ENGINEEERING PROPERTIES OF CONSTRUCTION MATERIAL BEING USED IN HAZARA MOTORWAY E-35 PAKAGE 3 SARAI SALEH HARIPUR, PAKISTAN

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

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ABSTRACT

Geo technical exploration is the fundamental part of every construction project .Our study area lies at Sarai Saleh which is the segment of E-35 Expressway. The study objectives consist of evaluating engineering properties of earth materials which are exposed in the study area and can be a good resource for road construction for the Expressway package 3 in Haripur, Hazara District. During field investigation different test pits were excavated as per project's specification along the proposed road alignment. In-situ compaction was determined by using Sand-cone method (AASHTO T224) which ranges from to 90%-100 %. The gradation analysis (AASHTO T27) resulted percentage passing of 40.8%, 33.5%,28.3% and 22.7% at sieve number 4,10,40 and 200 respectively for sample 1, while the percentage passing for the sample 2 were 85%,35.8%,11.3% and 4.47% at sieve number 4,10,40 and 200 respectively. Atterberg limits (AASHTO T89) showed plasticity index of 2.7 and 4.6 for sample 1 and 2 respectively. The Californian Bearing Ratio (AASHTO T193) at 1" penetration ranges from 38.3% to 64.6% and 36.3%-57.0% at 0.2" respectively. According to AASHTO classification sample 1 was classified in A-1-b which is good to excellent for embankment and sub-grade while sample 2 is good for road base.

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CHAPTER 1

INTRODUCTION

1.1 Introduction to the study area

Sarai Saleh is one of the 44 union councils of Haripur district in Khyber Pakhtunkhwa province of Pakistan. Geographically Sarai Saleh is located 5km northeast of the Haripur city. It is 597 km above the sea level. It is mostly green valley surrounded by hills covering an area of 2-3 square km. it has a famous Dor river located in the northern part of the main GT road called Shahr-e-Resham (Silk Way). Sarai Salah is one of the very old town of Haripur. There is an old railway station founded in 1928 located in southern side of GT road. The primary language spoken in the region is Hindko. Pahaari is spoken by some inhabitants. The climate in Sarai Saleh is warm and temperate. The summers are much rainier than the winters in Sarai Saleh and average rain fall is 893 mm.

There is only one main road from Hassan Abdal to Manshera. All heavy and light traffic uses this road, usually traffic jams and accidents are seen and there are no underpasses and flyovers on this road The government of Pakistan has planned a motorway to overcome the problems faced by citizens in travelling from Hassan Abdal to Havellian and this would help in decreasing the load on the existing one road to which all the towns are linked.

This expressway project is in 3 pakges.Pakage 1 is from Hassan abdal to Jari Kass carried out by **GRC** (Ghulam Rasool & company). Package 2 is from Jari kass to Sarai Salah carried out by **AMJ** (Ameer Jan & company).Our present research is to focus on the quality assurance of construction material for earth work from Sarai Salah to Havellia package 3 carried out by **ZKB J.V Limak** (Turkish company). During our field investigation following test have been performed i.e. sieve analysis, plastic index, plastic limit, liquid limit, CBR (California bearing ratio) and Los Angeles by using **AASHTO** (American Association of State Highways and Transportation Officials) and

ASTM.(American Society of Testing and Materials) based procedures and concrete test that is using in bridges.

1.2 Location and accessibility

Sarai Saleh is the tehsil of Haripur district in Khyber Pakhtunkhwa province of Pakistan. Geographically Sarai Saleh is located 5km northeast of the Haripur city. It is 597 km above the sea level. It is mostly green valley surrounded by hills covering an area of 2-3 square km. it has a famous Dor river located in the northern part of the main GT road called Shahr-e-Resham (Silk Way). The coordinates of the sample area as following; sample 1 (33^{0}58’9.81” N) and the longitude is (72^{0}59’35” E). Sample 2 (33^{0}57’25” N) and the longitude is (73^{0}0’44” E). Sample 3 (33^{0}58’10” N) and the longitude is (72^{0}03’51” E)

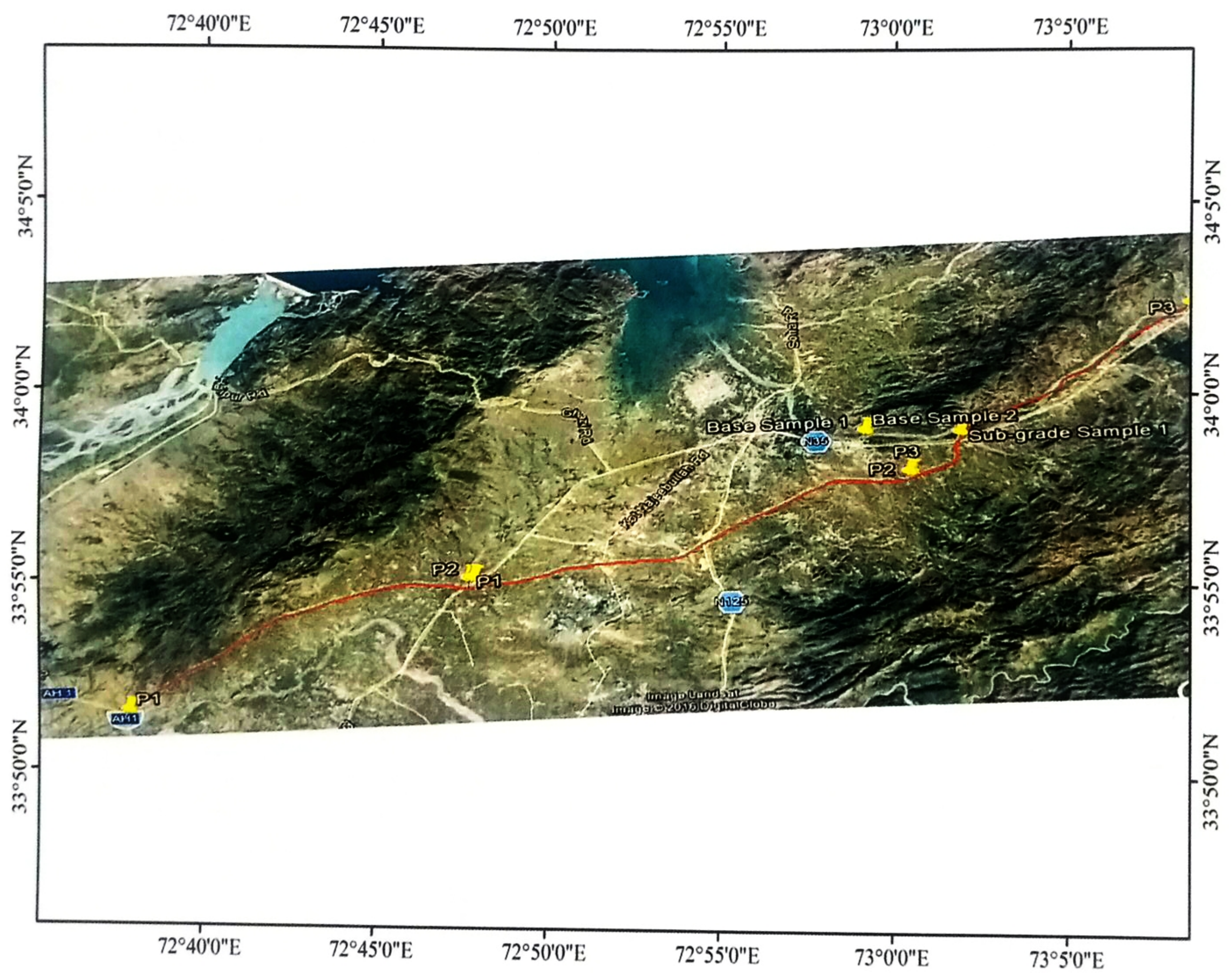


Figure 1.1 (map of Hazara motorway)

1.3 Purpose of the project

The primary purpose of the thesis work was to determine the strength properties of construction material used at Hazara Motorway Package 3 (Sari Saleh to Havellian) and its comparison with international standards. To assess the suitability of construction material used in road. The primary work is to conduct the geotechnical survey to find out the texture and nature of the soil. Several major tests were conducted on the site for finding compaction of surface and to find density and moisture content of material to be used in the project.

1.4 Methodology

Number of soil and aggregate tests were carried out to evaluate the quality of material used for motorway construction (Sari Saleh to Havellian) different geotechnical tests were performed at site and in laboratory in order to compare it with international standards to assure the suitability for construction of road. The soil test include Moisture content, Plastic limit, liquid limit, Sieve analysis, CBR (California Bearing Ratio) and Modified Proctor test. Aggregate test includes sand equivalence and Los Angeles of abrasion.

