CHAPTER 1

INTRODUCTION

1.1 GENERAL BACKGROUND

Infrastructures in Nepal are being constructed day by day due to rapid growing population. With the process of civilization and increasing needs of people, transportation plays an important role for the development of the nation. For country like Nepal, road is the main means of transportation. For the people of Nepal, road transport is the only affordable mode for mobility next to walking. Road transport has many advantages such as wide geographical coverage, large influence area, flexibility and highest employment potential. So with the continuous increase in demand for road development it should be realized that good road network has to be developed in most planned way with most qualitative materials.

Rocks and soils are the major source of construction materials which are used as road aggregates. In Kathmandu, aggregates are supplied from the peripheries and from outside the valley. Generally, rocks of Cambrian and Ordovician Periods of the Tistung and the Chandragiri Formation (Stocklin 1980) respectively are the major sources of aggregates in the Kathmandu Valley. For supplying against an increasing demand of aggregates several quarry sites have been established in the Kathmandu Valley. They are mainly located in the Adeshwar in the west, North-East part and Tikabhairav in the Southern part of the Kathmandu Valley.

Being a developing country, roads are under construction and there is tendency of constructional growth in future. Highways and some other roads of major cities are black topped, whereas all others are simply graveled and left unbound (earthen). According to statistics of Strategic Road Network (2006/07), in Nepal 45.3% road is black topped, 21.9% graveled and 32.8% earthen.

Except highways constructed on foreign investments, all other roads in Nepal get damaged after the few months of construction, which is either due to improper use of materials of low grade or due to lack of proper and timely maintenance. More than 50% of the roads in Nepal are graveled or earthen. That means roads are unbound. Unbound roads are much more applicable in context of Nepal due to low economy of the country. Unbound pavements should possess better quality materials for better foundation, for proper drainage and load spreading. For undergoing construction it is essential that aggregates used in construction purpose are strong, hard, tough, durable, and should have proper shape and sizes, good adhesion and cementation characteristics.

Aggregates in the road construction are generally used without specification. Unfortunately, there is a growing practice of road construction without following the specifications or the standard of the materials used. Negligence of quality of aggregates to construct roads would be disastrous and such roads may not be long lasting (durable) and may be slippery, degradable etc. Therefore, being aware of these things, quality of road structure and aggregate should be kept at high priority. Thus, this study aims in evaluating rocks of the Chandragiri Formation from Adeshwar area for unbound pavements.

1.2 OBJECTIVES

- To evaluate the quality of crushed rock aggregates for unbound pavement.
- To analyze the grade and estimate the reserve.
- To develop safe mining techniques in the area.

1.3 LOCATION AND ACCESSIBILITY

The study area lies in Sitapila VDC–5, Kathmandu District of Bagmati Zone. Geographically, it extends from longitude $85^0 15' 46''$ to $85^0 16' 43''$ and latitude from $27^0\ 42'\ 93''$ to $27^0\ 44'\ 02''$ as shown in Fig. 1.1. The study area is bounded by Narayanthan in the north, Harisiddhi in the south, the Juge Khola in the east and the Lupan Khola in west covering the area of about 1.45 sq. km. The area is accessible by the motorable roads which links the quarry sites to the Ring road at Sitapila. Many foot trails are helpful in traverse.

1.4 TOPOGRAPHY AND DRAINAGE

The topography of the area is rugged. The highest altitude of the area is 1606 m at Kallabari and the lowest altitude is 1380 m near Halchok, on the river valley of Juge Khola.

The study area is the western sub–watershed of the Kathmandu Valley and exhibits dendritic drainage pattern as in Fig. 1.2. The Juge Khola and the Lupan Khola drain the area. These are the tributaries of the Manamati Kholsi.

