil + container (b)	g	13.8	13.60		
Mass of container (c)	g	9.4	9.5		
Mass of moisture (a-b)	g	0.9	0.90		
Mass of dry soil (b-c)	g	4.4	4.10		
Moisture content (w=a-b/b-c x 100)	g	20.5	22.0		
plastic limit		21.2			
plasticity index		11.1			

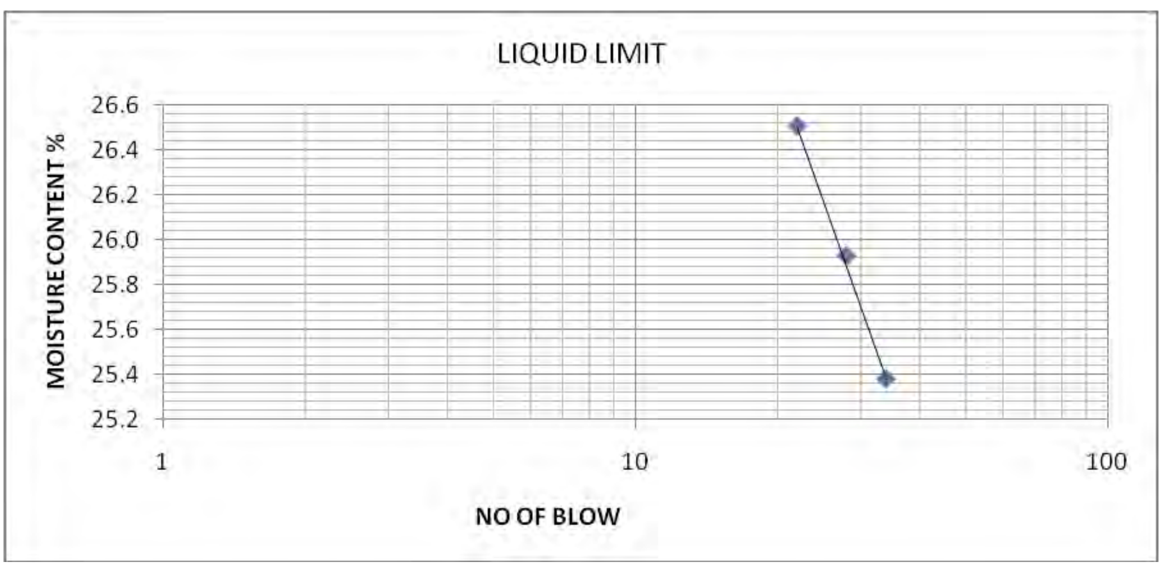
APPINDEX 2- TABLE C



APPINDEX 2 – FIGURE C

Atterberg limits					
sample code	SN-5				
LIQUID LIMIT					Remarks
No. of blows		34	28	22	
Container No.		23	MM	BH-4	
Mass of wet soil + container (a)	g	41.2	41	38.3	
Mass of dry soil + container (b)	g	36.2	36	33.9	
Mass of container (c)	g	16.5	16.6	17.3	
Mass of moisture (a-b)	g	5	5	4.4	
Mass of dry soil (b-c)	g	19.7	19.4	16.6	
Moisture content (w=a-b/b-c x 100)	%	25.4	25.9	26.5	
Liquid limit		26.1			
PLASTIC LIMIT					
Container No.		6	C		
Mass of wet soil + container (a)	g	12.9	16.8		
Mass of dry soil + container (b)	g	12.4	16.30		
Mass of container (c)	g	9.5	13.7		
Mass of moisture (a-b)	g	0.5	0.50		
Mass of dry soil (b-c)	g	2.9	2.60		
Moisture content (w=a-b/b-c x 100)	g	17.2	19.2		
plastic limit		18.2			
plasticity index		7.9			

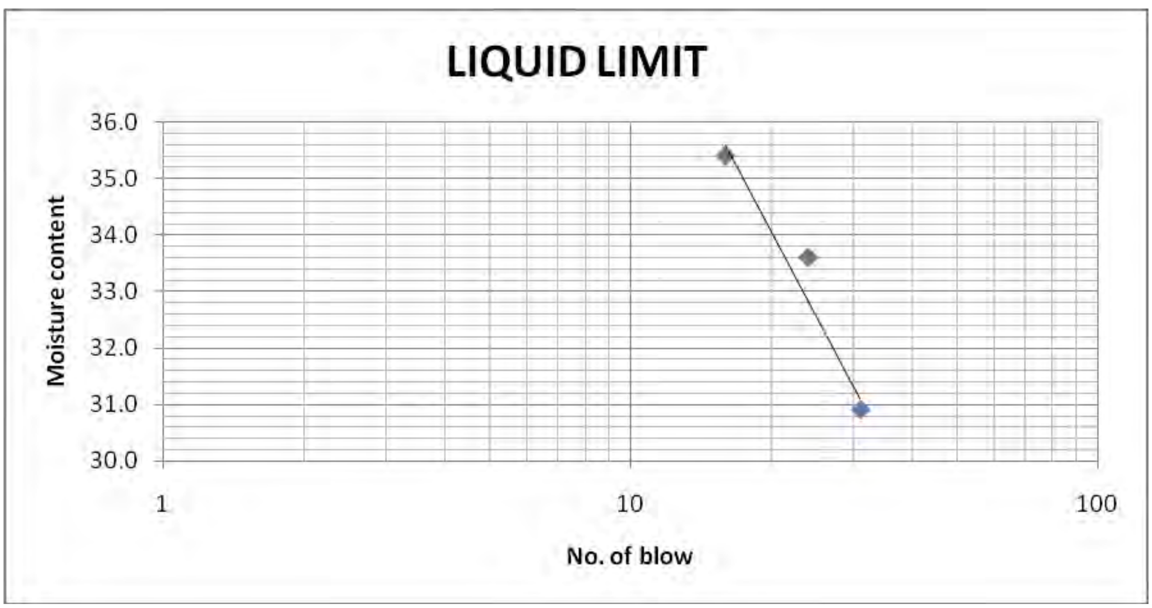
APPINDEX 2 – TABLE E



APPINDEX 2- FIGURE E

Atterberg limits						
sample code	SN-6					
LIQUID LIMIT						Remarks
No. of blows		31	24	16		
Container No.		M1	BH4	B9		
Mass of wet soil + container (a)	g	45.61	40.5	43.3		
Mass of dry soil + container (b)	g	38.5	34.6	36.7		
Mass of container (c)	g	18.4	17.3	18.8		
Mass of moisture (a-b)	g	7.11	5.9	6.6		
Mass of dry soil (b-c)	g	20.1	17.3	17.9		
Moisture content (w=a-b/b-c x 100)	%	25.4	25.8	26.5		
Liquid limit		33.5				
PLASTIC LIMIT						
Container No.						
Mass of wet soil + container (a)	g					
Mass of dry soil + container (b)	g					
Mass of container (c)	g					
Mass of moisture (a-b)	g					
Mass of dry soil (b-c)	g					
Moisture content (w=a-b/b-c x 100)	g					
plastic limit		NP				
plasticity index						

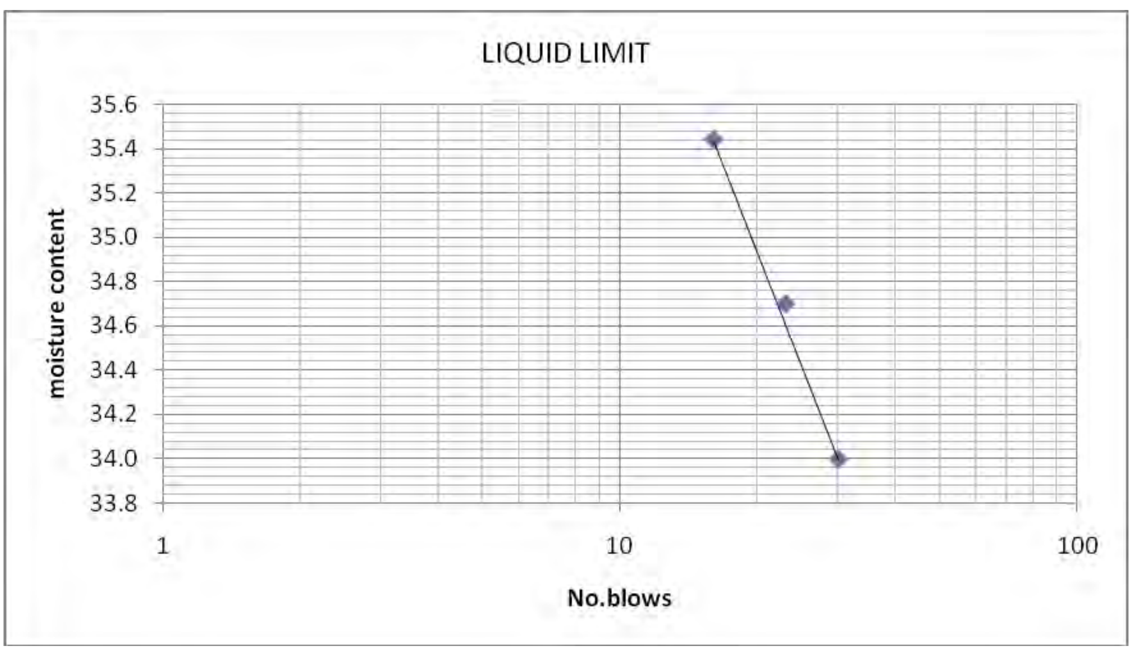
APPINDEX 2 – TABLE F



APPINDEX 2- FIGURE F

Atterberg limits					
sample code		SN-7			
LIQUID LIMIT					Remarks
No. of blows		30	23	16	
Container No.		A6	BH-4	MM	
Mass of wet soil + container (a)	g	41.01	40.6	38	
Mass of dry soil + container (b)	g	35.4	34.7	32.4	
Mass of container (c)	g	18.9	17.3	16.6	
Mass of moisture (a-b)	g	5.61	5.9	5.6	
Mass of dry soil (b-c)	g	16.5	17.4	15.8	
Moisture content (w=a-b/b-c x 100)	%	34.0	34.7	35.4	
Liquid limit		34.3			
PLASTIC LIMIT					
Container No.		Q	12		
Mass of wet soil + container (a)	g	12.5	12.9		
Mass of dry soil + container (b)	g	12	12.30		
Mass of container (c)	g	9.6	9.5		
Mass of moisture (a-b)	g	0.5	0.60		
Mass of dry soil (b-c)	g	2.4	2.80		
Moisture content (w=a-b/b-c x 100)	g	20.8	21.4		
plastic limit		21.1			
plasticity index		13.2			

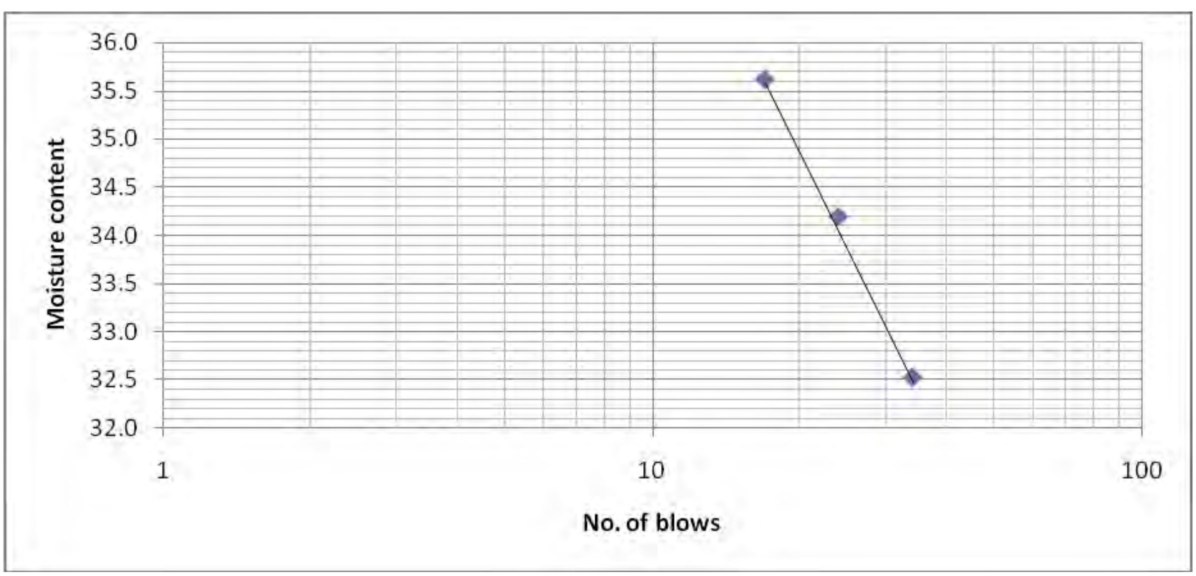
APPINDEX 2 – TABLE G



APPINDEX 2- FIGURE G

Atterberg limits					
sample code		SN-8			
LIQUID LIMIT					Remarks
No. of blows		34	24	17	
Container No.		BH-2	W1	A1	
Mass of wet soil + container (a)	g	39.9	39	38.4	
Mass of dry soil + container (b)	g	34.5	33.7	32.7	
Mass of container (c)	g	17.9	17.8	16.7	
Mass of moisture (a-b)	g	5.4	5.3	5.7	
Mass of dry soil (b-c)	g	16.6	15.9	16	
Moisture content (w=a-b/b-c x 100)	%	32.5	34.2	35.6	
Liquid limit		33.7			
PLASTIC LIMIT					
Container No.		X	C		
Mass of wet soil + container (a)	g	12.6	16.4		
Mass of dry soil + container (b)	g	11.9	15.80		
Mass of container (c)	g	9.6	13.7		
Mass of moisture (a-b)	g	0.7	0.60		
Mass of dry soil (b-c)	g	2.3	2.10		
Moisture content (w=a-b/b-c x 100)	g	30.4	28.6		
plastic limit		29.5			
plasticity index		4.2			