CHAPTER 2

STRATIGRAPHY AND TECTONICS OF PAKISTAN

2.1 Tectonic settings

Himalaya, the youngest and perhaps the most magnificent of all the continent – continent collisions on the earth was created due to the collision involving Eurasian plate in the north with the Indian plate in the south at about 65 -50 Ma(Gansser, 1964; LeFort, 1975; Molnar and Tapponier, 1975; Fraser et al., 2001).

The Himalaya originated as a result of the separation of the Indian plate from the mother Gondwanaland and its northward drift at about 130 Ma as a result of this northward drift of Indian plate the Neo-Tethys started shrinking which was located between the Indian and Eurasian plates. An intra-oceanic seduction generated Nuristan , Kandahar and Kohistan-Ladakh arcs at the time of Neo-Tethys closure. For a time of almost 40 million years, this arc magmatism continued. The Kohistan-ladakh arc collided onto the Eurasian plate in the north forming an Andean type of continental margin as a result of the back arc basin closure. According to Coward the collision between Kohistan-Ladakh arc and Eurasian plate gave birth to a boundary named as the Main Karakoram Thrust (MKT) at 72 hundred million years ago. The main Karakoram Thrust (MKT) separates igneous , meta -sedimentary and deformed sedimentary rocks of the southern Eurasian plate from the Kohistan Island Arc (KIA) terrain situated in the north.

In Hazara Basin (HB), the tectonic activities severely disturbed the area as it is a part of active Himalayan belt. The Hazara Basin is located within the Hazara- Fold- and-Thrust- Belt (HFT) in lesser Himalaya. The HFTB is an east-west trending north- western sector of the main Himalayan –fold-and-Thrust-Belt and is bounded by Main Boundary Thrust (MBT) in south and Panjal Thrust (PT) in the north. The HFTB merges into the Hazara-Kashmir Syntaxis towards northeast. Major structural features of the region includes Hazara-Kashmir Syntaxis (HKS), Main Karakoram Thrust (MKT), Main Mantle Thrust (MMT), Main Boundary Thrust (MBT), Panjal Thrust (PT) and Hazara Thrust (HT).

The litho-tectonic domains of Pakistani Himalayas have been explained briefly as below.

2.1.1 Main Karakorum Thrust

The Main Karakorum Thrust (MKT) is making the southern boundary of Karakorum block. In northern Pakistan MKT is present as a major tectonic feature. The Main Karakorum Thrust has resulted from collision of the north lying Karakorum plate and southward Kohistan Island Arc (KIA).

2.1.2 Main Boundary Thrust

The Main Boundary Thrust (MBT) is extended along the front of the Northern Fold and Thrust Belt around Hazara-Kashmir Syntaxes from Northeast to Southwest, representing the Southward migration of Himalayan deformation from the site of MMT in the north. The hanging wall of MBT carries the pre-collisional Paleozoic and Mesozoic sedimentary and meta- sedimentary rocks of the Northern Deformed Fold and Thrust Belt and post-collisional folded Miocene foreland basin deposits in its foot-wall. According to Seeber and Armbuster and Yeats and Lawrence the MBT is connected to the Hazara and Murree faults that bound the northern margins of Hazara and Kala Chitta Range.

2.1.3 Main Mantle Thrust

The Main Mantle Thrust (MMT) lies in the north of the Northern Deformed Fold and Thrust Belt (NDFTB). It involves the lower crust and is dipping towards north between 25-45 degrees. However, the formation of MMT did not stop the convergence and continued at the rate of 5 millimeter per year since Eocene that resulted in Continent-arc-Continent collision (Karakorum-Kohistan-India)

2.1.4 Hazara-Kashmir Syntaxis

Northern Pakistan is characterized by two major structural arcuations. Northern one is called Nanga-Parbat Syntaxis and southern one is called Hazara-Kashmir Syntaxis. The Hazara Syntaxis consists of a complex series of overlapping nappes made up of various Pre-Cambrian, Paleozoic and Mesozoic Formations which have been overthrust on a group of predominantly red-brown-coloured plastic sediments, the Murree Formation, of Tertiary age.

2.1.5 Panjal Thrust

The Panjal Thrust demarcates two zones; The Tethyan and Himalayan.The Panjal Thrust was regarded as an analogue of the Main Central Thrust (MCT) in the northwest Himalaya. The Panjal Thrust (PT) extends uninterrupted from the Panjal range towards NE through Poonch, Reshian and follows the eastern limb of the Hazara-Kashmir Syntaxis and terminates at its apex in Kaghan Valley.

2.1.6 Hazara Thrust

The Hazara Fold- Thrust (HT) is located in lesser Himalayas, bounded by Panjal Thrust in the north and MBT in the south. Hazara Fold Thrust Belt runs in the form of E-W elongated linear belt with turns northward in the east to merge into the Hazara-Kashmir syntaxis.

2.1.7 Punjab Fore-deep

The Punjab Fore-deep rims the southern-most extension of Himalayan mountain chain in Indo-Pakistani shield. Unconsolidated Quaternary sediments overlie the Punjab Fore deep and are the present day epicenter for the eroded debris from the Himalayan chains in the north.

2.2 Stratigraphy of hazara basin

The surrounding areas of Attock and Haripur basins can be divided into three tectonic blocks. The southern block is referred to as the Kala Chitta-Margalla Hill block. The central block is known as Nathia Gali Hissarthang block and northern block is known as Panjal-Khairabad block

2.2.1 Panjal-Khairabad block

Panjal-Khairabad block is composed of Proterozoic, Paleozoic and Mesozoic formations. These are briefly described below:

a) Proterozoic Formations

The oldest exposed rocks of Proterozoic age in this block are known as Gandaf Formation located, 3 Km north of Tarbella Dam. These are also exposed in the Gandghar range where these have transitional contact with the overlying Manki Formation. These rocks are mainly Carbonaceous and Calcareous Phyllite and Schist and Carbonaceous Marble. Manki Formation consists of Argillite, Slate, Phyllite and Argillaceous meta-siltstone. It is overlain by Shahkot Formation (Limestone), Utch Khattak Formation (Slate and Argillite) and Shekhai Formation (Dolomite and Arenaceous Limestone and Marble). The Tanawal Formation consists of Feldspathic Sandstone, Siltstone and Shale. It is exposed near Tarbella Dam.

b) Paleozoic and Mesozoic Formations

The Paleozoic strata are exposed in the northwestern margin of the Attock basin. Amber formation of early Cambrian age is exposed in this section. It overlies the Tanawal Formation and lithologically similar to the Sibran Formation of the Abbottabad group.

2.2.2 Nathiagali Hissarthang Block

Nathia-Gali Hissarthang block is composed of Proterozoic, Cambrian, Mesozoic and Tertiary age. These are briefly discussed below:

a) Proterozoic Formation

The oldest rocks of the Proterozoic age exposed in this block belong to the Hazara Formation. Shale and Sandstone are the dominated lithologies of this formation. The Dakhner Formation of Attock Cherat range is lithologically identical to the southern Hazara Formation. The exposed thickness of both formations are more than 1000m.

b) Cambrian Formation

Near Abbottabad, rocks are subdivided into three formations such as Sibran, Kakul and Tanawal Formation. The Kakul and Sibran Formations are part of Abbottabad group. Kakul formation consists of Tanakki conglomerates which are derived primarily from the overlying Hazara Formation. Tanawal formation consists of a lower Galdanian member composed of Siltstone, Mudstone, Glauconitic and Phosphatic Shales and Siltstone.

c) Mesozoic Formation

Mesozoic Datta Formation present at Northeast of Abbottabad consists of shale and sandstone. The overlying Shinwari Formation consists of the shale interbedded with limestone. Middle Jurassic Samana Suk Formation consists of limestone.

d) Tertiary Formation

Paleocene rocks unconformably overlie the Jurassic Samana Suk Formation near Hasan Abdal. Shale of the Patala Formation is the youngest bed rock in this area.