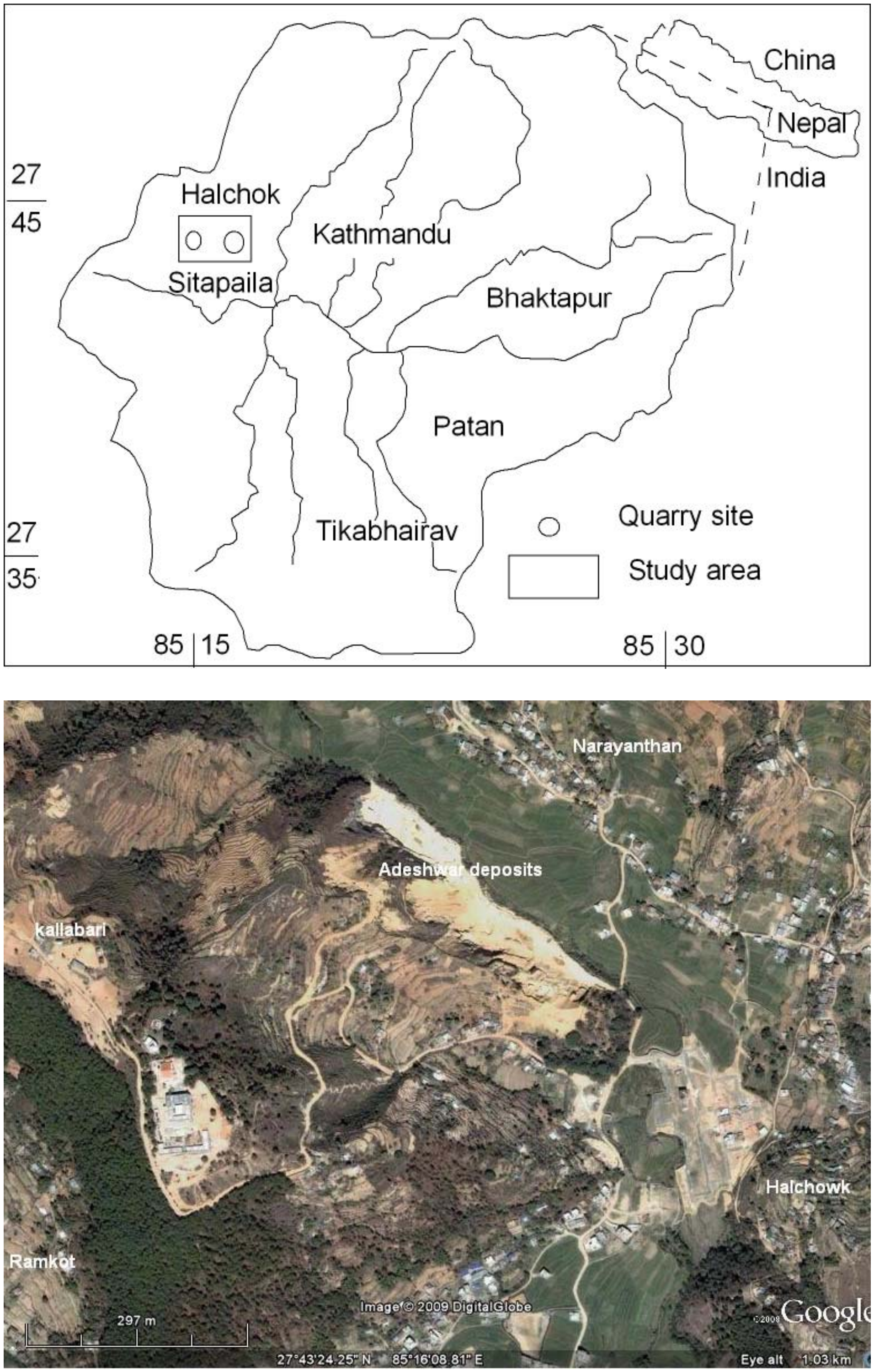


Fig. 1.1 Maps showing location of the study area

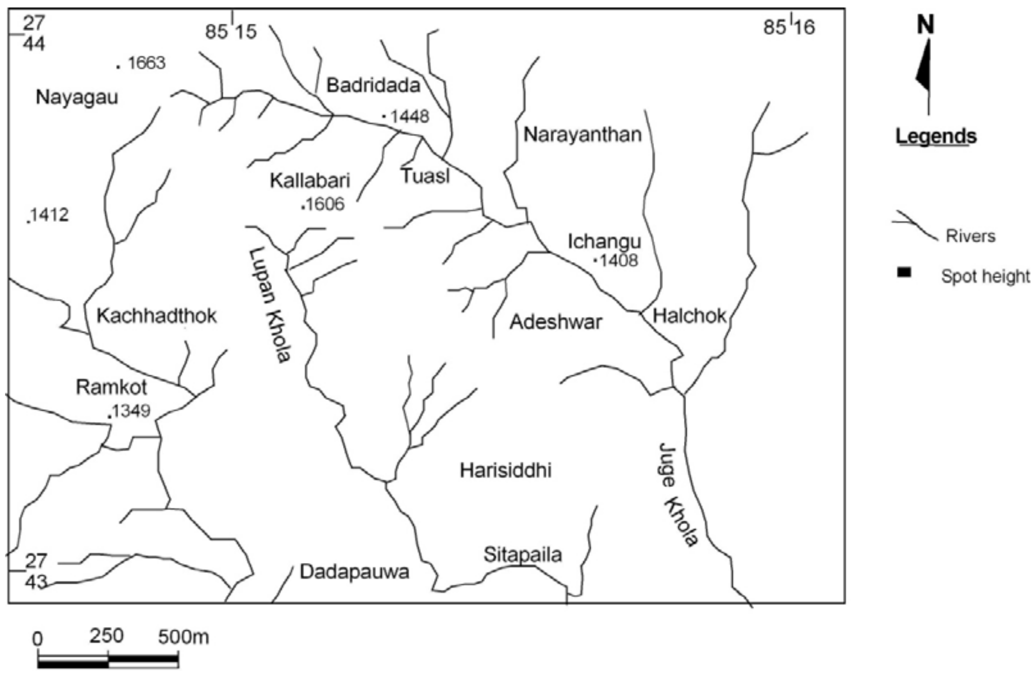


Fig. 1.2 Drainage map of the study area

1.5 CLIMATE

The Kathmandu valley lies in a warm temperate climatic zone. It is rather cold in winter and relatively hot to warm in summer. The average temperature is 18^0C and the mean minimum temperature of the coldest month is 1^0C.

1.6 METHODOLOGY

The methodology used in the present work can be summarized as:

- Desk study
- Field work
- Laboratory work
- Data analysis and interpretation

The work plans of the study are shown in the Fig. 1.3.