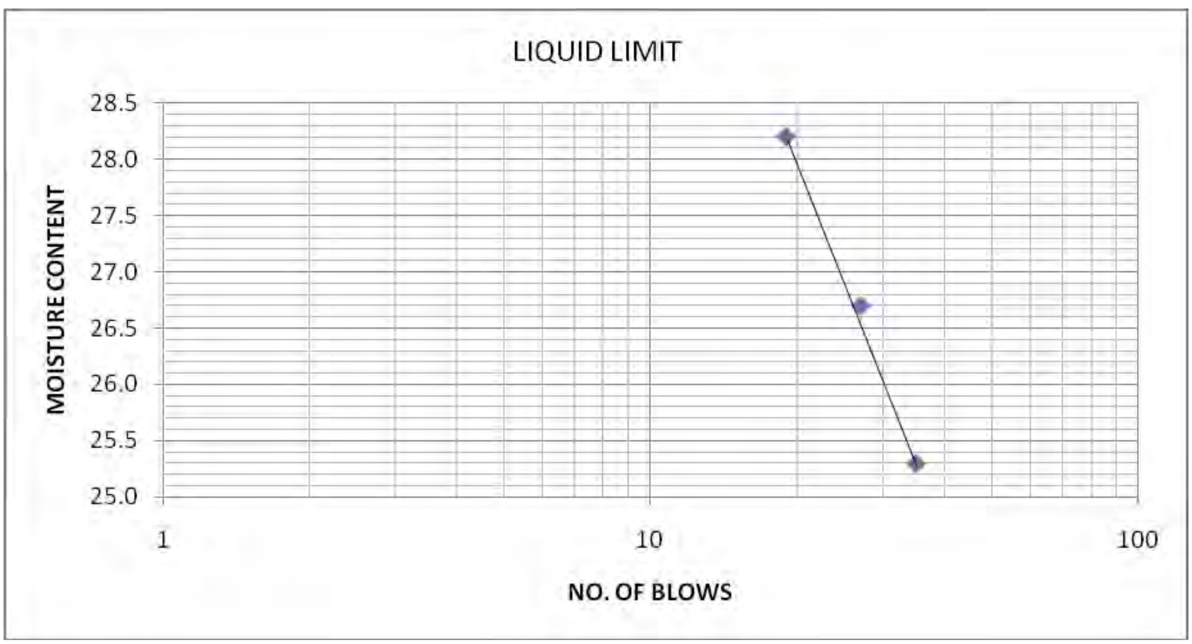
APPINDEX 2 – TABLE G



APPINDEX 2- TABLE G

Atterberg limits					
sample code	SN-9				
LIQUID LIMIT					Remarks
No. of blows		19	27	35	
Container No.		23	N2	12	
Mass of wet soil + container (a)	g	38.9	42.8	42.4	
Mass of dry soil + container (b)	g	34.4	37.9	37.1	
Mass of container (c)	g	16.6	19.4	16.6	
Mass of moisture (a-b)	g	4.5	4.9	5.3	
Mass of dry soil (b-c)	g	17.8	18.5	20.5	
Moisture content (w=a-b/b-c x 100)	%	25.3	26.5	28.2	
Liquid limit			28.0		
PLASTIC LIMIT					
Container No.		SH3	SH0		
Mass of wet soil + container (a)	g	13.3	13.7		
Mass of dry soil + container (b)	g	12.7	13.20		
Mass of container (c)	g	10.9	10.6		
Mass of moisture (a-b)	g	0.6	0.50		
Mass of dry soil (b-c)	g	1.8	2.60		
Moisture content (w=a-b/b-c x 100)	g	33.3	19.2		
plastic limit		26.3			
plasticity index		1.7			

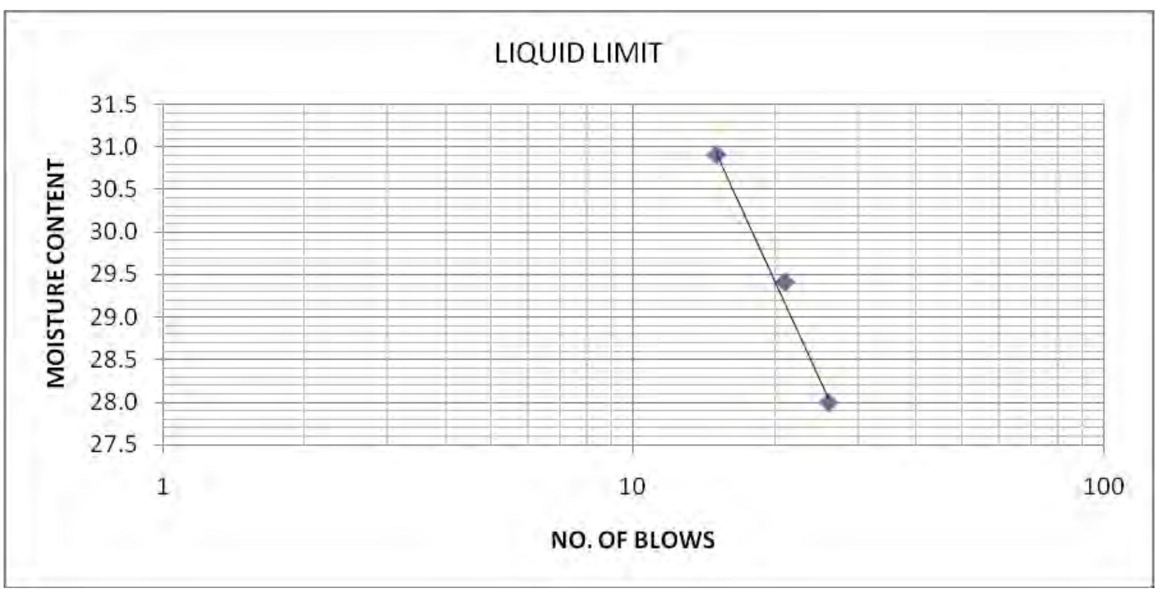
APPINDEX 2 –TABLE H



APPINDEX 2 – TABLE H

Atterberg limits						
sample code	SN-10					
LIQUID LIMIT						Remarks
No. of blows		26	21	15		
Container No.		B2	A1	NO		
Mass of wet soil + container (a)	g	41.1	38.8	41.3		
Mass of dry soil + container (b)	g	35.7	34.3	37.2		
Mass of container (c)	g	19.1	19	19.3		
Mass of moisture (a-b)	g	5.4	4.5	4.1		
Mass of dry soil (b-c)	g	16.6	15.3	17.9		
Moisture content (w=a-b/b-c x 100)	%	28.0	29.4	30.9		
Liquid limit		28.3				
PLASTIC LIMIT						
Container No.		SH-6	A5			
Mass of wet soil + container (a)	g	14.2	14.5			
Mass of dry soil + container (b)	g	13.6	13.90			
Mass of container (c)	g	10.7	11.1			
Mass of moisture (a-b)	g	0.6	0.60			
Mass of dry soil (b-c)	g	2.9	2.80			
Moisture content (w=a-b/b-c x 100)	g	20.7	21.4			
plastic limit		21.1				
plasticity index		7.2				

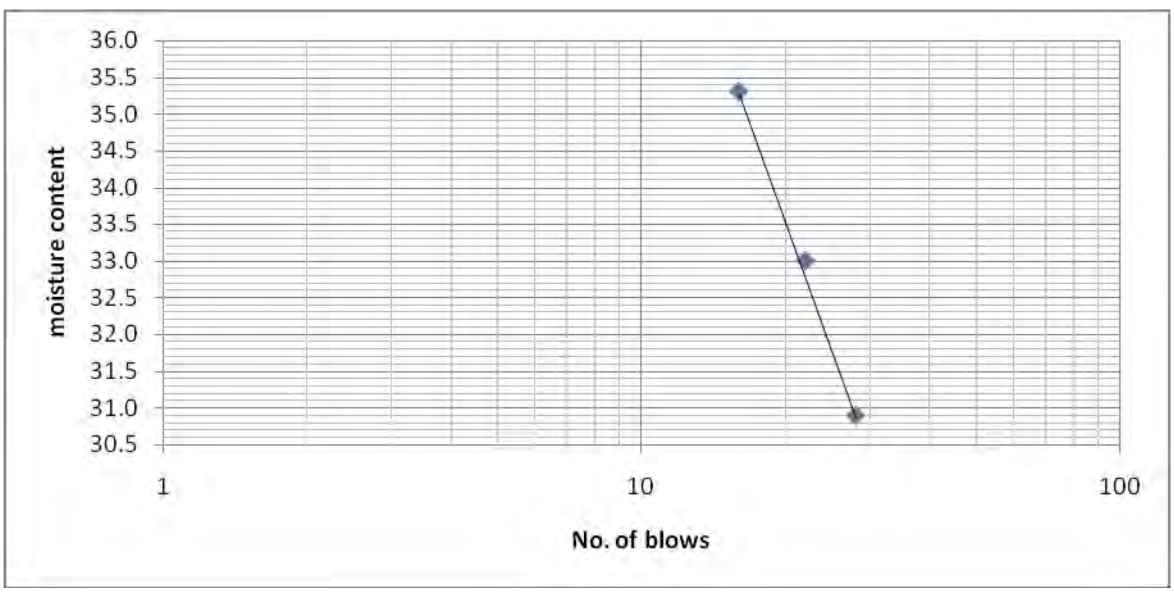
APPINDEX 2- TABLE I



APPINDEX 2- FIGURE I

Atterberg limits					
sample code	SN-4				
LIQUID LIMIT					Remarks
No. of blows		28	22	16	
Container No.		B2	W1	N2	
Mass of wet soil + container (a)	g	44.7	42.5	40.4	
Mass of dry soil + container (b)	g	38	36.1	34.6	
Mass of container (c)	g	19	17.7	19.5	
Mass of moisture (a-b)	g	6.7	6.4	5.8	
Mass of dry soil (b-c)	g	19	18.4	15.1	
Moisture content (w=a-b/b-c x 100)	%	35.3	34.8	30.9	
Liquid limit			34.2		
PLASTIC LIMIT					
Container No.		N3	BH-2		
Mass of wet soil + container (a)	g	22.6	21.7		
Mass of dry soil + container (b)	g	22.2	21.20		
Mass of container (c)	g	19.1	17.9		
Mass of moisture (a-b)	g	0.4	0.50		
Mass of dry soil (b-c)	g	3.1	3.30		
Moisture content (w=a-b/b-c x 100)	g	12.9	15.2		
plastic limit		14.0			
plasticity index		20.2			

APPINDEX2-TABLE J



APPINDEX2-FIGURE J

APPINDEX 3

SPECIFIC GRAVITY

Determination of specific gravity		
Sample No	**SN-1**	
Traials	**1st**	**2nd**
Bottle No	**P1**	**P3**
Weight of bottle(w1)	167.5	157.5
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	750.20	738.10
weight of bottle full of water(w4)	687.4	675.0
volume of bottle(w4+w2+)-w3	37.2	36.9
S.G	2.688	2.710
Average specific gravity	2.699	
Gs (20 ºc)	2.695	

APPINDEX 3 – TABLE A

Determination of specific gravity		
Sample No	SN-2	
Traials	**1st**	**2nd**
Bottle No	**P2**	**P1**
Weight of bottle(w1)	157.4	167.7
weight of oven dry sample (w2)	100.20	100.20
Weight of bottle+sample+water(w3)	749.60	738.40
weight of bottle full of water(w4)	674.8	687.2
volume of bottle(w4+w2+)-w3	25.4	49.0
S.G	3.945	2.045
Average specific gravity	2.995	
Gs (20 ºc)	2.990	

APPINDEX 3-TABLE B

Determination of specific gravity		
Sample No	**SN-3**	
Traials	**1st**	**2nd**
Bottle No	**P3**	**P4**
Weight of bottle(w1)	157.9	158.5
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	741.30	736.90
weight of bottle full of water(w4)	677.1	673.4
volume of bottle(w4+w2+)-w3	35.8	36.5
S.G	2.793	2.740
Average specific gravity	2.767	
Gs (20 ºc)	2.76	

APPINDEX 3- TABLE

Sample No	SN-4	
Bottle No	**P3**	**p4**
Weight of bottle(w1)	158	159.3
weight of oven dry sample	100.00	100.00
Weight of bottle+sample+water(w3)	741.50	737.60
weight of bottle full of water(w4)	677.8	673.9
volume of bottle(w4+w2+)-w3	36.3	36.3
S.G	2.755	2.755
Average specific gravity	2.755	

APPINDEX 3- TABLE D

Determination of specific gravity		
Sample No	**SN-5**	
Traials	**1st**	**2nd**
Bottle No	**P4**	**P3**
Weight of bottle(w1)	158.9	157.9
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	737.50	741.50
weight of bottle full of water(w4)	674.0	677.8
volume of bottle(w4+w2+)-w3	36.5	36.3
S.G	2.740	2.755
Average specific gravity	2.747	
Gs (20 ºc)	2.74	

APPINDEX 3- TABLE E

Determination of specific gravity		
Sample No	SN-6	
Traials	**1st**	**2nd**
Bottle No	**P1**	**P4**
Weight of bottle(w1)	167.9	159.2
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	752.50	739.00
weight of bottle full of water(w4)	687.5	674.2
volume of bottle(w4+w2+)-w3	35.0	35.2
S.G	2.857	2.841
Average specific gravity	2.849	
Gs (20 ºc)	2.847	

APPINDEX 3 –TABLE F

Determination of specific gravity		
Sample No	**SN-7**	
Trails	**1st**	**2nd**
Bottle No	**P3**	**P5**
Weight of bottle(w1)	158	158.3
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water (w3)	738.00	740.40
weight of bottle full of water(w4)	675.8	678.1
volume of bottle(w4+w2+)-w3	37.8	37.7
S.G	2.646	2.653
Average specific gravity	2.649	
Gs (20 ºc)	2.647	

APPINDEX 3- TABLE G

Determination of specific gravity		
Sample No	**SN-8**	
Traials	**1st**	**2nd**
Bottle No	**P4**	**P1**
Weight of bottle(w1)	159.2	167.9
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	735.60	748.70
weight of bottle full of water(w4)	674.2	687.5
volume of bottle(w4+w2+)-w3	38.6	38.8
S.G	2.591	2.577
Average specific gravity	2.584	
Gs (20 ºc)	2.582	

APPINDEX 3-TABLE H

Determination of specific gravity		
Sample No	SN-9	
Traials	**1st**	**2nd**
Bottle No	**P3**	**P5**
Weight of bottle(w1)	158	158.3
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	739.90	742.50
weight of bottle full of water(w4)	675.8	678.1
volume of bottle(w4+w2+)-w3	35.9	35.6
S.G	2.786	2.809
Average specific gravity	2.797	
Gs (20 ºc)	2.795	