2.2.3 Kala Chitta-Margalla Hill Block

Kala Chitta –Margalla Hill Block is composed of Paleocene, Cenozoic and Mesozoic age. These are briefly discussed below

a) Paleocene Formation

The Hungu Formation dominantly white quartzitic sandstone. In Kala Chitta range, it overlies disconformably over the Kawagarh formation. Oldest exposed rocks in this block are limestone and marl of lower Triassic Miawali Formation. The Jurassic Samana Suk Formation is present in the east of Kala Chitta range.

b) Cenozoic formation

Lockhart Formation conformable ovelies the Hungu Formation. It consists of predominantly marine limestone and subordinate intercalations of marl and shale. Limestone is pale grey to dark grey , medium grained, thick bedded and fossiliferous. Marl is greyish black and fossiliferous. The shale is grey to greenish grey and has weakly developed cleavage.

c) Mesozoic formation

Chichali formation comprises of mainly sandstone and shale. Sandstone is greyish to green, glaouconitic and massive hard shale is greenish black thin bedded and fissile. It is of Late Jurrasic age. Lumshiwal Formation is generally grey, thick bedded feldspathic and ferrogenous sandstone. However, it contains silty and sandy glaouconitic shale in the basal part. Samana Suk Formation is composed of thick to medium bedded limestone but at places it is shelly or dolomitic limestone with interbedded marl and shale. Its contact with overlying Lumshiwal Formation is unconformable; however, the base is not exposed.

CHAPTER 3
ENGINEERING GEOLOGY

3.1 Introduction

Engineering geology is the application of the geologic science to engineering practice for the purpose of assuring that the geologic factors affecting the location, design, construction, operation and maintenance of engineering works.

Engineering geologists investigate and provide geologic and geotechnical recommendations, analysis, and design that are associated with human development. This type of studies can be utilized during the planning, civil engineering design and structures, and environmental impact analysis. Works completed by engineering geologists include; geologic hazards, geotechnical, material properties, landslide and slope stability.

3.2 Scope of engineering geology

Engineering geology may perform a vital role on in the daily life of human beings. The following are the major applications of Engineering Geology;

1. Applicable for Residential, commercial, and industrial development;
2. For government and military installation;
3. Public works such as power plant, tunnel, trenches, canal, dam, reservoir, building, road, bridges etc.
4. For Mining such as tunnel excavation;
5. It also provides knowledge about material that is used in construction.

By the start of twentieth century there was abundant documentary evidence that a prior understanding of geology should be an essential component of any major construction project. It was however, to be some decade before this come to be universally recognized, and still longer for the consequences to begin to be implemented. Civil engineers, for some time to come, either ignored geology or sought out geological knowledge for themselves.

Although the growth of engineering geology in the first four decade of the twentieth century can be best illustrated from the United State, there were significant developments elsewhere. The occurrence of major landslides in quick clays on the railway system in Sweden resulted in the establishment of first, a research institute and then a geotechnical commission in 1914. Maurice Lugeon, in France, became closely associated with the geological aspect of the sitting, investigation and construction of dams. His contribution is still preserved as the name of the unit applied to the result of in situ permeability tests in boreholes in rock which he devised (Lugeon, 1993). The status which engineering geology had achieved by the late 1930's is effectively illustrated by Robert Leggett's book (1939, 1962), which was derived from a comprehensive study of case history experience. Leggett recognized that his approach was descriptive; containing neither mathematics nor formulae, cogently arguing that there was no substitute for sound observation and good judgment. By this time, therefore, there was increasing acceptance and application of the role of geology in civil engineering of many countries. Engineering students were being taught some geology and engineering geology on a systematic basis, but training of geologists in engineering geology was effectively non-existent. The consequence was that geological advice on engineering projects was still normally provided by geological consultants who were commonly based in universities. The large scale deployment of geologists on engineering projects, as in the United States, was exceptional.

During the opening decades of the twentieth century Terghazi recognized that close link existed between geology and the engineering behavior of soils and rocks and throughout his life and writings, there was a continual search to understand the geological influence on engineering projects through engineering geology.

The growth in construction from the 1940's onwards resulted in both larger and more sophisticated structures being built as well as the need to develop sites which were less favorable geologically. The consequence was the recognition that some aspects of geological uncertainty could be answered by the application of fast, sophisticated computational methods in rock mechanics associated, for example, with concrete dam foundations, large underground structures, tunnel support and rock slope design which were capable of handling large amount of data. The construction industry is responsible for the building of visible structures that can directly influence and contribute to day-to-day life. The role of engineering geologists has progressively widened into the environmental impact of surface development and mineral extraction, the remediation of Brownfield sites, waste disposal, and geological conservation. In addition, engineering geologists have the appropriate skills and awareness needed for the identification of risks, and the consequences of geological hazards and processes. The aspect of engineering geology, which self–evidently embraces social issues, is generally termed environmental geology.

By the close of twentieth century, therefore, engineering geology had established itself, with soil mechanics and rock mechanics, as one of the component members of the field of geotechnical engineering. The strength of the subject undoubtedly lies within the practical application of the subject which has, almost universally, been highly successful. However, unlike soil and rock mechanics, the practical application of the subject has not been matched by the development of a scientific rationale for engineering as a subject. It could be that the parts of the jigsaw exist but they have never been brought together to create a logical whole but, it may be that those parts of the jigsaw which appear to exist could never be fitted together.

3.3 Engineering aspect of Study area

The package 3 consists of thick sandstone and shale deposited. The area is mostly Plateau type. Half of the area is mountainous which contain sedimentary deposits. Several tests are conduct for the embankment, subgrade, sub base, and base. These tests help us to observe the geotechnical behavior of area from the Motorway is going to be passing.

3.4 ASSHTO soil classification.

Table 3.1 soil classification for layer

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	…	…	…	…	…	…	…	…	…	…
0.425 (No. 40)	30 max	50 max	51 min	…	…	…	…	…	…	…	…
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	…		…	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			
Group index (GI)	Zero			4 max				8Max	12 max	16 max	20 max
Max .Dry density T/m3	2.08min			1.29 to 2.03		1.92 to 2.08		1.76 to1.92	1.28 to 1.60	1.28 to 1.76	1.28 to 1.76
C.B.R %	60-90	20-70		10-80		8-70		4-20	2-7	2-15	2-15

Table 3.2 Grading Requirement for Aggregate Base Material

Sieve Designation		Mass Percent Passing Grading	
mm	inch	A	B
50.0	2	100	100
25.0	1	70-5	75-95
9.5	3/8	30-65	40-75
4.75	No.4	25-55	30-60
2.00	No.10	15-40	20-50
0.425	No.40	8-20	12-25
0.075	No.200	2-8	5-10

CHAPTER 4

METHODOLOGY

4.1 Gradation (sieve analysis) for soil

The grain size characteristics of soils are evaluated by a sieve analysis. Gradation (Sieve analysis) (AASHTO T-27) is a test conducted for the classification of soil materials. There are different classes of soil, started from A-1 to A-7 based on a standard classification called AASHTO (American Association of state Highways and Transportation Officials). This test is basically done for all layers i.e. embankment, sub-base, sub-grade, water bound and asphalt in roads construction. We did it in embankment stage. The standard particle size analysis testing are distributed in a certain size range is the relative proportions of the different grain sizes.

4.1.1 Needs and scope

Particle size analysis is used in the classification of the soil. Soil components may be categorized as gravel, sand, silt, or clay. Curve obtained from Particle size distribution from the data is also being used for Earth Dam filter construction, permeability test is more commonly used to predict the movement of soil water and particle size analysis may use the information obtained from it.

4.1.2 Required equipment

Includes an extra pan and cover the body's stack(With an accuracy of 0.01 g),Rubber pestle, and mortar Mechanical sieve Shaker, Oven, Scale Electronic balance, Bowl, Sampling tray, Scoop, Quartering box.

Notice: use the balance to be 0.1% of the total weight of shoot samples must be sensitive enough.

4.1.3 Test procedure

The material proposed for the project was collected randomly from the site area. The collected samples were quartered through quartering box in order to mix it thoroughly and dried these in oven for 24 hours. After the dried sample was weighted and washed with passing from sieve No.40 and No.200, again washed sample was kept in oven for 24 hours. In the last step different numbers of sieves were used to find out the class of material from calculating the retaining and passing weights of the materials after shaking it through mechanical shaker. The sieve were placed above one another in such manner that the coarser sieve should be at the top and the finer one at the bottom.

Table 4.1.Result from embankment gradation.

Sieve No (Max Size, mm)	Cumulative weight. Retained (g)	Retained	Passing
4 (4.75mm)	2111	59.2	40.8
10 (2.0mm)	2371	66.5	33.5
40 (0.425mm)	2575	71.7	28.3
200 (0.075mm)	2756	77.3	22.7

Figure 4.1. Sieve analysis diagram.

4.1.4 Gradation test formulae

The following formulas are used to obtain the retained and passed percent of materials from different sieves. The results for different number of sieves are shown in table.