During desk study topographic maps, geological maps, various published and unpublished reports are reviewed and secondary data were collected consulting various reports, journals and related research papers.

The field work was carried out for 14 days, during field work geological traverse along streams, on motorable roads and foot trails were taken using Brunton compass. Detail columnar section was prepared. Representative samples were taken by bulk sampling methods from different horizons. Rock mass properties like color, discontinuities, weathering grade were also evaluated.

In laboratory, thin sections were prepared for compositional analysis. From the block of samples a set of core samples were drilled for determining water absorptions and dry density. Petrographic, physical, chemical and mechanical properties of the limestone were determined in the laboratory.

Data obtained from the physical, mechanical and chemical test were analyzed and evaluated for unbound pavements. The outcomes of the study are evaluation of crushed rock aggregates for unbound pavements, its geological map, cross sections, columnar section, and reserve calculation, development of mining techniques and analysis of environmental impact assessment of the area.

1.7 SCOPE AND LIMITATION OF THE STUDY

Demand of road aggregate is increasing. The evaluation of future supply and hence the planning of development projects can be rationally made only on the basis of availability of an aggregate. Quality and quantity of an aggregate play an important role in the durability of the road, so this study helps to evaluate the crushed rocks. Beside these, properties of geological materials have not been given priority as compared to engineering structure. The estimated reserve of the deposits during this study is only probable since no drilling data were available. The irregular and spacing samples used in the analysis would incorporate some extent of error to the overall quality of the deposits. Laboratory facilities available in the department, time and cost of chemical testing are other limitation of the study.

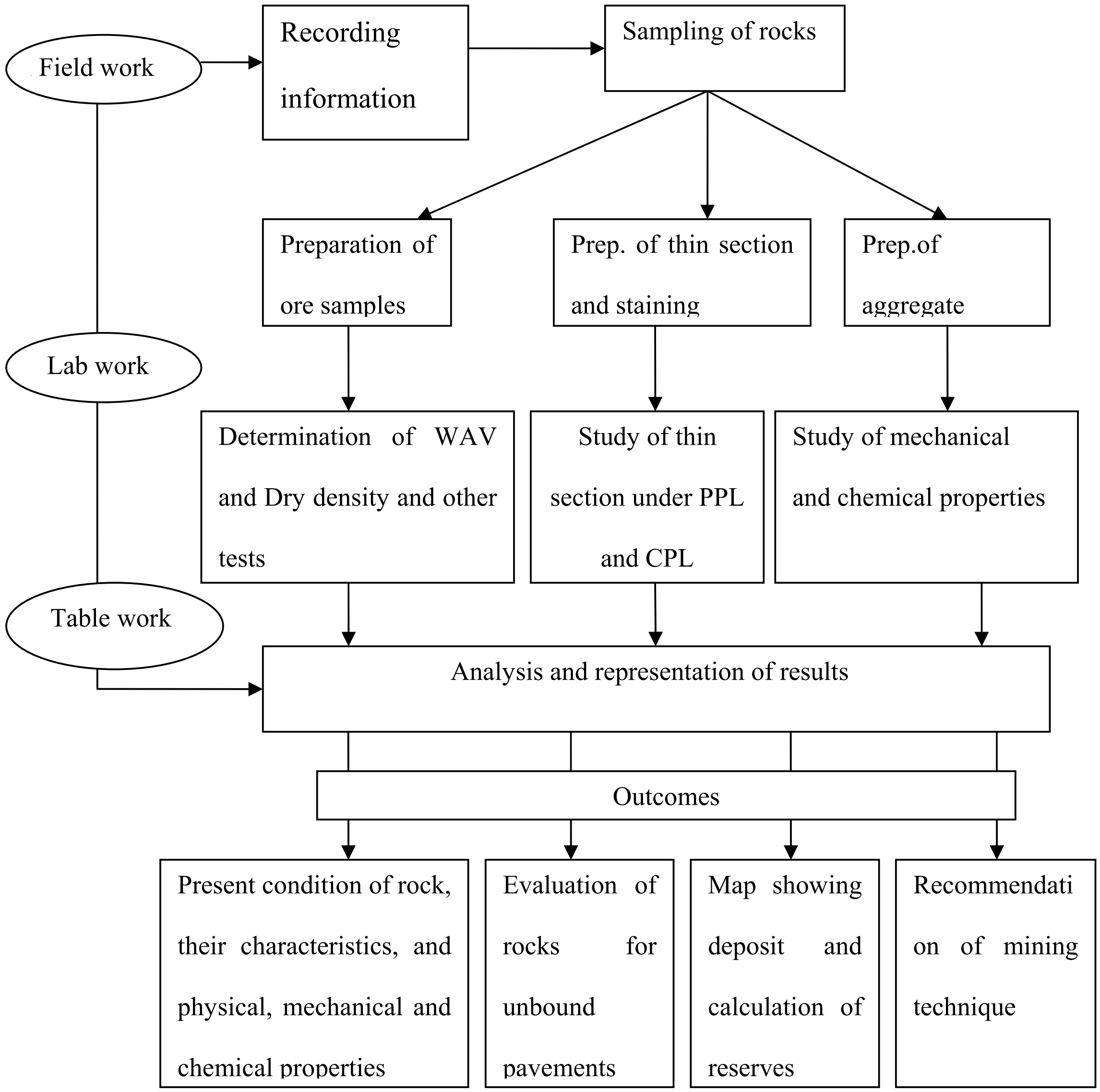


Fig. 1.3 Flow chart showing study plan

CHAPTER 2

PREVIOUS WORKS

A number of native and foreign geoscientist has worked in Geology of central Nepal at regional and local scales since 1875.