APPINDEX 3- TABLE I

Determination of specific gravity		
Sample No	**SN-10**	
Traials	**1st**	**2nd**
Bottle No	**P4**	**P3**
Weight of bottle(w1)	159.2	167.9
weight of oven dry sample (w2)	100.00	100.00
Weight of bottle+sample+water(w3)	737.10	751.00
weight of bottle full of water(w4)	674.2	687.5
volume of bottle(w4+w2+)-w3	37.1	36.5
S.G	2.695	2.740
Average specific gravity	2.718	
Gs (20 ºc)	2.696	

APPINDEX 3- TABLE J

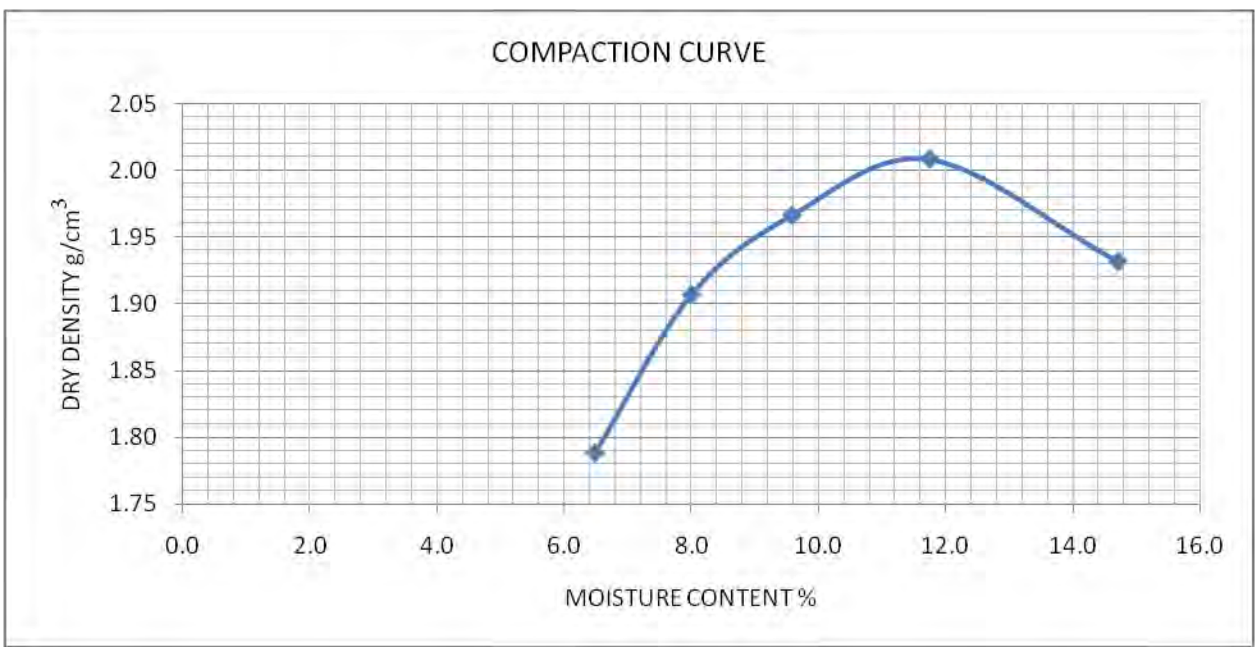
APPINDEX 4

COMPACTION TESTS

sample date Test date:-						
Type of Material-- SN-1						
Compaction Method: D						
COMPACTION CURVE						
Moisture content %	2%	4%	6%	8%	10%	
Mass of wet soil + mould A (g)	11359	11689	11892	12082	12021	
Mass of mould B (g)	7315	7315	7315	7315	7315	
Mass of wet soil C=A-B (g)	4044	4374	4577	4767	4706	
VOLUME OF MOULD (CC)	2124	2124	2124	2124	2124	NMC
Bulk density C / V = W	1.90	2.06	2.15	2.24	2.22	
Moisture determination container No.	B9	B16	B7	B6	B14	B12
Mass of container + wet soil a (g)	540.9	563.1	531.7	552.8	595.1	779.7
mass of container + dry soil b (g)	520	539.3	502.7	516	543.4	752.8
Mass of container d (g)	197.9	192.4	200.1	202.6	191.6	203.1
Mass of dry soil b - d = e (g)	322.1	346.9	302.6	313.4	351.8	549.7
Mass of moisture a - b = f (g)	20.9	23.8	29	36.8	51.7	26.9
Moisture content f/e*100 = m (%)	6.5	8.0	9.6	11.7	14.7	4.9
Dry density W / (100+m) *100 (g/cc)	1.79	1.91	1.97	2.01	1.93	0.00
MAXIMUM DRY DENSITY (g / cc)	**2.010**					
OPTIMUM MOISTURE CONTENT (%)	**11.50**					

APPINDEX 4- TABLE

A

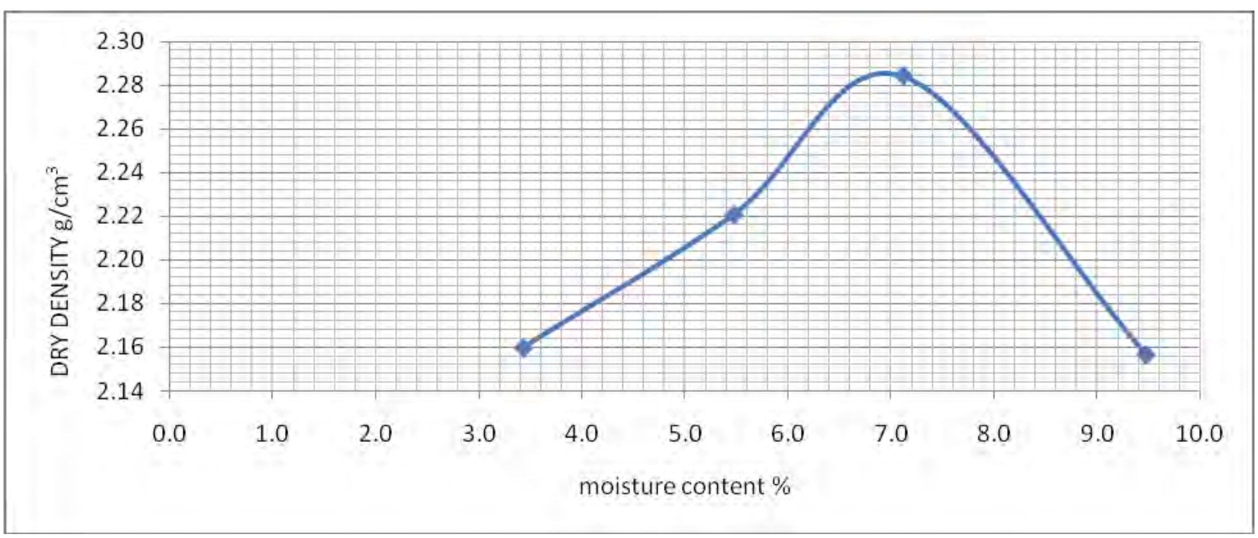


APPINDEX 4- FIGURE A

Sample date:- **Type of Material-- SN-2** **Test date:-** **Compaction Method: D**		

COMPACTION CURVE						
Moisture content	%	2%	4%	6%	8%	10%
Mass of wet soil + mould	A (g)	12060	12290	12512	12330	
Mass of mould	B (g)	7315	7315	7315	7315	
Mass of wet soil	C=A-B (g)	4745	4975	5197	5015	
Volume of mould	(cc)	2124	2124	2124	2124	
Bulk density	C / V = W	2.23	2.34	2.45	2.36	
Moisture determination container No.		R4	I	21	R5	
Mass of container + wet soil	a (g)	441.9	500.2	390.7	470.4	
mass of container + dry soil	b (g)	428.6	476.3	367.4	433.2	
Mass of container	d (g)	40.4	39.5	39.9	40.4	
Mass of dry soil	b - d = e (g)	388.2	436.8	327.5	392.8	
Mass of moisture	a - b = f (g)	13.3	23.9	23.3	37.2	
Moisture content	f/e*100 = m (%)	3.4	5.5	7.1	9.5	
Dry density	W / (100+m) *100 (g/cc)	2.16	2.22	2.28	2.16	
MAXIMUM DRY DENSITY (g / cc)		2.282				
OPTIMUM MOISTURE CONTENT (%)		7.30				

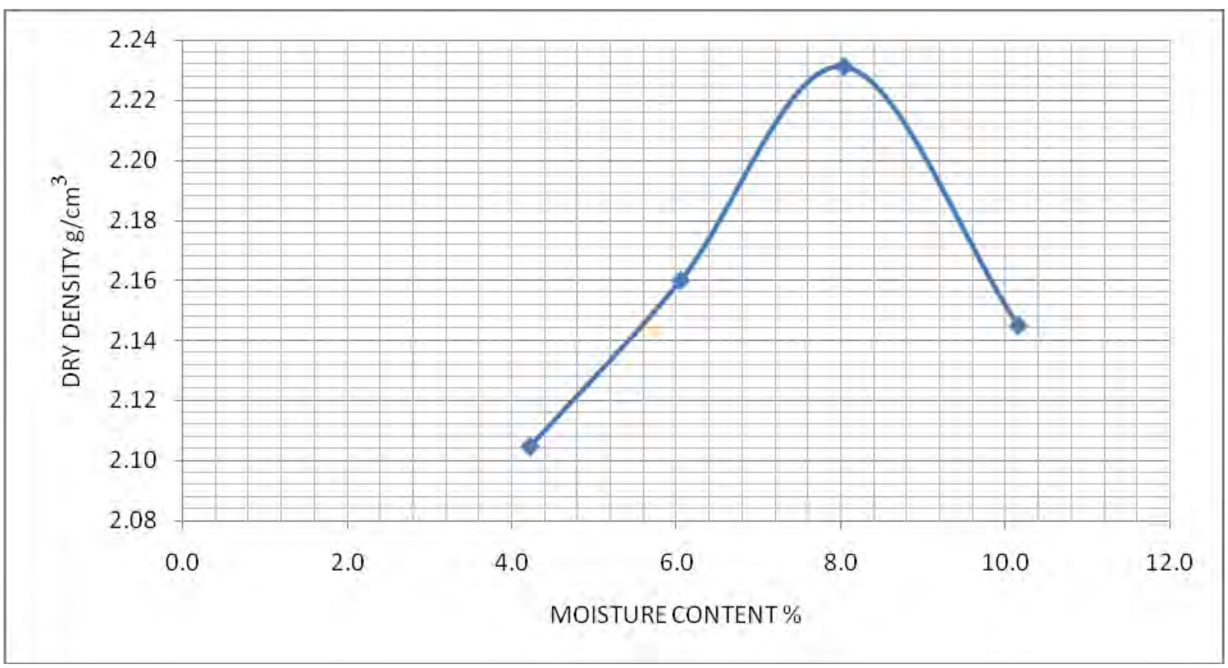
APPINDEX 4 – TABLE B



APPINDEX 4- FIGURE B

type of material :- SN-3 sample date tested date					
COMPACTION CURVE					
Moisture content %	2%	4%	6%	8%	
Mass of wet soil + mould A (g)	11974	12180	12434	12333	
Mass of mould B (g)	7315	7315	7315	7315	
Mass of wet soil C=A-B (g)	4659	4865	5119	5018	
volume of mould	2124	2124	2124	2124	NMC
Bulk density C / V = W	2.19	2.29	2.41	2.36	
Moisture determination container No.	91	B13	93	B41	B81
Mass of container + wet soil a (g)	657.7	655.7	556.5	610.4	672.7
mass of container + dry soil b (g)	638.9	629.4	530.2	572.2	663.8
Mass of container d (g)	193.1	194.3	202.5	195.7	200.2
Mass of dry soil b - d = e (g)	445.8	435.1	327.7	376.5	463.6
Mass of moisture a - b = f (g)	18.8	26.3	26.3	38.2	8.9
Moisture content f/e*100 = m (%)	4.2	6.0	8.0	10.1	1.9
Dry density W / (100+m) *100 (g/cc)	2.10	2.16	2.23	2.14	0.00
MAXIMUM DRY DENSITY (g / cc)	2.23				
OPTIMUM MOISTURE CONTENT (%)	8.26				