Retained % = cumulative retain weight/ Total dry weight x100

Passing% = 100- Retain %

Weight of dry samples = 7340

4.1.5 Precaution

1. The sieves must be arranged in a proper order i.e. 3/8, 4, 10, 40, and 200.
2. The sieves should be clean with a fine brush, should not scrap.
3. The sieves must be free of dust before use.

4. After shaking remove the sieves immediately

4.2.1 FDT (field density test):

When the compaction work is progressing in the field, knowing whether the specified unit weight has been achieved is useful. Sand cone method is the standard procedure for determining the field unit weight of compaction.

4.2.2 Apparatus:

1. Glass or plastic jar
2. Dry ottawa sand
3. Balance of 15 kg capacity
4. Calcium carbide (CaC_2)
5. Soil tray with central hole

4.2.3 Procedure and calculations:

a) Sand Cone Method

In this method we used sand cone method for determining the compacted dry unit weight of soil.(AASHTO T-224)

b) Procedure:

We drilled a hole on the ground surface which should be vertical and about 15cm in length. The diameter of the hole is 6 inches. The device consists of a glass or plastic jar with a metal cone attached at its top. The jar is filled with uniform dry Ottawa sand. The combined weight of the jar, the cone and the sand filling the jar is determined (W1). In the field, a small hole is excavated in the area where the soil has been compacted. If the weight of the moist small excavated from the hole (W2) is determined and the moisture content of the excavated soil is known, then we can calculate the dry unit weight of the soil can be obtained as (W3) by dividing the weight of moist soil and moisture content of the excavated soil as shown below in results.

After excavation of the hole, the cone with the sand filled jar attached to it is inverted and placed over the hole as shown in figure. Sand is allowed to flow out of the jar to fill the hole and the cone. After that, the combined weight of the jar, the cone and the remaining sand in the jar is determined (W4). By subtracting the W4 from W1 we get the weight of sand to fill the hole and cone (W5). Also note the weight of the sand to fill the cone only (WC) and dry unit weight of Ottawa sand from the calibration done in the laboratory.

To calculate the dry unit weight of the field soil we have to find the volume of the hole (v), which is calculated by first subtracting the WC from W5 then dividing it by dry unit weight of Ottawa sand. Dry unit weight of compaction made in the field can then be determined by dividing dry unit weight of excavated soil (W3) to volume of the hole (v).

Figure No. 4.2 sand cone apparatus (image taken from field test on motoway)

4.2.4 Calculations and formula

Dry Unit Weight of Excavated Soil $W3 = 1 + \frac{w2/w\%}{100}$

Where w =Moisture content

$$W5 = W1 - W4$$

W5= Weight of the sand to fill cone and hole

W4=Weight of Jar + cone + remaining sand

The volume of cone can be determined as $V = \frac{W_5 - W_c}{\gamma d\ (sand)}$

Where Wc = Weight of sand to fill the cone only

γd = Denity of ottawa sand

$$\gamma d = w3/v$$

Table 4.2 result of sand cone method on field

Determination of Density in Situ By Sand Cone Method (AASHTO-T-224)			
Field Density			
1	WT. OF Cont+Wet Soil	gm	4799
2	WT. OF Container	gm	10
3	WT. OF Wet Soil	gm	4789
4	WT. OF Sand Used+Cone	gm	8000
5	WT. OF Residual Sand+cone	gm	3376
6	WT. OF Sand in the Hole and Cone	gm	4624
7	WT. OF Sand to Fill Cone	gm	1649
8	WT. OF Sand Hole	gm	2975
9	Unit WT. OF Sand	gm/cc	1.413

10	Gross Vol of Hole	cc	2105
11	WET Unit of Soil	gm/cc	2.275
12	Dry Unit Of Soil	gm/cc	2.158
13	Optimum M.C	%	5.4%
14	Compaction	%	95.7%
15	Required Compaction	%	93%

4.3 Plasticity index (P.I.)

This test is used to find out the liquid limit and plastic limit of material used in the road construction. By applying this test is to determine the class of soil according to the AASHTO classification that either the material are from A-1, A-2 class or resemble to the other classes in AASHTO classification table.(AASHTO T-89)

Plastic index = Liquid limit- Plastic limit

4.3.1 Liquid limit test determination

4.3.2 Apparatus

Following are the apparatuses used in liquid limit test:

Evaporation plate about 4 ½ (114 mm) in diameter, Mill-mortar and pestle rubber cover, Sieve.40 (0.425 mm), Balance sensitive to 0.01 g Bottles, water Distillation. The mechanical liquid limit device and rod size of 3mm in diameter, manually operated-brass Cup Combined machine, tool and gauge grooving tools and Oven.

4.3.3 Procedure of test sample

After performing the gradation test, the material from same sample was taken for P.I test after passing it through sieve No. 4, Sieve No. 10, Sieve No. 40and Sieve No. 200in dry condition. Normally 200 grams of sample is used in this test. For this 15- 20 ml of water was added to the sample and was mixed thoroughly with the help of spatula. The liquid limit cup was filled up to half limit, and then by using grooving tool the material was divide into equal halves within the liquid limit cup. After grooving the material, it was analyzed for moisture content with the number of blows in three different steps until the material combined at central point in the range of 15-25, 20-30 and 25-35 strikes of blows. The sample was taken from the middle part of one and half inch attached sample and placed it in cane and weighed it which was the required weight (cane weight plus weight sample). The cane number and cane weight was noted and placed it in oven for 24 hours. When the oven process completed, take the cane sample in room temperature for cooling. Again weighed this dry sample cane which gave us final weight (cane weight plus dry sample). The height of sample in cup was 1 inch and number of blows per second was two times. This process was repeated three times using the same materials. The results for plastic index are shown in detail.

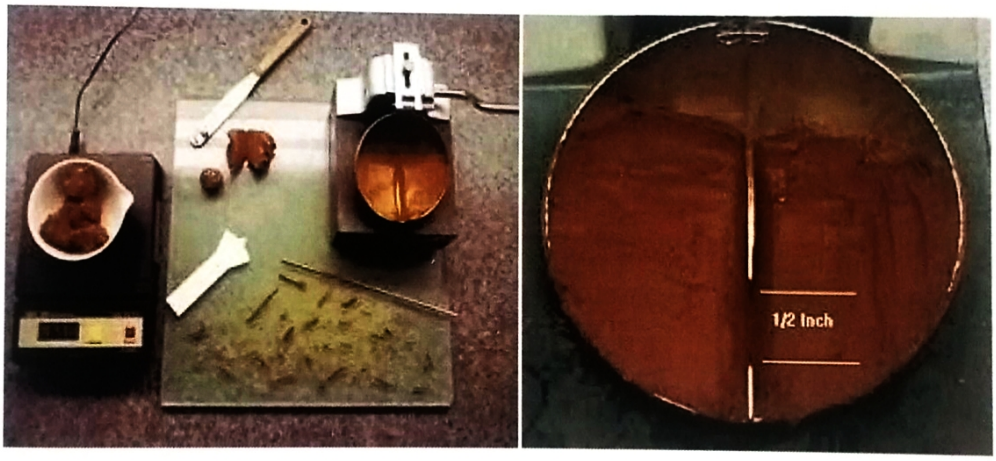


Figure 4.3.Apparatus for determination of liquid limit and plastic limit.

4.3.4 Precautions

1. The test should always be processed from dryer to the wetter condition of the soil
2. The breeding of the edge should be avoided and the cut should be very fine
3. The handle rotation speed should be uniform
4. The apparatus should be clean and should be placed in a proper place

4.4 Plastic limit test determination

4.4.1 Apparatus

Following are the apparatuses used in plastic limit test:

Evaporation dishes, Pulverizing machines,rubber coated mortar and pestle,U.S. No.40 sie ve (0.425 mm) spatula, about3 inches (75 mm) inlength and about¾ inch (19 mm) wide, Balance sensitiveto 0.01 g,Watering bottle with distilled dematerialized water , Surface for rolling a ground glass plate, Oven drying oven for controlled drying moisture 230°±9° F (110°±5° C) have the ability to maintain the temperature, A ⅛ (3 mm) diameter rod to help the operator to predict the size and Desiccators.