In Nepal, standard specification i.e. Nepal Road Standard (NRS) was first been published in 1970 with the objective of establishment of consistency in road design and necessities. This standard was therefore based on studies conducted with almost identical geographical features and traffic composition. NRS was revised later in 1998.

Nepal Bureau of Standard and Measurement (NBSM) established under the legal Act 1970 have published basic standards, testing methods and code of practice for aggregates since 1973.

Stocklin and Bhattarai (1977) and Stocklin (1980) studied the geology of the Kathmandu area and have included all the rocks of the Kathmandu area into the Kathmandu Complex which is further divided into the Bhimphedi Group and the conformably overlying the Phulchoki Group. The Bhimphedi Group consists of relatively high-grade metasedimentary rocks of Precambrian age. It is about 8 km thick succession of rocks, and is divided into six formations. Similarly, the Phulchauki Group comprises the unmetamorphosed or weakly metamorphosed sediments containing fossils of the early–middle Paleozoic periods. It is 5 to 6 km thick sequence of the rock, and is divisible into five formations (Table 2.1).

Table 2.1: Stratigraphic sequences of basement rocks of Kathmandu Basin (after Stocklin and Bhattarai 1977; Stocklin 1980)

Unit			Main Lithology	Approx. Thickness (m)	Age
Kathmandu complex	Phulchauki Group	Godavari Limestone	Limestone, dolomite	300	Devonian
		Chitlang Formation	Slate	1,000	Silurian
		Chandragiri Limestone	Limestone	2,000	Cambrian–Ordovician
		Sopyang Formation	Slate, calcphyllite	200	? Cambrian
		Tistung Formation	Metasandstone, phyllite	3,000	E. Cambrian or L. Precambrian
	Transition				
	Bhimphedi Group	Markhu Formation	Marble, schist	1,000	Precambrian
		Kulekhani Formation	Quartzite, schist	2,000	Precambrian
		Chisapani Quartzite	Quartzite	400	Precambrian
		Kalitar Formation	Schist, quartzite	2,000	Precambrian
		Bhainsedobhan Marble	Marble	800	Precambrian
		Raduwa Formation	Garnet-schist.	1,000	Precambrian

Smith and Collis (1993) reviewed the influence of the occurrence, mineral composition and geological history on the engineering properties of aggregate materials, identified main factors influencing aggregate behavior in various operational and environmental conditions and tried to identify relevant standards and other specifications and code of practice. They concluded that the performance of aggregates depends upon their intrinsic properties.

Tamrakar et al. (1999) reported porosity of the Siwalik sandstones from the Surai Khola areas lies between 2.88 and 7.68%, and elastic wave velocity between 1.59 and 4.36×10^3 m/s. The sandstones have the uniaxial compressive strength of the order of 0.5 to 1.0×10^2 MPa. These indices closely depend on void filling cement in sandstones. Tamrakar et al. (1999) also suggested that the mechanical properties of sandstones depends on the lithofacies and content of calcium carbonate cement, and are independent on stratigraphic horizon or deposition age.

Bajarnason et al. (2000) studied the unbound aggregates in road construction and have mainly focused on comparing different test methods and different quality of different materials. All the test results were put through the Factor analysis calculation using Varimax rotation. The results showed that for Icelandic basalt the durability test results and abrasion test results were dependent on the degree of alteration while the fragmentation test results were dependent on porosity.

Department of Road (DOR) 2001 published standard specification for road and bridge work, has completely described about pavement structures i.e. about the different layers, their grading and thickness. The physical requirement of materials to be used as road aggregates has also been mentioned.

Tamrakar et al. (2002) studied Siwalik sandstones from Central Nepal and concluded that dry density and porosity are related well with uniaxial compressive strength, point load index and modulus ratio.

Maharjan and Tamrakar (2003) evaluated quality of siltstones samples of the Tistung Formation for concrete aggregate from the Tikabhairav, Kathmandu Valley. They studied to reveal their petrography. The chemical properties of the crushed rock fragments were also investigated to determine their overall aggregate properties. The test result indicated that the aggregates were physically, mechanically and chemically sound. Maharjan and Tamrakar (2003) recommended that siltstones from the Tistung Formation of the Nallu Khola area were appropriate for concrete aggregates.

Khanna and Justo (2004) published 'Laboratory manual of Highway material testing.' They classified the pavement materials into different groups depending upon their physical strength or stability characteristics. Most of the test on soil, aggregates, bituminous materials and mixes were standardized, test methods details and specification based on the British Standard Institution, American Society for Testing materials, Asphalt Institute had been followed by this publication

Thapalia (2005) performed physico–mechanical test such as WA by weight ranges from 0.35 to 2.45, LA lies within 21.9 to 32.0, UCS ranges from 186 to 283.68 Mpa which showed that the rocks are medium to high strength, durable and suitable for both monumental and construction purposes.