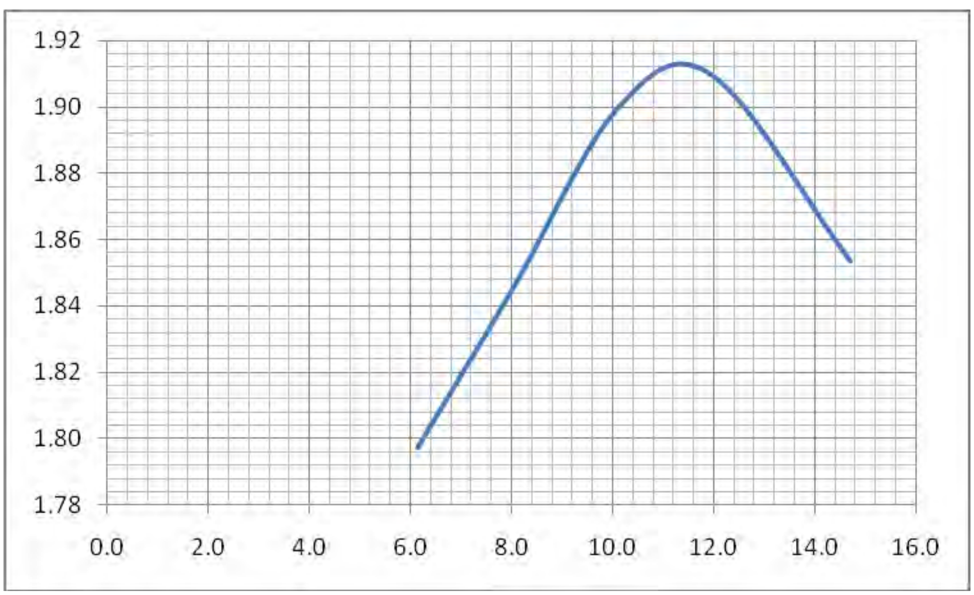
APPINDEX 4 – TABLE C



APPINDEX 4- FIGURE C

sample date	
Test date:-	
Type of Material-- SN-4	
Compaction Method: D	

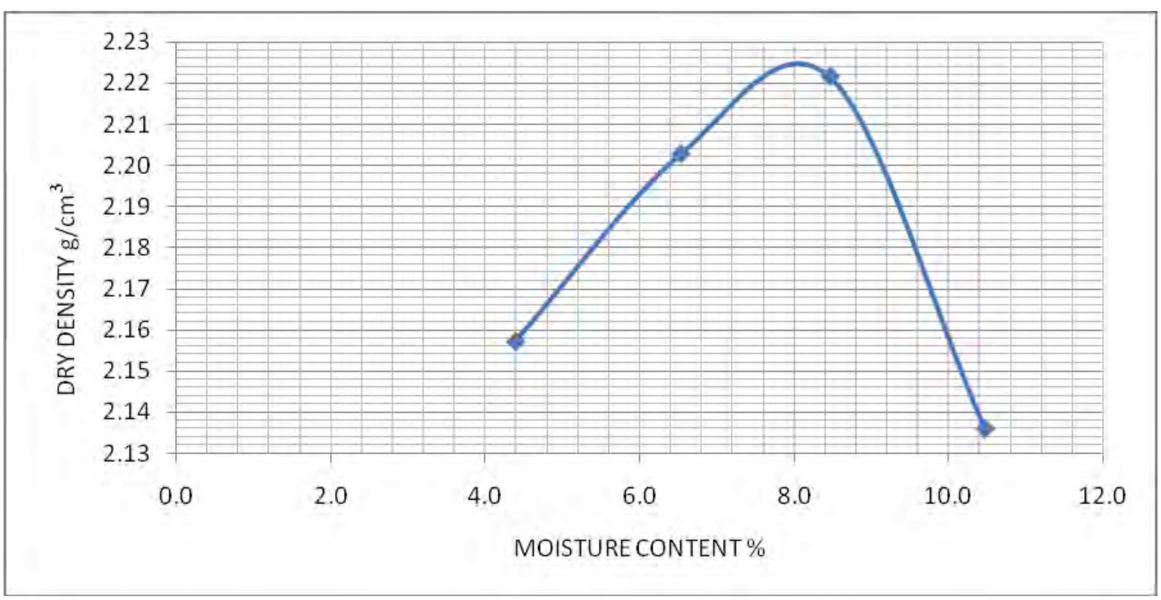
Moisture content %	0%	2%	4%	6%	8%
Mass of wet soil + mould A (g)	11366	11546	11758	11856	11830
Mass of mould B (g)	7315	7315	7315	7315	7315
Mass of wet soil C=A-B (g)	4051	4231	4443	4541	4515
VOLUME OF MOULD (CC)	2124	2124	2124	2124	2124
Bulk density C / V = W	1.91	1.99	2.09	2.14	2.13
Moisture determination container No.	R5	21	5	15	H
Mass of container + wet soil a (g)	345.1	347.9	322.4	346.11	340
mass of container + dry soil b (g)	327.5	325.1	296.7	313.6	302.7
Mass of container d (g)	40.5	39.9	42.3	41.6	48.7
Mass of dry soil b - d = e (g)	287	285.2	254.4	272	254
Mass of moisture a - b = f (g)	17.6	22.8	25.7	32.51	37.3
Moisture content f/e*100 = m (%)	6.1	8.0	10.1	12.0	14.7
Dry density W / (100+m) *100 (g/cc)	1.80	1.84	1.90	1.91	1.85
MAXIMUM DRY DENSITY (g / cc)	**1.890**				
OPTIMUM MOISTURE CONTENT (%)	**11.40**				



APPINDEX 4 –TABLE D

sample date Test date:-					
Type of Material-- SN-5					
Compaction Method: D					
Moisture content %	2%	4%	6%	8%	10%
Mass of wet soil + mould A (g)	12098	12299	12433	12327	
Mass of mould B (g)	7315	7315	7315	7315	
Mass of wet soil C=A-B (g)	4783	4984	5118	5012	
VOLUME OF MOULD (CC)	2124	2124	2124	2124	
Bulk density C / V = W	2.25	2.35	2.41	2.36	
Moisture determination container No.	T5	J	B	b1	
Mass of container + wet soil a (g)	394	412.6	347.7	425.9	
mass of container + dry soil b (g)	379.2	389.9	324.6	389.3	
Mass of container d (g)	42	42.2	51.7	39.7	
Mass of dry soil b - d = e (g)	337.2	347.7	272.9	349.6	
Mass of moisture a - b = f (g)	14.8	22.7	23.1	36.6	
Moisture content f/e*100 = m (%)	4.4	6.5	8.5	10.5	
Dry density W / (100+m) *100 (g/cc)	2.16	2.20	2.22	2.14	
MAXIMUM DRY DENSITY (g / cc)	**2.224**				
OPTIMUM MOISTURE CONTENT (%)	**8.10**				

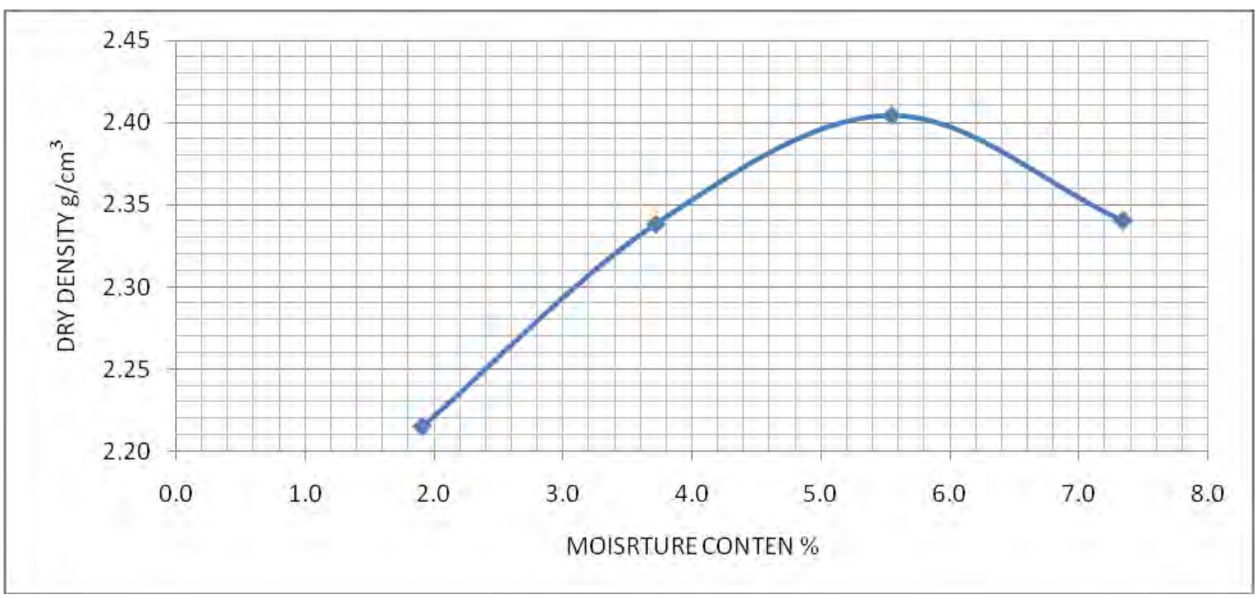
APPINDEX 4 –TABLE F



APPINDEX 4- FIGURE F

COMPACTION CURVE						
sample date Test date:-						
Type of Material-- SN-6						
Compaction Method: D						
Moisture content %	0	2%	4%	6%	8%	10%
Mass of wet soil + mould A (g)	12109	12466	12705	12651		
Mass of mould B (g)	7315	7315	7315	7315		
Mass of wet soil C=A-B (g)	4794	5151	5390	5336		
VOLUME OF MOULD (CC)	2124	2124	2124	2124		
Bulk density C / V = W	2.26	2.43	2.54	2.51		
Moisture determination container No.	R4	H6	I	21		
Mass of container + wet soil a (g)	409.6	420.1	355.5	444.8		
mass of container + dry soil b (g)	402.7	406.5	338.9	417.1		
Mass of container d (g)	40.6	41	39.8	40		
Mass of dry soil b - d = e (g)	362.1	365.5	299.1	377.1		
Mass of moisture a - b = f (g)	6.9	13.6	16.6	27.7		
Moisture content f/e*100 = m (%)	1.9	3.7	5.5	7.3		
Dry density W / (100+m) *100 (g/cc)	2.21	2.34	2.40	2.34		
MAXIMUM DRY DENSITY (g / cc)		**2.400**				
OPTIMUM MOISTURE CONTENT (%)		**5.50**				

APPINDEX 4- TABLE G



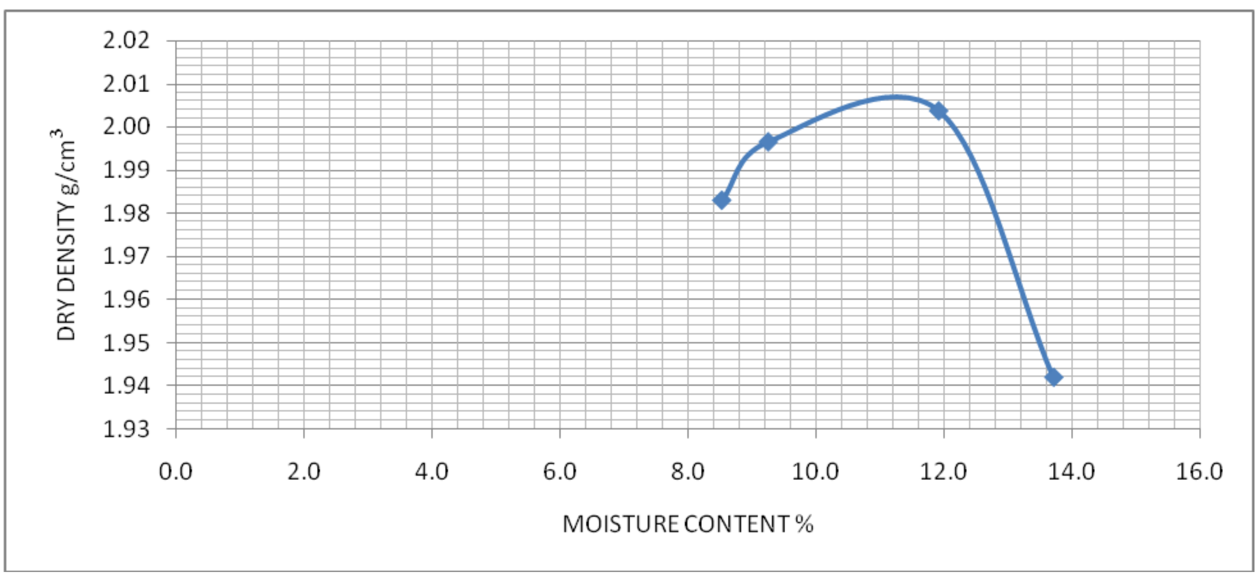
APPINDEX 4- FIGURE G

COMPACTION CURVE

sample date Test date:-	
Type of Material-- SN-7	
Compaction Method: D	

		2%	4%	6%	8%	10%
Moisture content	%	2%	4%	6%	8%	10%
Mass of wet soil + mould	A (g)	11695	11886	11948	12078	12005
Mass of mould	B (g)	7315	7315	7315	7315	7315
Mass of wet soil	C=A-B (g)	4380	4571	4633	4763	4690
VOLUME OF MOULD	(CC)	2124	2124	2124	2124	2124
Bulk density	C / V = W	2.06	2.15	2.18	2.24	2.21
Moisture determination container No.		B	19	T2	A4	R4
Mass of container + wet soil	a (g)	584.8	406.4	496.1	394.2	473.5
mass of container + dry soil	b (g)	566.4	377.6	458.6	356.4	421.3
Mass of container	d (g)	52.1	39.7	53.1	39	40.5
Mass of dry soil	b - d = e (g)	514.3	337.9	405.5	317.4	380.8
Mass of moisture	a - b = f (g)	18.4	28.8	37.5	37.8	52.2
Moisture content	f/e*100 = m (%)	3.6	8.5	9.2	11.9	13.7
Dry density	W / (100+m) *100 (g/cc)	1.99	1.98	2.00	2.00	1.94
MAXIMUM DRY DENSITY (g / cc)		**2.005**				
OPTIMUM MOISTURE CONTENT (%)		**11.20**				