4.4.2 Procedure

For plastic limit determination test the sample was taken and placed on glass plate and rolled it to make 1 inch thread. Check the diameter of thread with the rod of 3mm diameter. After appearing cracks in the thread it was kept in the cane and noted its weight (cane weight plus thread weight). After taking the weight the sample was put in the oven for drying up to 4 hours. The dry weight was taken by removing the sample from oven and the results were determined from the calculated weights.(AASHTO T-89)

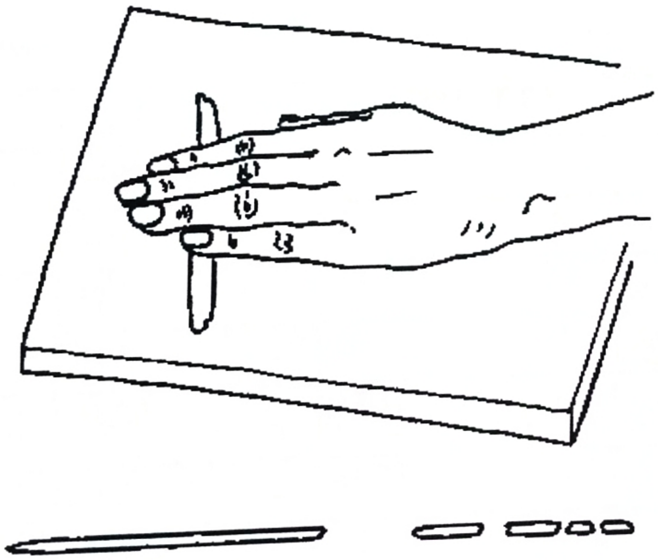


Figure 4.4.Method of rolling thread on glass plate

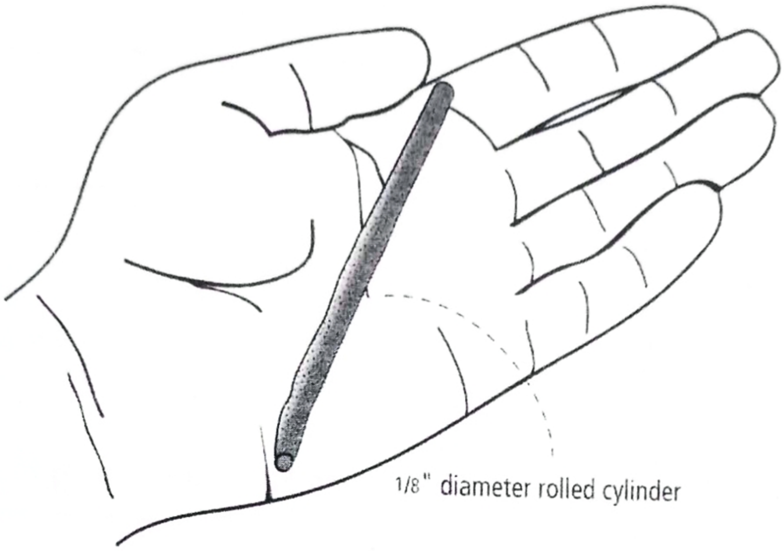


Figure 4.5.Diameter of roll thread .

4.4.3 Precautions

1. Clean the glass plate and container thoroughly before use
2. The materials/soil and water should be mixed thoroughly and allow for 15 minutes for maturation. A small amount of water should be added step by step
3. Graph 4.1.Liquid limit value.

Table 4.3 Result of Consistency

Test method : AASHTO T 89					
Type of test	**Liquid limit**			**Plastic limit**	
Test no	1	2	3	1	2
Number of Blows	16	24	34		
Container NO	**M**	**D**	**O**	K	L
Container + Wet Soil (g)	51.20	47.81	42.14	26.90	26.86
Container + Dry Soil (g)	43.61	41.25	37.00	25.21	24.99
Weight of Water (g)	7.59	6.56	5.14	1.69	1.87
Weight of Container (g)	18.48	18.72	18.82	18.78	17.95

Weight of Dry Soil (g)	25.13	22.53	18.18	6.43	7.04
Moisture Content %	30.2	29.1	28.3	26.3	26.6

PI = (L.L – P.L)

PI = (29.2 – 26.5) = 2.7

Table 4.4 .Result of plastic limit, liquid limit and plastic index.

Liquid limit	29.2
Plastic limit	26.5
Plasticity index	2.7

4.5 Modified Proctor test

In Modified Proctor Test we state the method of soil passing through sieve #4. OMC (Optimum Moisture Content) and MDD (Maximum Dry Density) values are the basic requirement for FDT (Field Density Test) and CBR (California Bearing Ratio). For these two values we do the modified proctor test. Proctor test is normally conducted for materials of all layers i.e embankment, sub-grade, sub-base, aggregates and water bound materials etc. (AASHTO(T-180)D)

4.5.1 Apparatus

The following are the basic apparatus used in proctor test.

Cane, gloves, steel hammer, rubber hammer, tray, mole tube, scoop, sieve No. 4, mould, balance, rammer (10 lb., 18 inches), brush, graduated tube, wash bottle and straight edge plate etc.

4.5.2 Procedure

For modified proctor test a representative sample was collected through random method from site. This sample was further quartered in the laboratory in order to separate the coarse and fine material after passing from sieve No. 4. After drying the sample in oven, 7000 grams of sample was taken in sampling tray and the specific mole % of water (3% water) was added in materials to mix it thoroughly by using hands wearing gloves. Water mole percent = water percent / Total weight of sample x 100.

After mixing the water, the whole sample was distributed equally into five equal layers. Furthermore, the sample was taken in the mould with the help of scoop in five layers and each layer was pressed with rammer by giving 56 numbers of blows. The collar attached to the upper part of mould was removed, the material was leveled with the help of straight edge and weight of mould and soil was noted. After finding the weight material was removed from the mould and further water of specified mole % was added. Similarly the material was divided again into five equal layers and same process was repeated till to find the weight of mould and soil. This procedure was repeated again and again until at fifth attempt the weight of material plus mould lowered down because of adding more and more water. All samples which have been placed in oven for determination of moisture content have been removed after 24 hour and weighted for dry weight. From the data calculation moisture content, dry density, and wet density were determined. (AASHTO T-180 D).

4.5.3 Modified Proctor test formulae

Weight of sample = (weight of sample + Mould) – (Weight of Mould)

Wet Density = weight of wet soil in mould / volume of mould

Dry Density = Wet density / 100 + Moisture content x 100

Weight of water = (weight of cane + wet sample) – (Weight of cane + Dry sample)

Weight of Dry Soil = (Weight of cane + Dry sample) – (Weight of cane).

Maximum Dry Density = 2.220 gm/cc

Optimum Moisture Content = 6.2%

Fig.4.6. proctor test machine

4.5.4 Precautions

1. The iron rod should be fallen from equal heights
2. The materials should be pressed equally through iron rod (rammer) in each layer.
3. After completing the test clean the mould through brush from all sides before weighing.

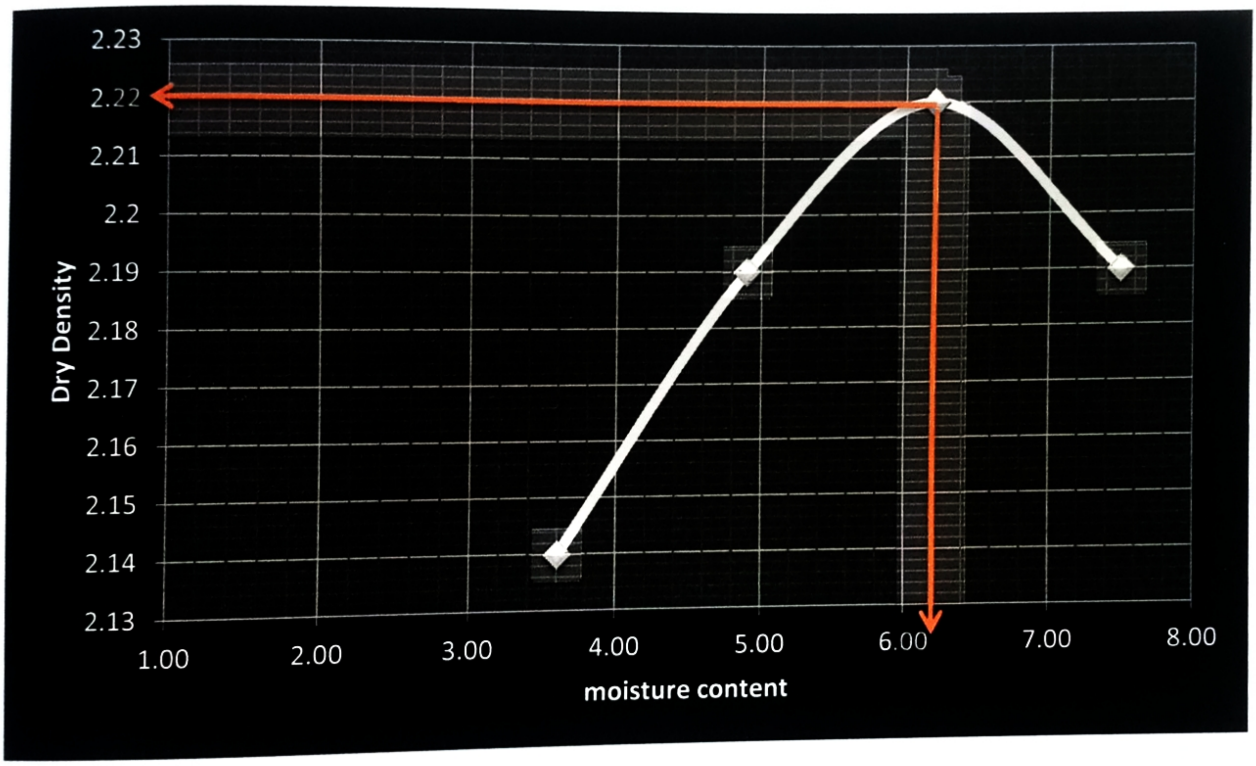
Table 4.5 Results of moisture content for proctor test.