Dhakal et al. (2006) conducted freeze–thaw experiments on the limestones and sandstones from Japan, and dolomite and schist from Nepal, and showed that the rock samples were resistant to deterioration and breaking. They also concluded that initiation and extension of cracks and subsequent wearing and deterioration occurred relatively faster in the rock having a high porosity. The weight loss due to freeze–thaw action was more on porous rocks than in non–porous ones. The durability of freezing–thawing was greatly influenced by mineralogy.

Maharjan, and Tamrakar (2007) evaluated quality of the river gravel from the Rapti River and the Narayani River for aggregates, and concluded that the majority of gravels of both rivers were prolate, triaxial, rod shaped with high roundness and high sphericity and of diverse chemical groups. The gravels were compositionally sound, mechanically and chemically durable and had good workability for road and concrete aggregates.

Tamrakar et al. (2007) analyzed physical, mechanical and petrographical properties of sandstones from the Siwalik group and concluded that the strength of the sandstones depends upon the % void, strong over weak contacts, strong cement over total cement packing, density and concavo–convex contacts.

Though, the works on geology and aggregates started few decades ago no more studies have been carried out in contest of aggregates for unbound pavements. Few organizations and geoscientist have worked on road design, aggregates properties and evaluated the aggregates on the basis of porosity, uniaxial compressive strength, point load index, and water absorption in terms of dry and weight density for aggregates and dimension stones. But these works are not sufficient for complete design and evaluation of road pavements. Therefore, present work is carried out for the complete analysis of qualitative and quantitative properties of aggregates and pavement design parameters in the context of Nepal.

CHAPTER 3

INTRODUCTION TO PAVEMENT STRUCTURE AND MATERIALS

3.1 INTRODUCTION

Roads in Nepal can be classified into four categories based on service and location (Nepal road standard, NRS 1998); National highways, Feeder roads, District roads and City roads.

3.2 CATEGORIES OF ROADS

3.2.1 National highways

National Highways are the main highways connecting to east to west and north to south of the nation. The road connecting National Highways to regional headquarters are also considered as National Highways. These serve directly to greater portion of the longer distance travel, provide consistently higher level of service in terms of traveling speeds and bear the inter community mobility. These roads are the main arterial routes passing through the length and breadth of the country as a whole. Intermediate carriageway width is 5.5 m.

3.2.2 Feeder roads

Feeder roads are the important roads of localized nature.

These serve the community wide interest and connect district headquarters and or zonal headquarters to National Highways.

3.2.3 District roads

District roads serve primarily by providing access to abutting land carrying little or no through movement. These roads serve as a collector to the feeder roads. These roads should give access to one or more villages to the nearest market or the higher types of road. Moderate travel speeds are typical on such roads.

3.2.4 City roads or streets

These include roads with in the urban limits except for the above classes passing through the city. These roads provide access to abutting residential, business or industrial properties.

3.3 PAVEMENT STRUCTURE

In most of the roads the natural soil is seldom strong enough to support the repeated application of even modest wheel loads without significant deformation. It is therefore necessary to interpose between the wheel and soil structure to supplement the natural strength of soil formation. The structure thus constructed is called "pavement".

According to "Design and Evaluation of Rigid and Flexible pavement" of Transportation Research Record (1990) pavement profiles can be two–layer, three–layer and four–layer profiles. “Design and Performance of Road Pavements, 1991 London” traditionally classified pavements as:

i) Flexible consisting of compacted stone beneath a bituminous surfacing. Modern flexible pavement consists of three layers, bituminous surfacing, road base and sub base as in Fig. 3.1a. The surfacing is generally subdivided into a wearing course and base course laid separately. The base and sub base may also be laid in composite form using different materials and are designated upper and lower road base or upper and lower sub base as in Fig. 3.1b.

ii) Rigid or concrete of a concrete slab lay either directly on the soil or on a shallow granular bed. Concrete pavements normally consist of only concrete slab and the sub base, but a bituminous surfacing may be added at a time of construction or later Fig. 3.2.