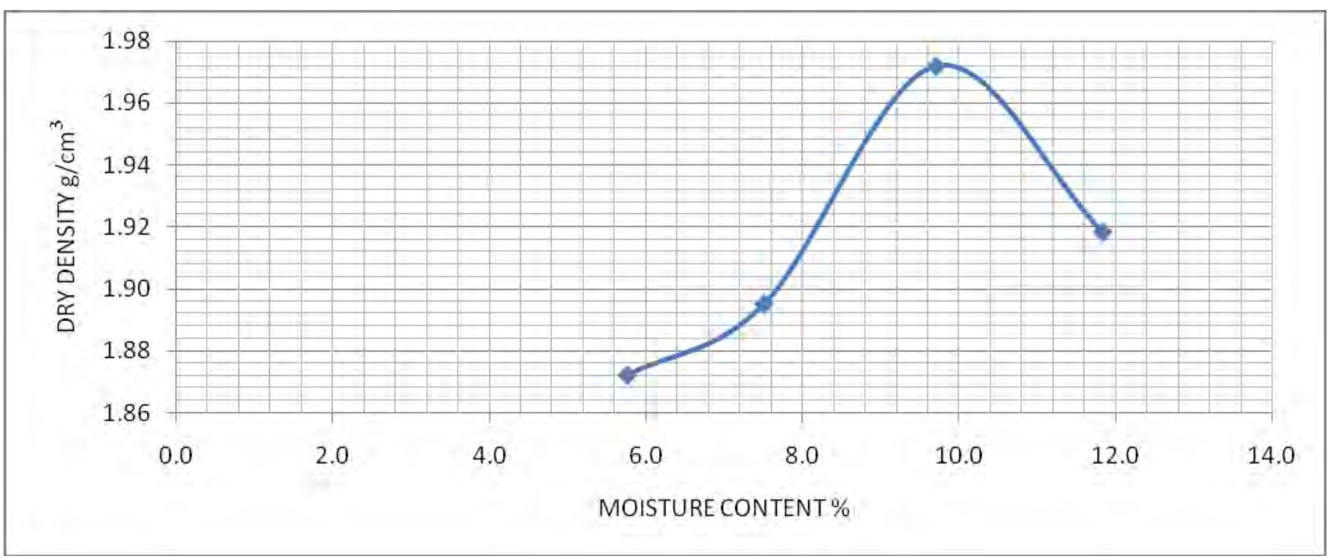
APPINDEX 4- TABLE H



APPINDEX 4- FIGURE H

COMPACTION CURVE						
sample date **Test date:-**						
Type of Material-- SN-8						
Compaction Method: D						
Moisture content %	2%	4%	6%	8%	10%	
Mass of wet soil + mould A (g)	11521	11643	11910	11872		
Mass of mould B (g)	7315	7315	7315	7315		
Mass of wet soil C=A-B (g)	4206	4328	4595	4557		
VOLUME OF MOULD (CC)	2124	2124	2124	2124		
Bulk density C / V = W	1.98	2.04	2.16	2.15		
Moisture determination container No.	H	T5	J	b1		
Mass of container + wet soil a (g)	408.4	421.4	389.1	411.2		
mass of container + dry soil b (g)	388.8	394.9	358.4	371.9		
Mass of container d (g)	48.8	42.2	42.2	39.8		
Mass of dry soil b - d = e (g)	340	352.7	316.2	332.1		
Mass of moisture a - b = f (g)	19.6	26.5	30.7	39.3		
Moisture content f/e*100 = m (%)	5.8	7.5	9.7	11.8		
Dry density W / (100+m) *100 (g/cc)	1.87	1.90	1.97	1.92		
MAXIMUM DRY DENSITY (g / cc)	**1.970**					
OPTIMUM MOISTURE CONTENT (%)	**9.70**					

APPINDEX 4- TABLE I



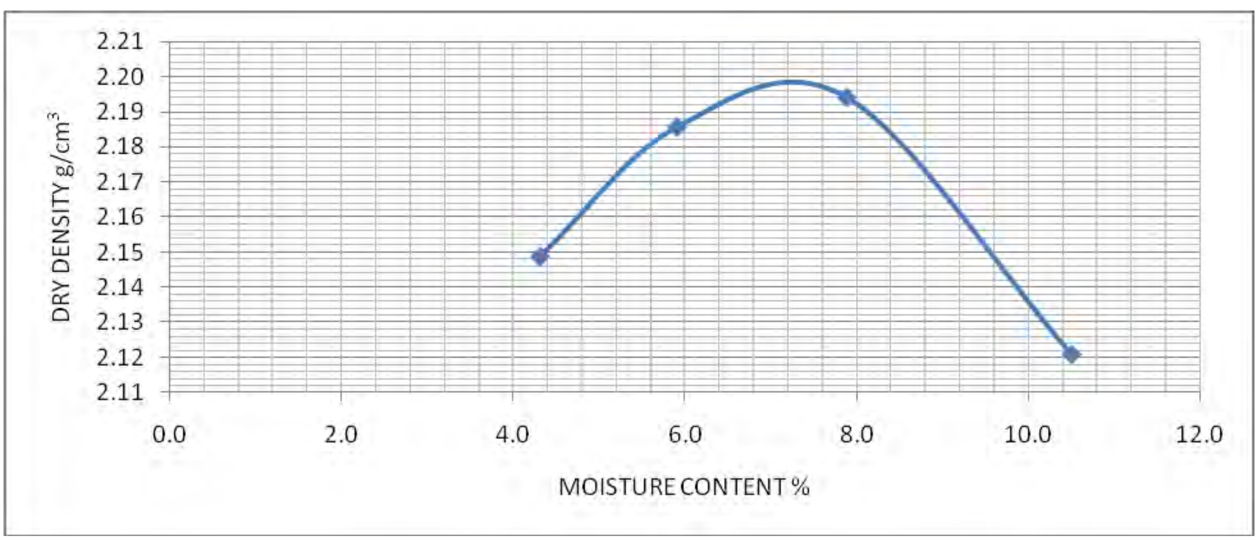
APPINDEX 4- FIGURE I

COMPACTION CURVE

sample date	
Test date:-	
Type of Material-- SN-9	
Compaction Method: D	

Moisture content %	2%	4%	6%	8%	10%	
Mass of wet soil + mould A (g)	12076	12232	12343	12293		
Mass of mould B (g)	7315	7315	7315	7315		
Mass of wet soil C=A-B (g)	4761	4917	5028	4978		
VOLUME OF MOULD (CC)	2124	2124	2124	2124		
Bulk density C / V = W	2.24	2.31	2.37	2.34		
Moisture determination container No.	R5	I5	H6	21		
Mass of container + wet soil a (g)	422.7	403.6	502	469.2		
mass of container + dry soil b (g)	406.9	383.4	468.3	428.4		
Mass of container d (g)	40.8	41.7	40.9	40.1		
Mass of dry soil b - d = e (g)	366.1	341.7	427.4	388.3		
Mass of moisture a - b = f (g)	15.8	20.2	33.7	40.8		
Moisture content f/e*100 = m (%)	4.3	5.9	7.9	10.5	21.1	
Dry density W / (100+m) *100 (g/cc)	2.15	2.19	2.19	2.12	5B	
MAXIMUM DRY DENSITY (g / cc)	**2.198**					
OPTIMUM MOISTURE CONTENT (%)	**7.40**					

APPINDEX 4- TABLE J

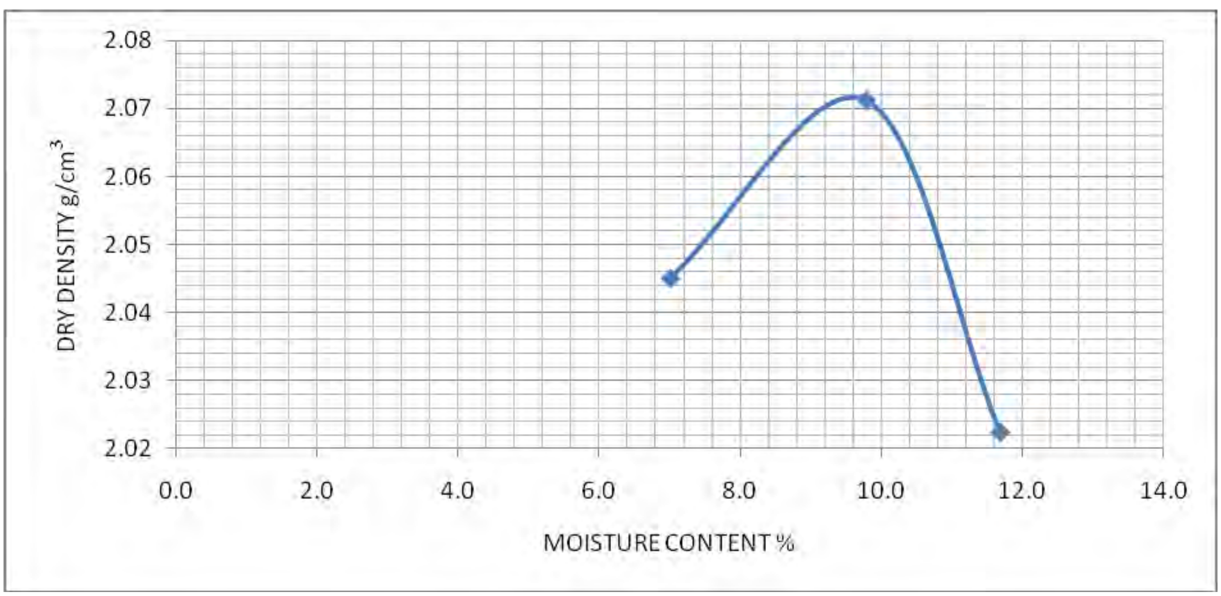


APPINDEX 4- FIGURE J

sample date **Test date:-**	
Type of Material-- SN-10	
Compaction Method: **D**	

COMPACTION CURVE				
Moisture content %	0%	2%	4%	
Mass of wet soil + mould A (g)	11963	12145	12113	
Mass of mould B (g)	7315	7315	7315	
Mass of wet soil C=A-B (g)	4648	4830	4798	
VOLUME OF MOULD (CC)	2124	2124	2124	
Bulk density C / V = W	2.19	2.27	2.26	
Moisture determination container No.	H	T5	I	
Mass of container + wet soil a (g)	488.6	369.6	472.2	
mass of container + dry soil b (g)	459.8	340.4	426.9	
Mass of container d (g)	48.7	42.1	39.5	
Mass of dry soil b - d = e (g)	411.1	298.3	387.4	
Mass of moisture a - b = f (g)	28.8	29.2	45.3	
Moisture content f/e*100 = m (%)	7.0	9.8	11.7	
Dry density W / (100+m) *100 (g/cc)	2.05	2.07	2.02	
MAXIMUM DRY DENSITY (g / cc)	**2.070**			
OPTIMUM MOISTURE CONTENT (%)	**9.80**			

APPINDEX 4- TABLE K



APPINDEX 4- FIGURE K

APPINDEX 5

PERMEABILITY TEST

sample number SN-1						
length of soil specimen, L, (cm)			12			
diameter ofsoil specimen(permeameter) ,cm			10			
volume of soil speciemen (cm3)			942			
dry density of soil (gm/cm3)			2.01			
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp,T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	600	14	27	0.000209816	0.000178501
2	17	1200	19.5	27	0.000146122	0.000124313
3	17	1800	24.2	27	0.000120894	0.00010285
						0.000135221

APPINDEX 5-TABLE A

sample number				SN-3		
length of soil specimen, L, (cm)				12		
diameter of soil specimen (permeameter) ,cm				10		
volume of soil speciemen (cm3)				942		
dry density of soil (gm/cm3)				2.224		
trial number	constant head (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp, T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	600	6.2	22	9.29187E-05	8.85732E-05
2	17	1200	10.2	19	7.64331E-05	7.83344E-05
3	17	1800	18.5	19	9.24191E-05	9.47181E-05
						8.72086E-05

APPINDEX 5-TABLE B

sample number				SN-5		
length of soil specimen, L, (cm)				12		
diameter ofsoil specimen(permeameter) ,cm				10		
volume of soil speciemen (cm3)				942		
dry density of soil (gm/cm3)				2.224		
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp, T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	600	6.2	23	9.29187E-05	8.65392E-05
2	17	1200	10.2	19.2	7.64331E-05	7.83344E-05
3	17	1800	18.5	19.2	9.24191E-05	9.47181E-05
						8.65306E-05

APPINDEX 5-TABLEC

sample number			SN-6			
length of soil specimen, L, (cm)			12			
diameter ofsoil specimen(permeameter) ,cm			10			
volume of soil speciemen (cm3)			942			
dry density of soil (gm/cm3)			2.01			
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp,T(0)	KT(cm/s)	K 20 (cm/s)
1	17	600	106	18	0.00158861	0.001669226
2	17	1200	315	18	0.002360435	0.002480218
3	17	1800	440	18	0.002198077	0.002309621
						0.002153022

APPINDEX 5-TABLE D

sample number			SN-7			
length of soil specimen, L, (cm)			12			
diameter ofsoil specimen(permeameter) ,cm			10			
volume of soil speciemen (cm3)			942			
dry density of soil (gm/cm3)			2.01			
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp,T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	600	14	27	0.000209816	0.000178501
2	17	1200	19.5	27	0.000146122	0.000124313
3	17	1800	24.2	27	0.000120894	0.00010285
						0.000135221

APPINDEX 5-TABLE E

sample number			SN-8			
length of soil specimen, L, (cm)			12			
diameter ofsoil specimen(permeameter) ,cm			10			
volume of soil speciemen (cm3)			942			
dry density of soil (gm/cm3)			2.01			
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp,T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	900	10	17	9.99126E-05	0.000107667
2	17	1200	13.5	17	0.000101161	0.000109013
3	17	1800	16	17	7.99301E-05	8.61336E-05
						0.000100938

APPINDEX 5-TABLE F

sample number				SN-9		
length of soil specimen, L, (cm)				12		
diameter ofsoil specimen(permeameter) ,cm				10		
volume of soil speciemen (cm3)				942		
dry density of soil (gm/cm3)				2.2		
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp,T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	600	4.5	18	6.7441E-05	7.08634E-05
2	17	1200	9.2	18	6.89397E-05	7.24381E-05
3	17	1800	14	18	6.99388E-05	7.34879E-05
						7.22631E-05

APPINDEX 5-TABLE G

sample number				SN-10		
length of soil specimen, L, (cm)				12		
diameter ofsoil specimen(permeameter) ,cm				10		
volume of soil speciemen (cm3)				942		
dry density of soil (gm/cm3)				2.07		
trial number	constant head,h (cm)	elapsed time ,t(sec)	out flow volume ,Q(cm3)	water temp,T(0c)	KT(cm/s)	K 20 (cm/s)
1	17	600	1	17	1.49869E-05	1.615E-05
2	17	1200	3.5	17	2.62271E-05	2.82626E-05
3	17	1800	8.2	17	4.09642E-05	4.41435E-05
						2.95187E-05

APPINDEX 5-TABLE H

APPINDEX 6

SCHMIDT HAMMER TEST

Rock unconfined compressive strength by Schmitt hammer test

Sample No.	Angle of measure	Hammer reading	U.C.S in Mpa
SN-1	0º	18	8
SN-2	0º	38	37
SN-3	0º	12	5
SN-4	45º	32	28.5
SN-5	0º	24	20
SN-6	-90º	66	69
SN-7	-90º	0	19
SN-8	45º	5	8
SN-9	0º	11	1
SN-10	0	20.5	0