Mould wt: 5697 Mould vol: 2137

Moisture Content **Mold Weight: 5105…Mold Volume: 2103**					
Container No	A-4	B-4	B-5	C-6	G-1
Wt. of container (g)	138.7	141.0	136.0	130.7	135.1
Wt. of wet soil + container (g)	399.0	410.9	388.6	429.4	419.5
Wt. of dry soil + container (g)	393.4	401.5	376.8	412.0	399.7
Wt. of dry soil (g)	254.7	260.5	240.8	281.3	264.6
Wt. of water (g)	5.6	9.4	11.8	17.4	19.8
Moisture Content %	2.2	3.6	4.9	6.2	7.5

Table 4.6 Results of maximum dry density for proctor test.

Maximum Density No of Blows: 56					
Determination No :	1	2	3	4	5
Wt. of Mold +Wet Soil (g)	10240	10445	10606	10736	10727
Wt. of wet soil (g)	4543	4748	4909	5039	5030
Wet Density	2.126	2.222	2.297	2.358	2.354
Dry Density	2.080	2.145	2.190	2.220	2.190



Graph 4.1 |Result of Proctor Test

Results: Max dry density MDD: 2.220 gm. /cm3   OMC: 6.2 %

4.6 CBR (California bearing ratio)

This test is used for evaluating the bearing capacity of subgrade soil for design of flexible pavement. CBR (AASHTO T-193) test was developed by California state highway department of USA for evaluation of sub grade strain of highway and airfield pavement. The test is commonly known as CBR test.

4.6.1 Definition

California bearing ratio is defined as the ratio of load per unit area required to penetrate soil mass by a standard plunger of specified rate to that corresponding required for penetration of standard material. The standard material is the one defined and is having CBR value of 100 %. CBR value is usually determined at 2.5 mm or 5 mm penetration.

4.6.2 Apparatus

Straight edge, rammer, moulds with internal diameter 150 mm and height 175 mm with a detachable collar, a detachable base plate having perforation at the bottom, tripod, CBR machine, spacer disk of 148 mm diameter and a height of 47.7 mm, surcharge weight of mass 2.5 kg each having a central hole 53 mm. A slotted weight of 2.5 kg. Penetration plunger height 100 mm diameter 50 mm. loading machine having capacity of 5000 kg and capable of traveling vertically at a rate of 1.25 mm per minute. Sampling tray, soaking tank, filter paper, oven, swell gauge, rubber gloves, moisture cane and scoop. Compaction rammer, proving ring, dial gages, balance, mixing tools, tray to contain the specimen and measuring cylinder.

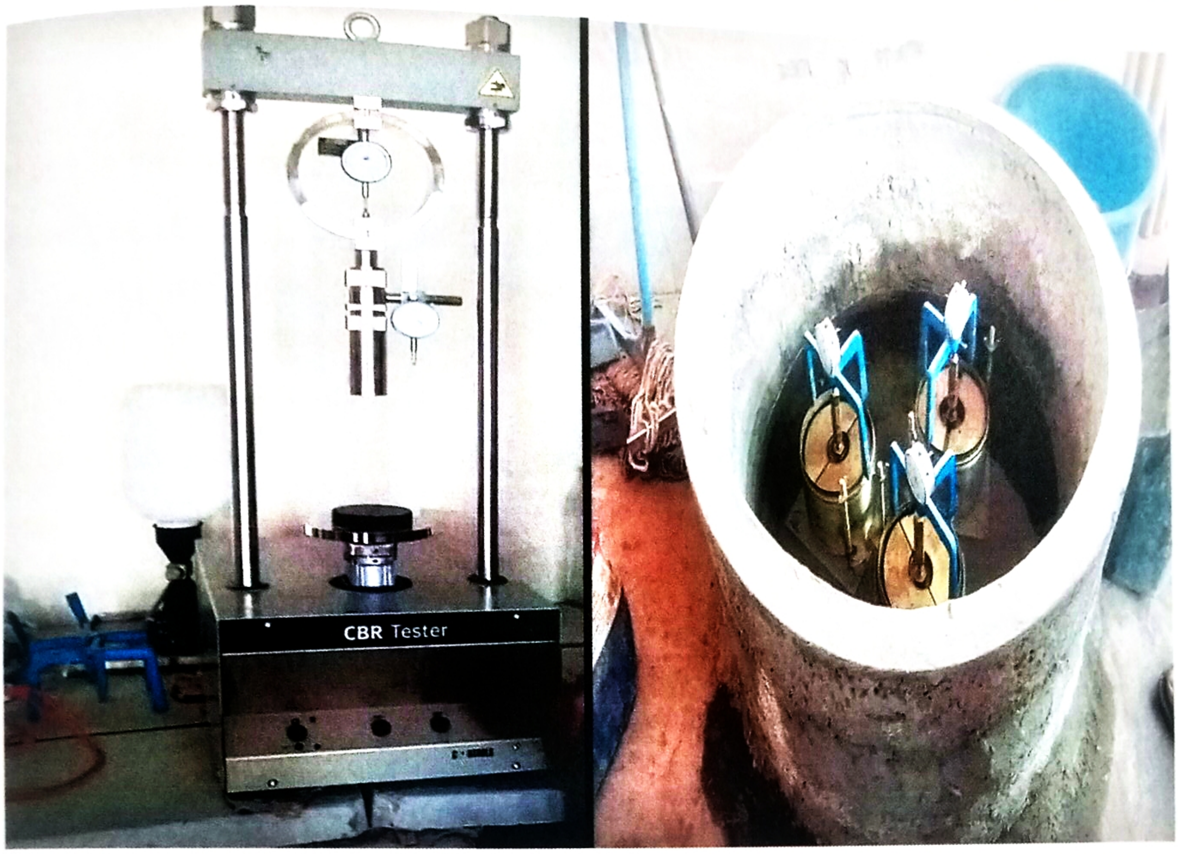


Figure 4.7.CBR test machine and mould soak in water.

4.6.3 Procedure

Normally 3 specimens each of about 7 kg must be compacted so that their compacted densities range from 95% to 100% generally with 10, 30 and 65 blows.We assemble the mould by placing the spacer disk facing bottom side at the bottom of base plate. Place a filter paper at the top of a spacer disk. We apply lubricating oil to the inner side of the mould, which is to prevent the stickiness of soil to the mould. We attach the collar and tighten the clamps.

According to AASHTO standard we took about 45 kg material and dried it in oven sieved through 20 mm sieve. The remaining soil was passing through sieve number 4 for further process. In this test we take 3 moulds and for each mould the weight of the sample was 7000 gram. We mix predetermine quantity of soil such that the water content of the soil is either equal to the (Optimum moisture content) or equal to the field moisture content. We Mix 469 ml of water to soil thoroughly and soaked for 2 to 3 hour. So, a mixture of uniform consistency was prepared. Then the sample was placed in 5 layer of mould and with the help of hammer 10 numbers of blows were given to each layer. In the same way the spacer disk and the filter paper was placed in the second mould and the sample was put in 5 layer for each layer now 30 blows were given. Same process was repeated for the third mould and this time for each layer 65 numbers of blows were produced.

The moisture content, dry density and wet density were found out respectively for each mould material. The stem plate, two surcharge weights and swell gauge were placed in each mould for 96 hour in order to know the swell. The reading of swell was noted every 24 hour for each mould through swell gauge. After 96 hour the material in mould was tested for crushing by taking the data through penetration piston and dial gauge. The results obtained by these tests are used with the empirical curves to determine the thickness of pavement and its component layers. This is the most widely used method for the design of flexible pavement. The results are shown in the following table

CBR= Unit load in kg / Standard load x 100

Table 4.7.Standard load value.