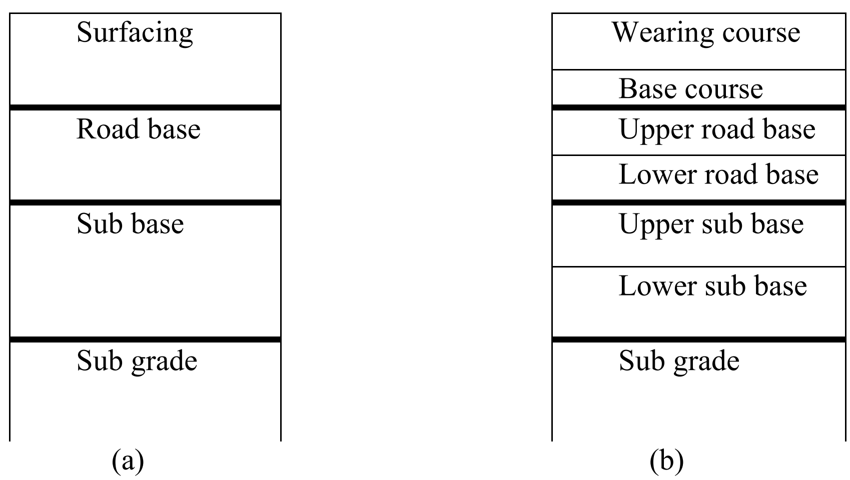


Fig. 3.1 Component of flexible pavement

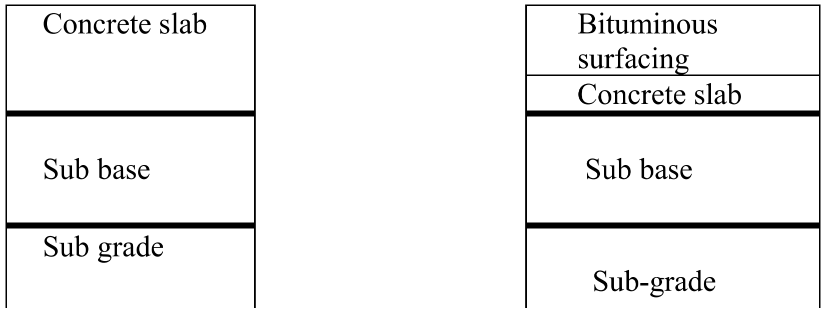


Fig. 3.2 Component of concrete pavement

Standard nomenclature for pavement as used in United Kingdom is illustrated in Fig. 3.3. Unbound layers are used in UK mainly for sub bases or capping but elsewhere may be used for bases or in the case of low volume roads the whole structure is considered as unbound pavement.

Different types of pavement are constructed on Roads for safe and comfortable movements of various types of vehicles at the desired speed. The standard nomenclature of the pavement used in Nepal consists of following layers according to the standard specification for road and bridge works, DOR (2001) and personal communication (Osti Durga Prasad–Senior Highway Consultant) (Fig. 3.4).

3.3.1 Sub–grade

This section deals with the treatment of upper layers of earthworks including preparation and surface treatment of the formation, the addition of layers of selected materials, the improvement of in situ materials by addition and mixing of selected materials or by addition or mixing of lime.

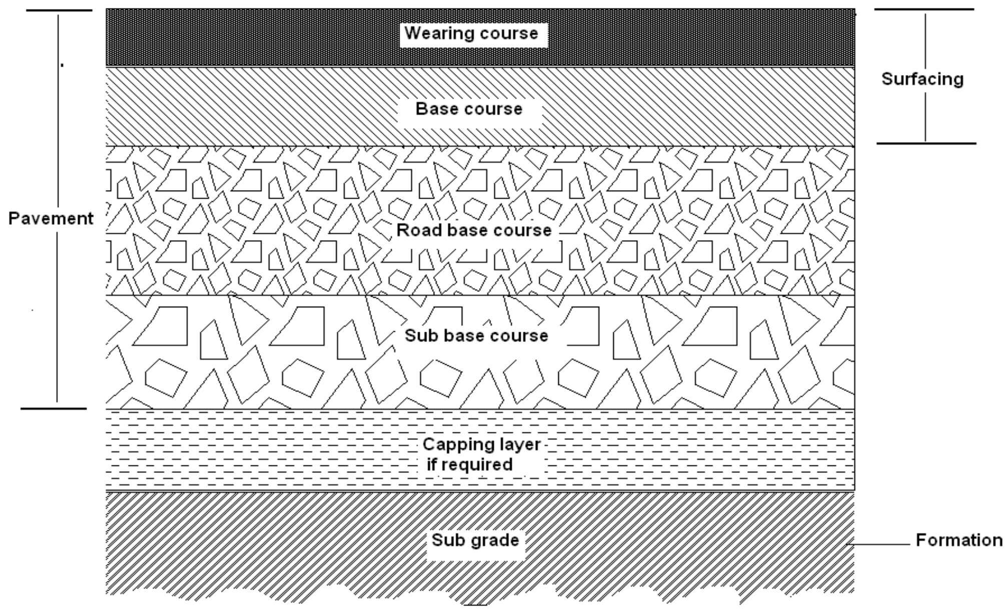


Fig. 3.3 Typical flexible pavement construction layers (1991)

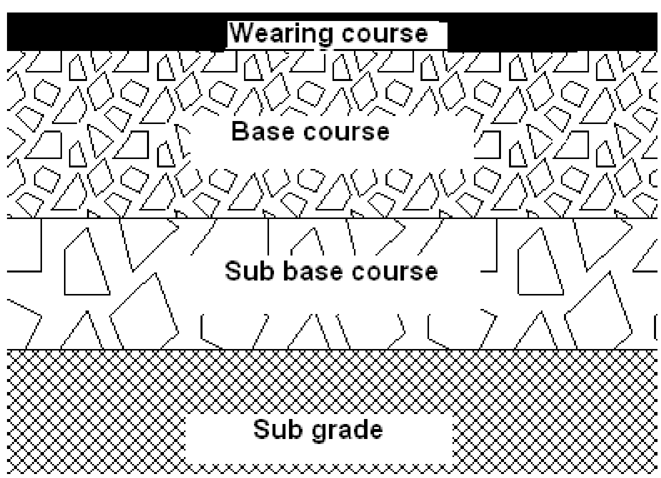


Fig. 3.4 Typical flexible pavement structure used in Nepal

3.3.2 Sub–base

Usually it consists of granular materials, either naturally occurring or crushed but they may be granular materials stabilized with cement. Materials used in sub base are gravel, sands, silty and clayey sands. Grading of the crushed rock aggregates is given in Table 3.1. Sub base performs three functions:

- It is structural layer which will accept greater compressive stresses than the sub grade.
- It provides a working platform on which the paving materials can be transported, laid and compacted.
- It acts as an insulating layer against freezing where the sub grade is the material likely to be weakened by the action of frost.