APPINDEX 6 – TABLE A

APPINDEX 7

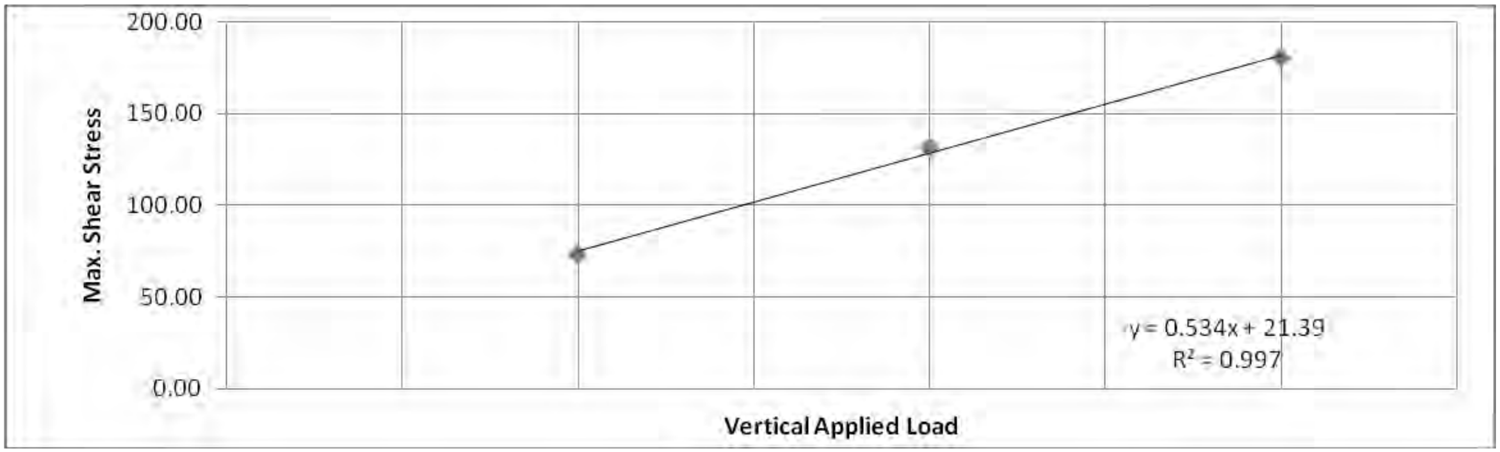
DIRECT SHEAR TEST

Sample No.	SN-1			Location:	
Thickness of sample:	25 mm	Ring Calib. Factor:	0.70 N/div	Wet unit weight, kN/M^3:	21.99
Length of sample :	60 mm	Rate of strain :	1.6 mm/min	Dry Unit Weight, kN/M^3:	19.72
Width of sample:	60 mm	Moisture content, %	11.5	Sample Condition:	Disturbed
Area of Sample	3600	**mm2**			

				Applied Vertical Stress **100 kPa**			Applied Vertical Stress **200 kPa**			Applied Vertical Stress **300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	124.00	0.09	24.13	70.00	0.05	13.62	100.00	0.07	19.46
100.0	1.0	0.0036	0.003594	198.00	0.14	38.56	185.00	0.13	36.03	221.00	0.15	43.04
150.0	1.5	0.0036	0.003591	243.00	0.17	47.37	250.00	0.18	48.73	315.00	0.22	61.40
200.0	2.0	0.0036	0.003588	364.00	0.25	71.01	284.00	0.20	55.41	405.00	0.28	79.01
250.0	2.5	0.0036	0.003585	366.00	0.26	71.46	300.00	0.21	58.58	494.00	0.35	96.46
300.0	3.0	0.0036	0.003582	364.00	0.25	71.13	380.00	0.27	74.26	559.00	0.39	109.24
350.0	3.5	0.0036	0.003579	362.00	0.25	70.80	450.00	0.32	88.01	616.00	0.43	120.48
400.0	4.0	0.0036	0.003576	356.00	0.25	69.69	517.00	0.36	101.20	669.00	0.47	130.96
450.0	4.5	0.0036	0.003573	353.00	0.25	69.16	568.00	0.40	111.28	720.00	0.50	141.06
500.0	5.0	0.0036	0.00357	355.00	0.25	69.61	607.00	0.42	119.02	766.00	0.54	150.20
550.0	5.5	0.0036	0.003567	360.00	0.25	70.65	637.00	0.45	125.01	811.00	0.57	159.15
600.0	6.0	0.0036	0.003564	365.00	0.26	71.69	654.00	0.46	128.45	855.00	0.60	167.93
650.0	6.5	0.0036	0.003561	370.00	0.26	72.73	665.00	0.47	130.72	883.00	0.62	173.57
700.0	7.0	0.0036	0.003558	372.00	0.26	73.19	668.00	0.47	131.42	900.00	0.63	177.07
750.0	7.5	0.0036	0.003555	372.00	0.26	73.25	665.00	0.47	130.94	914.00	0.64	179.97
800.0	8.0	0.0036	0.003552	371.00	0.26	73.11	656.00	0.46	129.28	914.00	0.64	180.12
850.0	8.5	0.0036	0.003549	369.00	0.26	72.78	640.00	0.45	126.23	909.00	0.64	179.29
900.0	9.0	0.0036	0.003546	368.00	0.26	72.65		0.00	0.00	898.00	0.63	177.27
950.0	9.5	0.0036	0.003543	366.00	0.26	72.31		0.00	0.00	885.00	0.62	174.85

	Maximum shear stress, kPa		**73.25**	*Maximum shear stress, kPa*		**131.42**	*Maximum shear stress, kPa*	**180.12**

APPINDEX 7- TABLE A



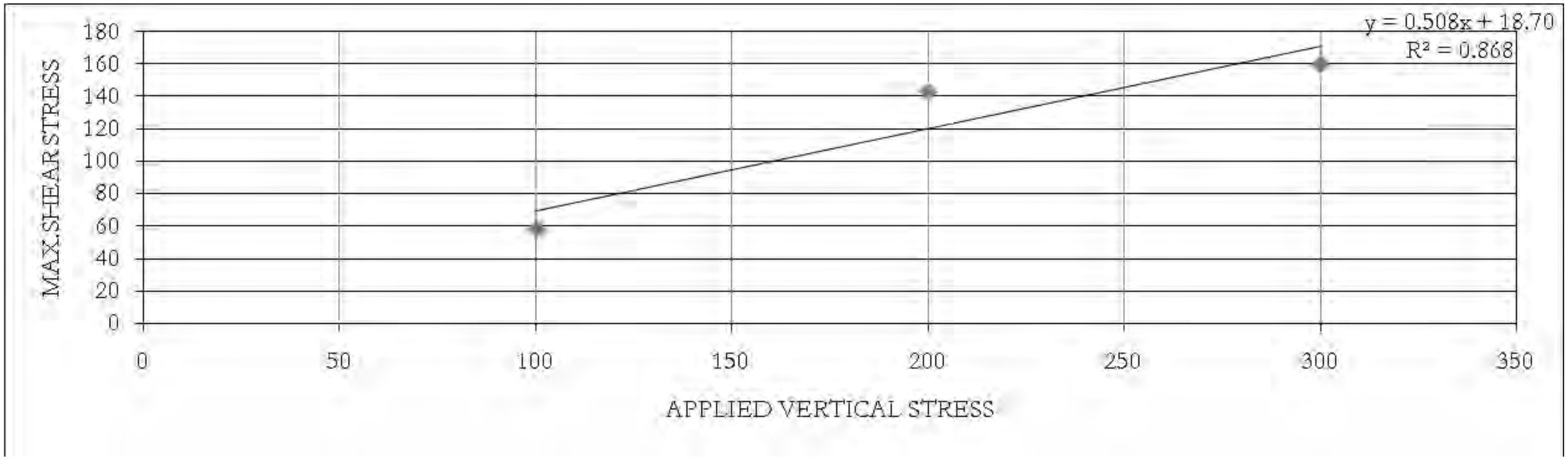
APPINDEX 7- FIGURE A

Angle of internal friction,

Cohesion, C (KN/m^2) =21.4

Sample No.				SN-3					Location:			
Thickness of sample:				25 mm	Ring Calib. Factor:		0.70 N/div		Wet unit weight, kN/M^3:			23.68
Length of sample :				60 mm	Rate of strain :		1.6 mm/min		Dry Unit Weight, kN/M^3:			21.88
Width of sample:				60 mm	Moisture content, %		8.3		Sample Condition:			Disturbed
Area of Sample				3600	**mm2**							
				Applied Vertical Stress **100 kPa**			Applied Vertical Stress **200 kPa**			Applied Vertical Stress **300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	96.00	0.07	18.68	107.00	0.07	20.82	141.00	0.10	27.44
100.0	1.0	0.0036	0.003594	116.0	0.08	22.59	210.00	0.15	40.90	289.00	0.20	56.29
150.0	1.5	0.0036	0.003591	209.0	0.15	40.74	338.00	0.24	65.89	374.00	0.26	72.90
200.0	2.0	0.0036	0.003588	262.0	0.18	51.11	416.00	0.29	81.16	437.00	0.31	85.26
250.0	2.5	0.0036	0.003585	284.0	0.20	55.45	486.00	0.34	94.90	515.00	0.36	100.56
300.0	3.0	0.0036	0.003582	296.0	0.21	57.84	548.00	0.38	107.09	577.00	0.40	112.76
350.0	3.5	0.0036	0.003579	297.0	0.21	58.09	597.00	0.42	116.76	625.00	0.44	122.24
400.0	4.0	0.0036	0.003576	293.0	0.21	57.35	651.00	0.46	127.43	665.00	0.47	130.17
450.0	4.5	0.0036	0.003573	283.0	0.20	55.44	693.00	0.49	135.77	705.00	0.49	138.12
500.0	5.0	0.0036	0.00357	270.0	0.19	52.94	718.00	0.50	140.78	737.00	0.52	144.51
550.0	5.5	0.0036	0.003567				730.00	0.51	143.26	763.00	0.53	149.73
600.0	6.0	0.0036	0.003564				728.00	0.51	142.99	781.00	0.55	153.40
650.0	6.5	0.0036	0.003561				712.00	0.50	139.96	800.00	0.56	157.26
700.0	7.0	0.0036	0.003558				690.00	0.48	135.75	812.00	0.57	159.75
				Maximum shear stress, kPa		**58.09**	*Maximum shear stress, kPa*		**143.26**	*Maximum shear stress, kPa*		**159.75**

APPINDEX 7- TABLE B



APPINDEX 7- FIGURE B

Angle of internal friction

Cohesion, C (KN/m^2) = 16

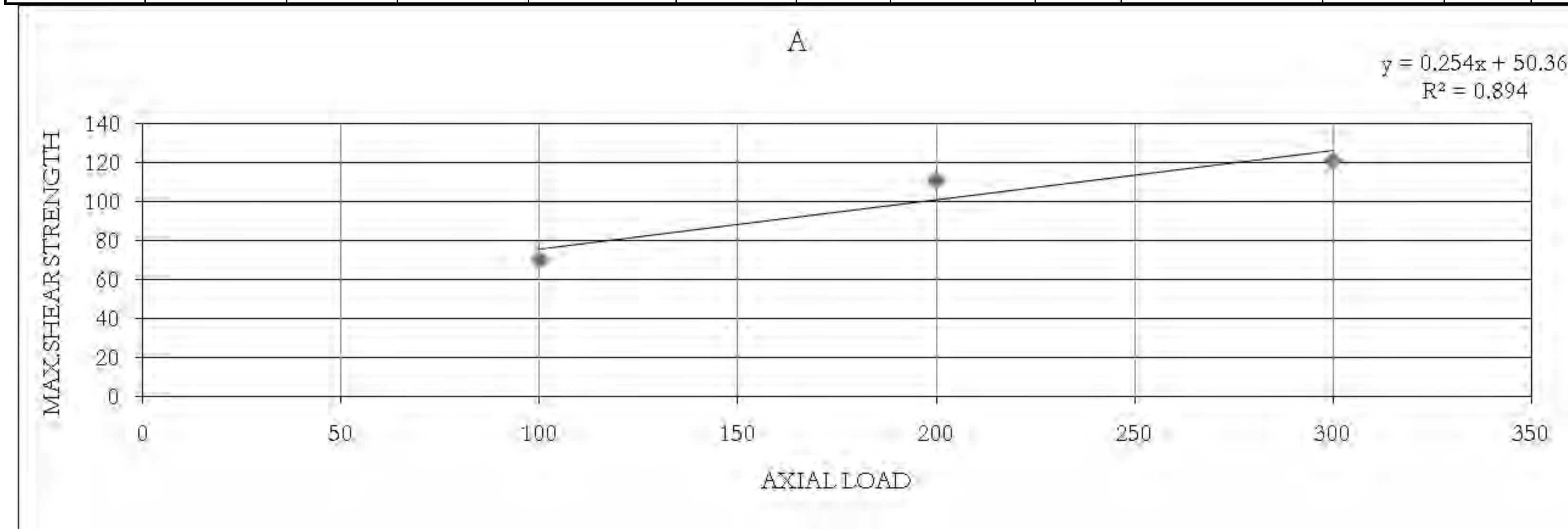
Sample No.	SN-5	Sample Depth, m:	1.50	Location:	
Thickness of sample:	25 mm	Ring Calib. Factor:	0.70 N/div	Wet unit weight, kN/M^3:	23.58
Length of sample :	60 mm	Rate of strain :	1.6 mm/min	Dry Unit Weight, kN/M^3:	21.82
Width of sample:	60 mm	Moisture content, %	8.1	Sample Condition:	Disturbed
Area of Sample	3600	**mm2**			

				Applied Vertical Stress **100 kPa**			Applied Vertical Stress **200 kPa**			Applied Vertical Stress **300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	87.00	0.06	16.93	139.00	0.10	27.05	128.00	0.09	24.91
100.0	1.0	0.0036	0.003594	138.00	0.10	26.88	243.00	0.17	47.33	244.00	0.17	47.52
150.0	1.5	0.0036	0.003591	196.00	0.14	38.21	347.00	0.24	67.64	311.00	0.22	60.62
200.0	2.0	0.0036	0.003588	295.00	0.21	57.55	442.00	0.31	86.23	383.00	0.27	74.72
250.0	2.5	0.0036	0.003585	356.00	0.25	69.51	496.00	0.35	96.85	457.00	0.32	89.23
300.0	3.0	0.0036	0.003582	362.00	0.25	70.74	529.00	0.37	103.38	508.00	0.36	99.27
350.0	3.5	0.0036	0.003579	354.00	0.25	69.24	549.00	0.38	107.38	539.00	0.38	105.42
400.0	4.0	0.0036	0.003576	341.00	0.24	66.75	564.00	0.39	110.40	556.00	0.39	108.84
450.0	4.5	0.0036	0.003573	327.00	0.23	64.06	568.00	0.40	111.28	571.00	0.40	111.87
500.0	5.0	0.0036	0.00357				561.00	0.39	110.00	591.00	0.41	115.88
550.0	5.5	0.0036	0.003567				549.00	0.38	107.74	606.00	0.42	118.92
600.0	6.0	0.0036	0.003564				540.00	0.38	106.06	619.00	0.43	121.58
				Maximum		**70.74**	*Maximum*		**111.28**	*Maximum*		**121.58**