Penetration mm	Unit load kg/cm2	Total load kg (f)
2.5	70.0	1350
5.0	105	2055

Table 4.8 CBR (California bearing ratio test) result.

Optimum Moisture Content		7.0 %	
Number of blows	10	30	65
Dry Density (g/cc)	2.034	2.109	2.152
C.B.R at 0.1	38.3	50.4	64.6
C.B.R at 0.2	36.3	47.1	57.0

Socked CBR at	MDD	Unit	Pen 0.1
93%compaction	2.102	g/cc	49
95% compaction	2.147	g/cc	63

Table 4.9 Result of dry density.

DRY DENSITY			
Mould No	G	H	I
No of Blows	10	30	65
Mold + Soil (g)	11484	11672	11697
Wt of Mould (g)	6840	6882	6782
Wt of soil (g)	4644	4790	4915
Vol of Mould (g/cc)	2130	2121	2131
Wet Density (g/cc)	2.180	2.258	2.306
Dry Density (g/cc)	2.034	2.109	2.152
Moisture %	7.2	7.1	7.2

Table 4.10 Result of moisture content.

Moisture Content			
Can No	A	B	C
Wt of CAN (g)	136.5	138.0	139.2
Wt of WET Soil + CAN (g)	409.9	416.5	401.8
Wt of Dry Soil + CAN (g)	391.5	398.0	384.2
Wt of Water (g) -	18.4	18.5	17.6
Wt of Dry Soil	255.0	260.0	245.0
Moisture %	7.2	7.1	7.2

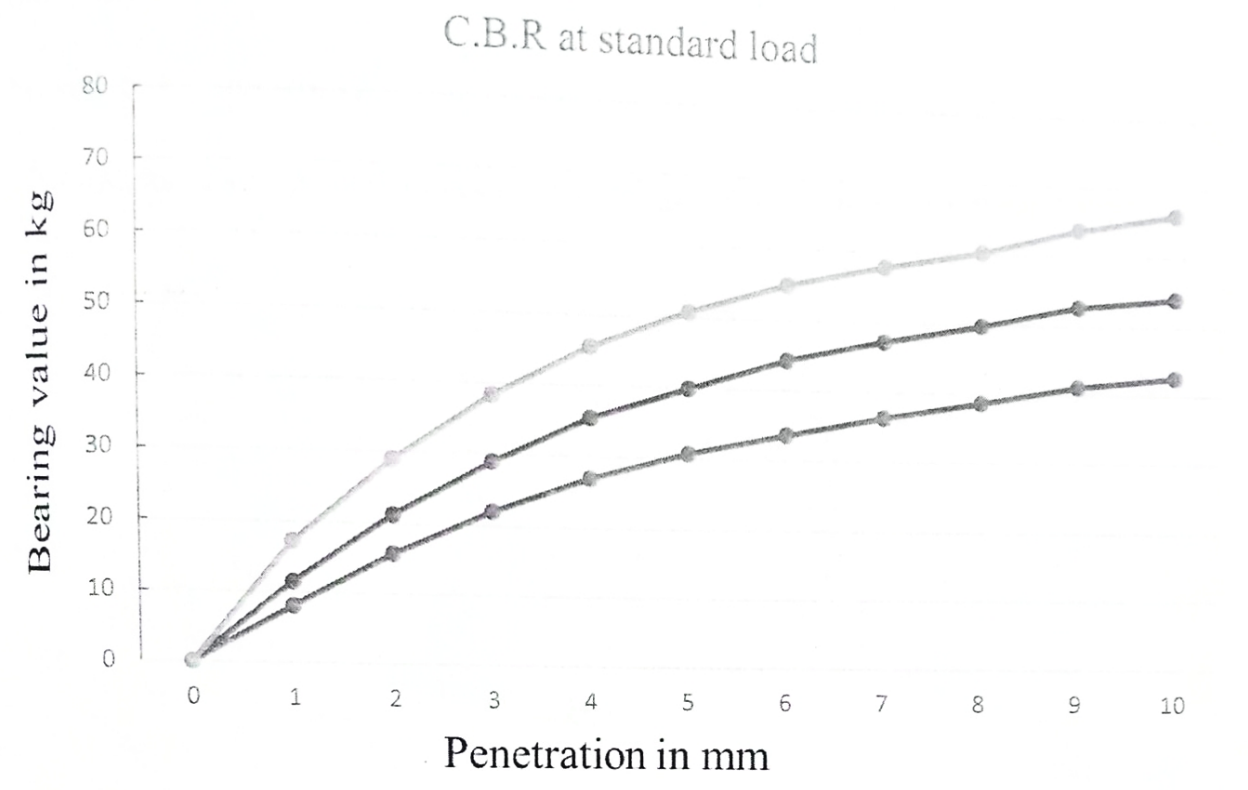
Table 4.11.Density C.B.R curve values for Mould No 1.

Mould no 1					
No of blows 10					
Penetration Inches / mm	Dial reading	Load in Kg	Bearing values in kg/cm3	Standard load Kg	CBR value
0.025/0.63	68	155.2	8.0		
0.050/1.27	132	301.4	15.6		
0.075/1.91	186	424.6	21.9		
0.100/2.54	228	520.5	26.9	70.3	38
0.125/3.175	260	593.6	30.7		
0.150/3.81	285	650.7	33.6		
0.175/4.48	308	703.2	36.3		
0.250/5.08	325	742.0	38.3	105.5	36
0.250/6.38	348	794.5	41.1		
0.300/7.62	361	824.2	42.6		

Mould no 2					
No of blows 30					
Penetration Inches / mm	Dial reading	Load in Kg	Bearing values in kg/cm3	Standard load Kg	CBR value
0.025/0.63	97	221.5	11.4		
0.050/1.27	179	408.7	21.1		
0.075/1.91	245	559.3	28.9		
0.100/2.54	300	684.9	35.4	70.3	50
0.125/3.175	338	771.7	39.9		

0.150/3.81	375	856.1	44.2		
0.175/4.48	400	913.2	47.2		
0.250/5.08	421	961.2	49.7	105.5	47
0.250/6.38	446	1018.2	52.6		
0.300/7.62	458	1045.0	54.0		

Mould no 3					
No of blows 65					
Penetration Inches / mm	Dial reading	Load in Kg	Bearing values in kg/cm3	Standard load Kg	CBR value
0.025/0.63	145	331.0	17.1		
0.050/1.27	245	559.3	28.9		
0.075/1.91	325	742.0	38.3		
0.100/2.54	385	879.0	45.4	70.3	65
0.125/3.175	430	981.1	50.7		
0.150/3.81	465	1061.6	54.9		
0.175/4.48	490	1118.7	57.3		
0.250/5.08	510	1164.4	60.2	105.5	57
0.250/6.38	540	1232.8	63.7		
0.300/7.62	560	1278.5	661		



Graph 4.2 curve of mould 1, 2, 3

4.7 Los Angeles Abrasion test

The Los Angeles abrasion test refers to the wearing and abrasion of material through mechanical process in a huge drum called Los Angeles Abrasion machine (AASHTO T-96). To produce the abrasive action by use of standard steel balls which when mixed with the aggregate and rotated in a drum for specific number of revolution cause impact on aggregate It is normally done for A1a, A1b or water bound material used in sub base or aggregate layers.

4.7.1 Apparatus

1. Los Angeles Abrasion machine,
2. Balance,
3. Scoop, oven,
4. Brush,

5. Sampling tray,

6. Sieve No. 1½ and No. 12.

7. 12 numbers of steel spherical balls (380-390 grams of each ball approximately),

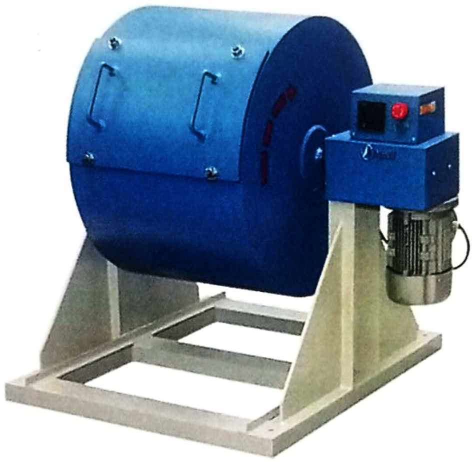


Figure 4.8 los Angeles abrasion machine

4.7.2 Procedure

For Los Angeles test, the sample of water bound material was washed and kept in oven for drying in the laboratory of the current project. The material was further passed through different sieves after 24 hours. After retaining weights from four different sieves with 1250 grams from each sieve, was further processed by giving 500 numbers of cycles in the loss Angeles Abrasion machine having 12 numbers of steel balls. The machine completed 500 cycles in 17 minutes. The cycles were 30 to 33 revolutions per minutes according to counter attached with the machine. After removing the material from machine the crushed material was passed through sieve No.10 and the weight was noted for the retained material. Calculate %age loss due to Abrasion by calculating the difference between the retained material (larger particles) compared to the original sample weight. The difference in weight is reported as a percent of the original weight and called the "percent loss".