3.3.3 Road base

Road base is the main structural component of the road. Its function is to reduce the compressive stresses in the sub grade and the sub base to an acceptable level and to ensure that the magnitude of the flexural stresses in the surfacing will not lead to cracking. It includes graded crushed stone premixed with water. Dry single size stone free from clay, organic or other deleterious matter is spread and then filled with fines using vibratory compaction. Cemented materials such as lean concrete and cement bound granular bases or bituminous granular materials using bitumen or tar as a binder. Typical grading of the samples for road base is given in Table 3.1.

Table 3.1: Grading envelope for graded crushed stone according to standard specification for road and bridge work DOR (2001)

Sieve size(mm)	Percentage passing by weight		
	Base	Sub base	
		*SB_1	*SB_2
63.0	–	100	100
40.0	100	75–100	85–100
31.5	85–100	42–75	75–95
20.0	62–92	25–60	60–87
10.0	40–70	15–45	50–80
4.75	26–55	12–37	12–32
2.36	21–53	6–25	7–21
0.60	12–28	5–21	6–17
0.075	2–10	3–12	3–10

*$SB_1$and SB_2 is classes of sub base according to DOR (2001)

3.3.4 Surfacing

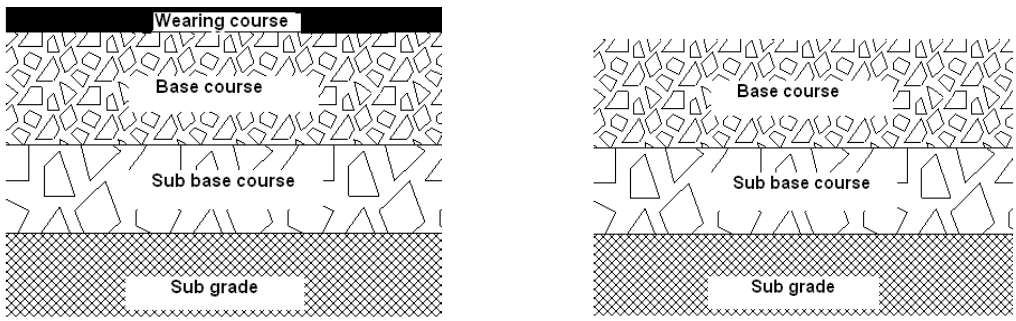
Surfacing includes a wide range of dense and more open and textured material bound with bitumen or tar binders.

Table 3.2: Grading envelop of wearing course materials

Seive size (mm)	Percentage passing by weight	
	Class 1	Class 2
37.5	–	100
25.0	100	85–100
20.0	95–100	85–100
14.0	80–100	65–100
10.0	65–100	55–100
4.75	45–85	35–92
2.00	30–68	23–77
1.00	25–56	18–62
0.425	18–44	14–50
0.075	12–32	10–40

Though these layers are the distinct constructive layers in the pavements; structurally pavements are divided in two groups on the basis of presence or absence of them. Pavements are of two types; bound and unbound.

In bound pavements aggregates are bound by cementations or bituminous binders. Unbound are used for sub bases or capping, but else where may be used for bases or in the case of low volume roads the whole structure is considered as unbound pavements. An unbound pavement is illustrated in Fig. 3.5.



(a) (b)

Fig. 3.5 Pavements structure a) Bound pavement, and b) Unbound pavement

3.4 PAVEMENT MATERIALS

Materials used in unbound layers in the pavements construction fulfill the following functions.

- A structural layer for load spreading and resistance to rutting
- A replacement for frost susceptible sub grade
- A drainage layer

According to Khanna and Jasto (2004), pavement materials can be classified in to:

3.4.1 Mineral materials

- Naturally occurring, semi processed or fully processed sub grade soil, sand (fine aggregates), stone chips, gravel (course aggregates), screened materials etc.

3.4.2 Binding materials

Binding materials can be of following types:

- Stone dust or cohesive soil that forms semi–rigid and semi–flexible bond between the minerals particles.
- Cement, lime and other inorganic binding materials
- Bitumen, tar and organic binding materials

3.4.3 Other materials

- Reinforcement, timber stones, bricks, boulders, cobbles and gabion wires.

Aggregate refers to granular mineral particles that are widely used for highway bases; sub bases and backfill. Aggregates used in roads can have following sources.

- Natural deposits of sands and gravels.
- Pulverized concrete and asphalt pavements
- Crushed stone
- Blast furnace slag

Aggregates form the major part of the pavement structure and it is the prime materials used in pavement construction. Aggregates used in contest of Nepal are natural deposits of sand and gravel and crushed stones. Aggregates used in road construction have to primarily bear load stresses occurring on the roads and runways and have to resists wear due to abrasive action of traffic. The performance of aggregate layer in unbound pavement depends on its intrinsic properties, i.e. the particle shape and grading, its petrography and, the method of placing.