Sample No.	SN-6	Sample Depth, m:	1.50	Location:	

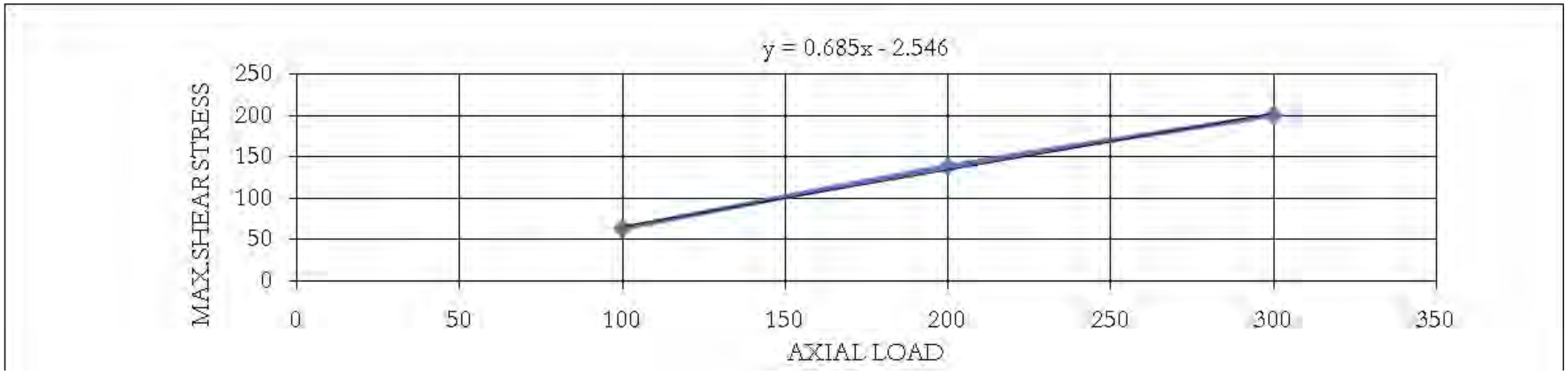
Thickness of sample:				25 mm	Ring Calib. Factor:		0.70 N/div		Wet unit weight, kN/M^3:			24.84
Length of sample :				60 mm	Rate of strain :		1.6 mm/min		Dry Unit Weight, kN/M^3:			23.54
Width of sample:				60 mm	Moisture content, %		5.5		Sample Condition:			Disturbed
Area of Sample				3600	**mm2**							
				Applied Vertical Stress **100 kPa**			Applied Vertical Stress **200 kPa**			Applied Vertical Stress **300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	86.00	0.06	16.74	128.00	0.09	24.91	152.00	0.11	29.58
100.0	1.0	0.0036	0.003594	104.00	0.07	20.26	225.00	0.16	43.82	287.00	0.20	55.90
150.0	1.5	0.0036	0.003591	183.00	0.13	35.67	261.00	0.18	50.88	405.00	0.28	78.95
200.0	2.0	0.0036	0.003588	237.00	0.17	46.24	307.00	0.21	59.89	490.00	0.34	95.60
250.0	2.5	0.0036	0.003585	310.00	0.22	60.53	340.00	0.24	66.39	567.00	0.40	110.71
300.0	3.0	0.0036	0.003582	326.00	0.23	63.71	385.00	0.27	75.24	629.00	0.44	122.92
350.0	3.5	0.0036	0.003579	318.00	0.22	62.20	437.00	0.31	85.47	696.00	0.49	136.13
400.0	4.0	0.0036	0.003576	299.00	0.21	58.53	495.00	0.35	96.90	760.00	0.53	148.77
450.0	4.5	0.0036	0.003573	291.00	0.20	57.01	544.00	0.38	106.58	816.00	0.57	159.87
500.0	5.0	0.0036	0.00357				585.00	0.41	114.71	865.00	0.61	169.61
550.0	5.5	0.0036	0.003567				622.00	0.44	122.06	912.00	0.64	178.97
600.0	6.0	0.0036	0.003564				655.00	0.46	128.65	950.00	0.67	186.59
650.0	6.5	0.0036	0.003561				671.00	0.47	131.90	982.00	0.69	193.04
700.0	7.0	0.0036	0.003558				685.00	0.48	134.77	997.00	0.70	196.15
750.0	7.5	0.0036	0.003555				697.00	0.49	137.24	1007.00	0.70	198.28
800.0	8.0	0.0036	0.003552				702.00	0.49	138.34	1008.00	0.71	198.65
850.0	8.5	0.0036	0.003549				705.00	0.49	139.05	1015.00	0.71	200.20
900.0	9.0	0.0036	0.003546				703.00	0.49	138.78	1017.00	0.71	200.76
950.0	9.5	0.0036	0.003543				701.00	0.49	138.50	1012.00	0.71	199.94
1000.0	10.0	0.0036	0.00354				696.00	0.49	137.63	1004.00	0.70	198.53

1050.0	10.5	0.0036	0.003537					0.00	0.00	992.00	0.69	196.32
				Maximum shear stress, kPa		**63.71**	*Maximum shear stress, kPa*		**139.05**	*Maximum shear stress, kPa*		**200.76**



APPINDEX 7- FIGURE C.3

Angle of internal friction, 16.1 **Cohesion , C (KN/m^2) = 45.8**



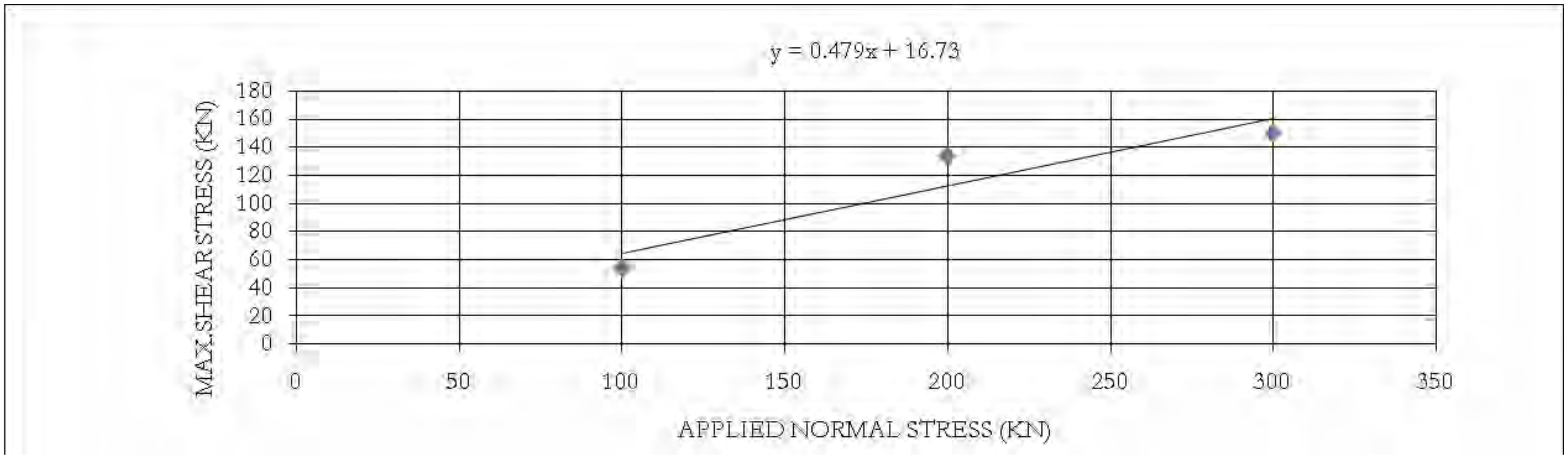
APPINDEX 7- FIGURE D

Angle of internal friction,	**34.4**	**Cohesion , C (kN/m^2) =**	**2.5**

Sample No.		SN-7			Sample Depth, m:		1.50		Location:			
Thickness of sample:		25 mm			Ring Calib. Factor:		0.70 N/div		Wet unit weight, kN/M^3:			21.87
Length of sample :		60 mm			Rate of strain :		1.6 mm/min		Dry Unit Weight, kN/M^3:			19.67
Width of sample:		60 mm			Moisture content, %		11.2		Sample Condition:			Disturbed
Area of Sample		3600			**mm2**							
				Applied Vertical Stress **100 kPa**			Applied Vertical Stress **200 kPa**			Applied Vertical Stress **300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	80.00	0.06	15.57	132.00	0.09	25.69	157.00	0.11	30.55
100.0	1.0	0.0036	0.003594	144.00	0.10	28.05	240.00	0.17	46.74	312.00	0.22	60.77
150.0	1.5	0.0036	0.003591	214.00	0.15	41.72	356.00	0.25	69.40	449.00	0.31	87.52
200.0	2.0	0.0036	0.003588	257.00	0.18	50.14	465.00	0.33	90.72	555.00	0.39	108.28
250.0	2.5	0.0036	0.003585	278.00	0.19	54.28	570.00	0.40	111.30	670.00	0.47	130.82
300.0	3.0	0.0036	0.003582	272.00	0.19	53.15	625.00	0.44	122.14	745.00	0.52	145.59
350.0	3.5	0.0036	0.003579	269.00	0.19	52.61	664.00	0.46	129.87	768.00	0.54	150.21
400.0	4.0	0.0036	0.003576	263.00	0.18	51.48	682.00	0.48	133.50	762.00	0.53	149.16
450.0	4.5	0.0036	0.003573		0.00	0.00	570.00	0.40	111.67	757.00	0.53	148.31
500.0	5.0	0.0036	0.00357		0.00	0.00	529.00	0.37	103.73	744.00	0.52	145.88
550.0	5.5	0.0036	0.003567		0.00	0.00	524.00	0.37	102.83		0.00	0.00

600.0	6.0	0.0036	0.003564		0.00	0.00	504.00	0.35	98.99		0.00	0.00
				Max. shear stress, kPa		**54.28**	*Max. shear stress, kPa*		**133.50**	*Max. shear stress, kPa*		**150.21**

APPINDEX 7- TABLE F

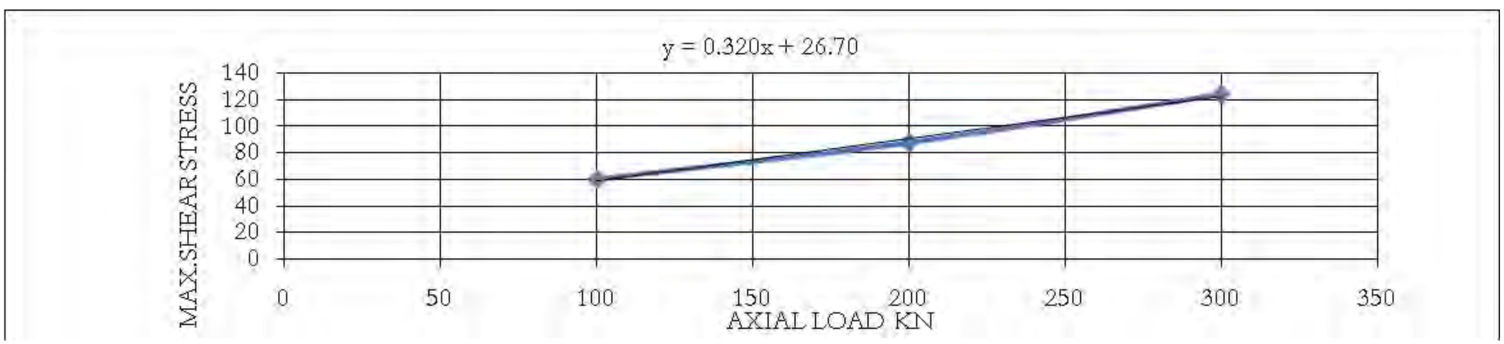


Angle of internal friction,	**25.6**	**Cohesion , C (kN/m^2) -**	**16.7**

Sample No.	SN-8	Sample Depth, m:		1.50		Location:			
Thickness of sample:	25 mm	Ring Calib. Factor:		0.70 N/div		Wet unit weight, kN/M^3:			21.20
Length of sample :	60 mm	Rate of strain :		1.6 mm/min		Dry Unit Weight, kN/M^3:			19.33
Width of sample:	60 mm	Moisture content, %		9.7		Sample Condition:			Disturbed
Area of Sample	3600	**mm2**							

				Applied Vertical Stress 100 kPa			Applied Vertical Stress 200 kPa			Applied Vertical Stress 300 kPa		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	109.00	0.08	21.21	115.00	0.08	22.38	124.00	0.09	24.13
100.0	1.0	0.0036	0.003594	197.00	0.14	38.37	232.00	0.16	45.19	289.00	0.20	56.29
150.0	1.5	0.0036	0.003591	256.00	0.18	49.90	287.00	0.20	55.95	440.00	0.31	85.77
200.0	2.0	0.0036	0.003588	299.00	0.21	58.33	376.00	0.26	73.36	510.00	0.36	99.50
250.0	2.5	0.0036	0.003585	286.00	0.20	55.84	433.00	0.30	84.55	565.00	0.40	110.32
300.0	3.0	0.0036	0.003582	291.00	0.20	56.87	449.00	0.31	87.74	630.00	0.44	123.12
350.0	3.5	0.0036	0.003579	304.00	0.21	59.46	445.00	0.31	87.04	636.00	0.45	124.39
400.0	4.0	0.0036	0.003576	308.00	0.22	60.29	429.00	0.30	83.98	607.00	0.42	118.82
450.0	4.5	0.0036	0.003573	244.00	0.17	47.80	420.00	0.29	82.28	590.00	0.41	115.59
500.0	5.0	0.0036	0.00357	232.00	0.16	45.49	413.00	0.29	80.98	574.00	0.40	112.55
550.0	5.5	0.0036	0.003567	228.00	0.16	44.74		0.00	0.00		0.00	0.00
				Maximum shear stress, kPa		**60.29**	*Maximum shear stress, kPa*		**87.74**	*Maximum shear stress, kPa*		**124.39**