4.7.3 Los Angeles Abrasion test calculation

Los Angeles = Initial weight of sample – weight of sample after test / total weight of sample x 100

Los Angeles = 5000 – 4135 / 5000 x 100

Los Angeles =865 / 5000 x 100

Los Angeles = 17.3 %

Maximum required = 40%

Table 4.12 Results of Los Angeles

Los Angeles Results	
Initial wt (g)	5000
Final wt after abrasion (g)	4135
Loss %	17.3

The maximum required value according to the standard is 40% , our values is 17.3% which lies in the range of required value.

4.8 Sieve analysis in Lab

These samples are taken form the source area of the project, use in the constructing of road. This material is mixture of aggregate and soil. Especially in the embankment, subgrade, sub base and base. (AASHTO T-27)

4.8.1 Sample A

Table 4.13 sieve analysis of sample A

Dry weight =2816

Sieve number	Cumulative Weight retained	Retained	Passing
4	403.6	14.3	85.7
8	1062.9	37.7	48
10	343.9	12.2	35.8
16	373.7	13.2	22.6
40	319.2	11.3	11.3
50	1.6	0.05	11.25
80	107.9	3.83	7.42
100	20.9	0.74	6.68
200	62.4	2.21	4.47
PAN	66.65	2.36	2.11

4.8.2 Sample B

Table 4.14 Sieve analysis of sample b.

Dry weight=2676.1

Sieve number	Cumulative Weight retained	Retained	Passing
4	530.8	19.8	80.2
8	574.8	21.4	58.8
10	300.9	11.2	47.6
16	721.7	26.9	20.7
40	294.5	11	9.7
50	16.3	0.60	9.1
80	157.9	5.90	3.2
100	23.4	0.87	2.33
200	10.68	0.39	1.94
PAN	5.9	0.22	1.72

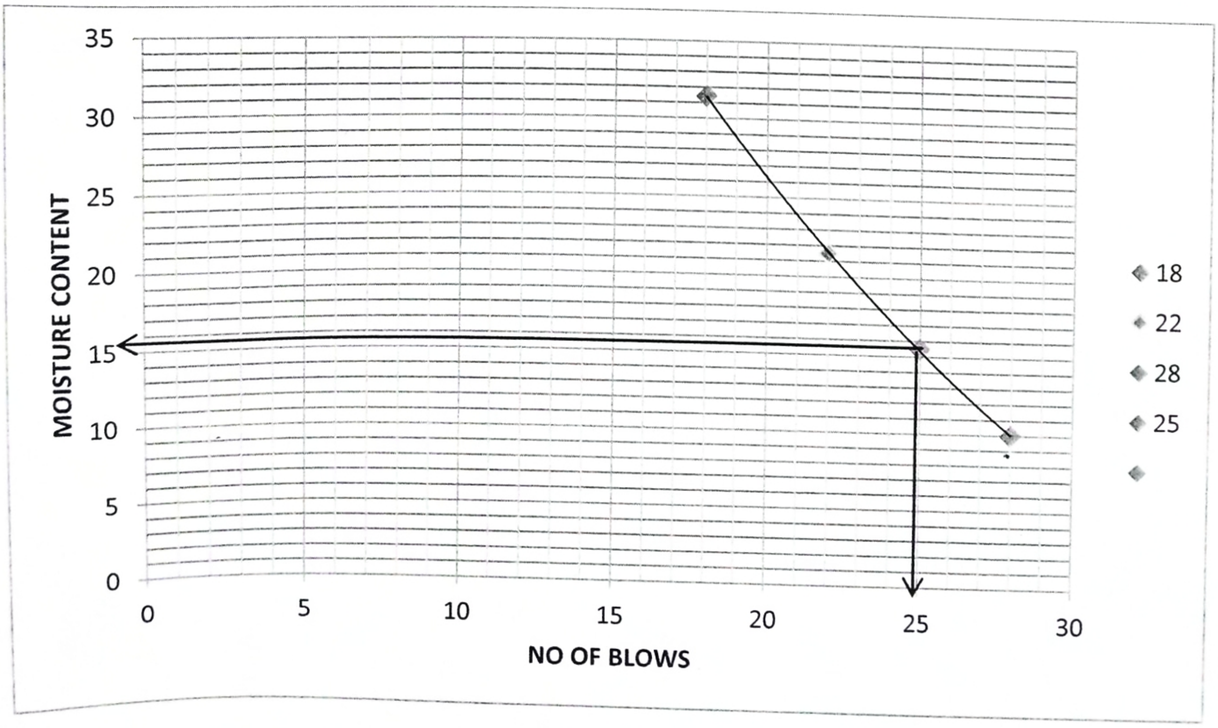
These results of these sample lies within ASSHTO standards. The results show that the material is good for the embankment as well as for the subgrade. The result suggests that subgrade material is excellent so there is no need of sub-base.

Fig 4.9 Sieve analysis in lab

4.9 Atterburg limit test in lab

Table 4.15 Result of Consistency

Test method : AASHTO T 89					
Type of test	**Liquid limit**			**Plastic limit**	
Test no	1	2	3	1	2
Number of Blows	18	22	28		
Container NO	**A**	**B**	**C**	A	B
Container + Wet Soil (g)	189.39	175.06	166.03	172.8	185.8
Container + Dry Soil (g)	176.8	165.75	158.2	166.6	175.6
Weight of Water (g)	18.89	8.92	9.52	6.2	8.7
Weight of Container (g)	116.68	119.55	116.3	116.6	116.1
Weight of Dry Soil (g)	60.12	46.2	41.9	48	56
Moisture Content %	31.4	19.3	22.72	10.3	12.9



Graph 4.3 Result of liquid limit.

4.9.1 Result

Table 4.15 .Result of plastic limit, liquid limit and plastic index.

Liquid limit	15.8
Plastic limit	11.6
Plasticity index	4.2

These results of liquid limit lies within ASSHTHO standards. The results show that the material is good for the subgrade. The result suggests that subgrade material is excellent so there is no need of sub-base.(AASHTO T-89)

RECOMMENDATION

It is recommended that further work should be carried out in order to propose material for surface course. The water table depth of the area is very shallow, so proper drainage system should be design for the project.

CONCLUSIONS

Different tests were conducted on the soil samples like soil to know the strength properties of material in road construction at Motorway in Package 3. These tests include sieve analysis test, liquid limit determination test, plastic limit determination test, field density test (FDT), California Bearing Ratio test (CBR), Los Angeles Abrasion test.

In Gradation (Sieve Analysis) test the passing from sieve No. 200 result was compared with AASHTO (American Association of State Highways and Transportation Officials) table which qualified A-2 sub class type material which is considered best soil according to AASHTO soil classification containing stones, gravel, sand and clay. For soil, Liquid limit determination value is 29.2% and plastic limit determination value is 26.5% which is standard value. The above results are satisfied according to the standard of AASHTO. The plasticity index value determined at lab was 2.7 which suggest that material is good for the embankment layer.

For proctor test (MDD) Maximum Dry Density value is 2.220 g/cc and (OMC) Optimum Moisture Content is 6.2 %, these values are further used in (FDT) Field Density Test are well within the limits of AASHTO classification and gives us the best compaction of embankment 95% and (OMC) is 5.4% which in the limit of NHA and the result of the proctor test.

The values for (LA) Los Angeles Abrasion were 18%, which is also according to the specification of project.

The results of tests carried out at earth material used for embankment fulfills requirements defined by American Association of State Highways and Transportation Officials which show that the material is suitable for the embankment. The test perform on sample taken from source area suggests that it is best for subgrade which means there is no need of sub base because the material which is using in embankment can also be used for subgrade which best for the project. The material comes from nearby source area which is granular. Sand silt and clay.

The result of test conducted in the lab on the material meet the requirement of ASSTHO for base material. Passing of material from sieve no 4, 10, 40, 200 shows that the material is excellent for the base.

All these tests were conducted for soil and aggregate. The results were concluded mostly in tabular form and were compared with the standard values for each test to determine their suitability and geotechnical behavior of material used in project. The results for all these different tests were satisfactory except the soil have plastic limit.

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