APPINDEX 7- TABLE G



APPINDEX 7- FIGURE G

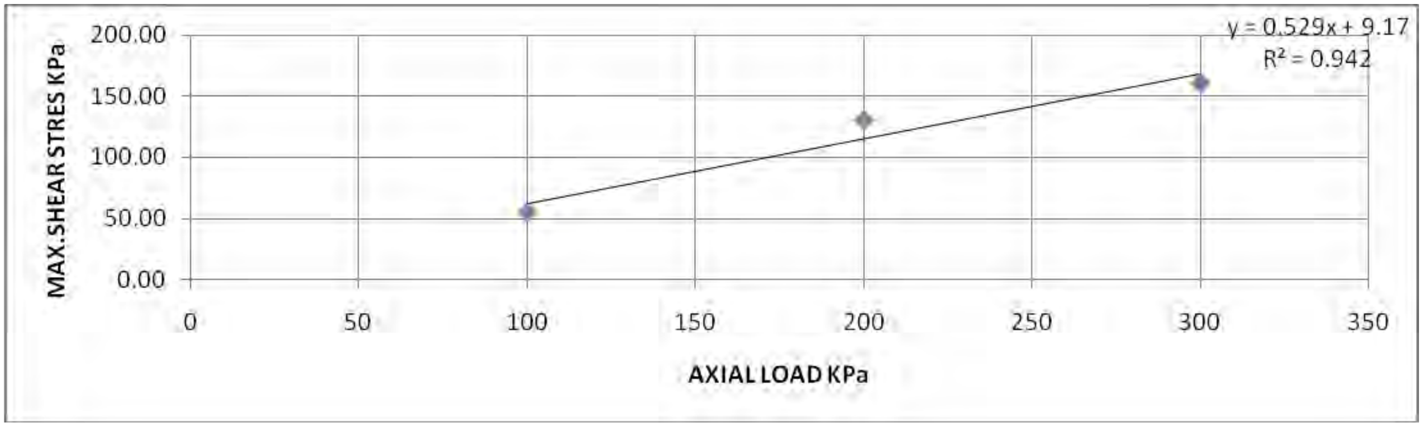
Angle of internal friction,	17.8	Cohesion , C (KN/m^2) =	26.7

Sample No.				SN-9	Sample Depth, m:		1.50		Location:			
Thickness of sample:				25 mm	Ring Calib. Factor:		0.70 N/div		Wet unit weight, kN/M^3:			23.07
Length of sample :				60 mm	Rate of strain :		1.6 mm/min		Dry Unit Weight, kN/M^3:			21.48
Width of sample:				60 mm	Moisture content, %		7.4		Sample Condition:			Disturbed
Area of Sample				3600	**mm2**							

				Applied Vertical Stress **100 kPa**			Applied Vertical Stress **200 kPa**			Applied Vertical Stress **300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	85.00	0.06	16.54	77.00	0.05	14.98	132.00	0.09	25.69
100.0	1.0	0.0036	0.003594	123.00	0.09	23.96	194.00	0.14	37.79	275.00	0.19	53.56
150.0	1.5	0.0036	0.003591	156.00	0.11	30.41	298.00	0.21	58.09	395.00	0.28	77.00
200.0	2.0	0.0036	0.003588	217.00	0.15	42.34	323.00	0.23	63.02	489.00	0.34	95.40
250.0	2.5	0.0036	0.003585	243.00	0.17	47.45	325.00	0.23	63.46	568.00	0.40	110.91
300.0	3.0	0.0036	0.003582	257.00	0.18	50.22	352.00	0.25	68.79	632.00	0.44	123.51
350.0	3.5	0.0036	0.003579	265.00	0.19	51.83	475.00	0.33	92.90	691.00	0.48	135.15
400.0	4.0	0.0036	0.003576	272.00	0.19	53.24	571.00	0.40	111.77	737.00	0.52	144.27
450.0	4.5	0.0036	0.003573	276.00	0.19	54.07	621.00	0.43	121.66	772.00	0.54	151.25
500.0	5.0	0.0036	0.00357	278.00	0.19	54.51	646.00	0.45	126.67	797.00	0.56	156.27
550.0	5.5	0.0036	0.003567	278.00	0.19	54.56	660.00	0.46	129.52	814.50	0.57	159.84

600.0	6.0	0.0036	0.003564	277.50	0.19	54.50	663.00	0.46	130.22	817.00	0.57	160.47
650.0	6.5	0.0036	0.003561	276.00	0.19	54.25	654.00	0.46	128.56	813.00	0.57	159.81
700.0	7.0	0.0036	0.003558	275.50	0.19	54.20	638.00	0.45	125.52	806.00	0.56	158.57
750.0	7.5	0.0036	0.003555	272.00	0.19	53.56	619.00	0.43	121.88	802.00	0.56	157.92
				Maximum		**54.56**	*Maximum shear*		**130.22**	*Maximum shear stress,*		**160.47**

APPINDEX 7- TABLE H



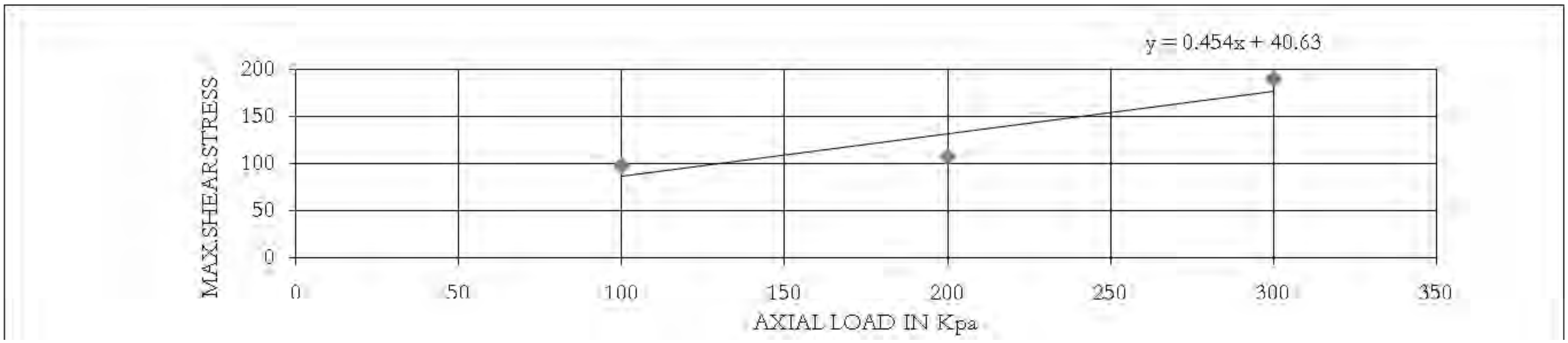
APPINDEX 7- TABLE I

Angle of internal friction,	**27.9**	**Cohesion , C (kN/m^2) =**	**9.2**

Sample No.				SN-10	Sample Depth, m:		1.50		Location:			
Thickness of sample:				25 mm	Ring Calib. Factor:		0.70 N/div		Wet unit weight, kN/M3:			22.30
Length of sample :				60 mm	Rate of strain :		1.6 mm/min		Dry Unit Weight, kN/M3:			20.31
Width of sample:				60 mm	Moisture content, %		9.8		Sample Condition:			Disturbed
Area of Sample				3600	**mm2**							
				Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress		
				100 kPa			**200 kPa**			**300 kPa**		
Horizontal Displacement (mm)	Sample Displacement	Area of sample (mm2)	Corrected Area (m2)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)	Proving Ring Reading	Shear load (KN)	Shear Stress (Kpa)
0.0	0.0	0.0036	0.0036	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
50.0	0.5	0.0036	0.003597	116.00	0.08	22.57	125.00	0.09	24.33	171.00	0.12	33.28
100.0	1.0	0.0036	0.003594	124.00	0.09	24.15	226.00	0.16	44.02	326.00	0.23	63.49
150.0	1.5	0.0036	0.003591	147.00	0.10	28.65	297.00	0.21	57.89	425.00	0.30	82.85
200.0	2.0	0.0036	0.003588	248.00	0.17	48.38	354.00	0.25	69.06	544.00	0.38	106.13
250.0	2.5	0.0036	0.003585	334.00	0.23	65.22	393.00	0.28	76.74	695.00	0.49	135.70
300.0	3.0	0.0036	0.003582	370.00	0.26	72.31	420.00	0.29	82.08	824.00	0.58	161.03
350.0	3.5	0.0036	0.003579	410.00	0.29	80.19	442.00	0.31	86.45	933.00	0.65	182.48
400.0	4.0	0.0036	0.003576	440.00	0.31	86.13	464.00	0.32	90.83	966.00	0.68	189.09
450.0	4.5	0.0036	0.003573	469.00	0.33	91.88	484.00	0.34	94.82	943.00	0.66	184.75
500.0	5.0	0.0036	0.00357	495.00	0.35	97.06	501.00	0.35	98.24	919.00	0.64	180.20
550.0	5.5	0.0036	0.003567	500.00	0.35	98.12	520.00	0.36	102.05	906.00	0.63	177.80
600.0	6.0	0.0036	0.003564	391.00	0.27	76.80	528.00	0.37	103.70			
650.0	6.5	0.0036	0.003561	350.00	0.25	68.80	536.00	0.38	105.36			
700.0	7.0	0.0036	0.003558		0.00	0.00	542.00	0.38	106.63			
750.0	7.5	0.0036	0.003555		0.00	0.00	545.00	0.38	107.31			

800.0	8.0	0.0036	0.003552		0.00	0.00	546.00	0.38	107.60			
850.0	8.5	0.0036	0.003549		0.00	0.00	542.00	0.38	106.90			
900.0	9.0	0.0036	0.003546		0.00	0.00	539.00	0.38	106.40			
950.0	9.5	0.0036	0.003543		0.00	0.00	534.00	0.37	105.50			
				Maximum shear		**98.12**	*Maximum shear stress, kPa*		**107.60**	*Maximum shear stress, kPa*		**189.09**

APPINDEX 7- TABLE J



APPINDEX 7- FIGURE J

Angle of internal friction, =24.5	**Cohesion , C (KN/m^2) = 40.6**

APPENDIX -2 USCS SOIL GROUPS, THEIR DESIRABILITY RATING, AND CORRELATIONS WITH PROPERTIES OF COMPACTED SOILS

Group Symbol	Soil Type	Range of Max. Dry Weight (pcf)	Range of Optimum Moisture (%)	Typical Value of Compression (% of original height)		Typical Strength Characteristics				Typical Hyd. Cond. (ft/min)	Range of CBR Value	Range of Subgrade Modulus k (lbs/in³)
				1.4 tsf = 20 psi	3.5 tsf = 50 psi	Compacted Cohesion (psf)	Saturated Cohesion (psf)	φ (deg)	tan φ			
GW	Well graded clean gravels, gravel-sand mixtures	125-135	11-8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40-80	300-500
GP	Poorly-graded clean gravels, gravel-sand mix	115-125	14-11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30-60	250-400
GM	Silty gravels, poorly-graded gravel-sand mix	120-135	12-8	0.5	1.1	–	–	>34	>0.67	$>10^{-6}$	20-60	100-400
GC	Clayey gravels, poorly graded gravel-sand-clay	115-130	14-9	0.7	1.6	–	–	>31	>0.60	$>10^{-7}$	20-40	100-300
SW	Well-graded clean sands, gravelly sands	110-130	16-9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20-40	200-300
SP	Poorly-graded clean sands, sand-gravel mix	100-120	21-12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10-40	200-300
SM	Silty sands, poorly-graded sand-silt mix	110-125	16-11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10-40	100-300
SM-SC	Sand-silt-clay mix with slightly plastic fines	110-130	15-11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	5-30	100-300
SC	Clayey sands, poorly graded sand-clay mix	105-125	19-11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5-20	100-300
ML	Inorganic silts and clayey silts	95-120	24-12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	≤15	100-200
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	–	–
CL	Inorganic clays of low to medium plasticity	95-120	24-12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	≤ 15	50-200
OL	Organic silts and silt-clays, low plasticity	80-100	33-21	–	–	–	–	–	–	–	≤ 5	50-100
MH	Inorganic clayey silts, elastic silts	70-95	40-24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	≤ 10	50-100
CH	Inorganic clays of high plasticity	75-105	36-19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	≤ 15	50-150
OH	Organic clays and silty clays	65-100	45-21	-	-	-	-	-	-	-	≤ 5	25-100

Note: 1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "Modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (–) indicates insufficient data available for an estimate.

(DOD, 2005)

Group Symbol	Soil Type	Relative Desirability for Various Uses (1 is most desirable. 14 is least desirable)							
		Rolled Earth Fill Dams			Lining			Foundation	
		Homogenous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Roadway Surfacing	Seepage Important	Seepage Not Important
GW	Well graded clean gravels, gravel-sand mixtures	-	-	1	1	-	3	-	1
GP	Poorly-graded clean gravels, gravel-sand mix	-	-	2	2	-	-	-	3
GM	Silty gravels, poorly-graded gravel-sand mix	2	4	-	4	4	5	1	4
GC	Clayey gravels, poorly graded gravel-sand-clay	1	1	-	3	1	1	2	6
SW	Well graded clean sands, gravelly sands	-	-	3, if gravelly	6	-	4	-	2
SP	Poorly graded clean sands, sand-gravel mix	-	-	4, if gravelly	7, if gravelly	-	-	-	5
SM	Silty sands, poorly graded sand-silt mix	4	5	-	8, if gravelly	5, erosion critical	6	3	7
SC	Clayey sands, poorly graded sand-clay mix	3	2	-	5	2	2	4	8
ML	Inorganic silts and clayey silts	6	6	-	-	6, erosion critical	-	6	9
CL	Inorganic clays of low to medium plasticity	5	3	-	9	3	7	5	10
OL	Organic silts and silt-clays, low plasticity	8	8	-	-	7, erosion critical	-	7	11
MH	Inorganic clayey silts, elastic silts	9	9	-	-	-	-	8	12
CH	Inorganic clays of high plasticity	7	7	-	10	8, vol.-change critical	-	9	13
OH	Organic clays and silty clays	10	10	-	-	-	-	10	14

(DOD, 2005)