Therefore, aggregates used in unbound pavements construction should have following desirable properties according to DOR (2001) as in Table 3.3.

Table 3.3: Physical requirements of graded crushed stone according to standard specification of road and bridge work (2001)

Tests	Base		Sub base
	*B	*C	
Los Angeles Abrasion Max. %	30	35	40
Aggregate Impact Value Max. %	20	25	30
Sodium Soundness Value Max. %	12	12	12
Flakiness Index %	25	25	30
California Bearing Ratio Min.%	80	80	60
California Bearing Ratio Max. %	100	100	30
Plasticity Index Max. %	NP	NP	6

*B and C are classes of materials

Though, the roads are classified on the basis of traffic flow, location/function and tonnage. In contest of Nepal two types of road classification do exists, classification by traffic flow and classification by service. Among these, classification by traffic is technical (Annex, Table 1) while classification by service is drawn up according to mobility interest.

Pavement structure differs in different country on the basis of national economy, defense, integrity and sovereignty. In Nepal pavement structure generally lacks layers such as capping layers and road base. Capping layer is only essential when sub-grade does not meet the requirement and road base is equivalent to base course in typical pavement profile.

Materials used in the road pavements should follow the specific standards so that qualitative materials used make the road durable.

CHAPTER 4

GEOMORPHOLOGY AND GEOLOGY

4.1 GEOMORPHOLOGY

The study area Adeshwar–Kallabari ridge with highest altitude 1606 m extending SE–NW meets ultimately the Nagarjun Range in the north. It has somewhat rugged topography with some flat lands at the foothills of the ridge. The major draining channels are the Jughe Khola in the north and the Lupan Khola in the south of the ridge. The Jughe Khola flows down from the NW direction towards SE with its many small tributaries coming from north and south. The Lupan Khola flows from the southern part of the ridge with its tributaries from the north and south. These both draining channel ultimately meets the Bishnumati Nadi at Shobhabhagabati and Kalimati respectively. Both the rivers are the straight channel that flows downhill with the channel width varying from 0.5 to 3 m.

The ridge has steep slopes with an angle of 65–85^0 in the north and south while slopes are gentle at the ridge. In the north, steep escarpment of the quarry exists which is covered with the thin residual soil. Rocks exposed at the quarry area are fresh while those exposed at different places around the quarry are weathered. At the top of the ridge the rocks are weathered and only core stones are developed. Rocks at different places of the ridge are being extracted at small scale.

Northern steep slope are covered with small trees and bushes where as southern slopes are barren in some places, covered with thick pine forests at some places. Gentle sloping lands at the top of the ridge are used for cultivation. Contour terracing has been used for the crops.

4.2 GEOLOGY

The geology of the Katmandu Valley can be described by considering the basement and the valley (basin) filling sediments. The major surface of the valley floor is covered by the soft sediment reaching thickness of several hundreds meters. Although the basement rocks in some localities within the valley floor protrude above the soft sediment, the basement rocks are being best exposed in the surrounding mountains.

Study area lies in the western watershed of Kathmandu Valley. Rocks of Kathmandu Complex are distributed in the area. The Kathmandu Complex has been divided in to the Precambrian Bhimphedi Group (Nadgir et al. 1968–73), consisting of relatively high grade rocks and the Phulchouki Group of unmetamorphic or weakly metamorphosed rocks containing fossils of early middle Paleozoic periods (Stocklin 1980). The two groups are possibly separated by a slight unconformity. The study area is confined within the Chandragiri Formation which is underlain by Sopyang and overlain by Chitlang Formation.

In the study area, both basement rocks and valley filling sediments are widely distributed as in Fig.4.1. The study area includes Lukundol Formation, Kalimati Formation, Tistung Formation, Sopyang Formation and Chandragiri Formation. Valley filling sediments i.e. sediments of Kalimati and Lukundol Formations are distributed in the southern part of study area. Lukundol Formation is distributed along the tributaries of Manamati Kholsi. The succession is composed of alternating beds of pebbly conglomerate, medium to coarse grained micaceous sands and black to dark grey clays with occasional lignite layers. The succession is about 10–20 m thick and is covered by 2–3 m thick layer of red soils at the top. Kalimati clays are distributed in the southern most part of the study area i.e. in lower part of Sitapila VDC. It is dominated by grey, dark brown to black clays or silts very rich in organic detritus.

Basement rocks the Tistung Formation was found in the central part of the study area in Harisiddhi, Dhansar, Marthagau and Ranipati. This formation comprises of fine clastic sequence of calcareous sandstones, siltstones, phyllites and slates. Very fine biotite is seen in the rocks of the lower parts of the formation, but sericite and chlorite are the only metamorphic minerals in the main parts and in the overlying units. Distinct color banding appears and intense purple weathering color is characteristics of Tistung Formation. Structures such as minor folds and ripple marks are commonly observed. The dip direction of the rocks is almost towards north and dip amount varies from 15^0 to 75^0.

Since, the study is confined with in the Chandragiri Formation. It is widely distributed in the northern part. Limestone is dominant in northern slope of Adeshwar–Kallabari ridge, Halchok danda, Nayagau and Nagarjun danda. The Chandragiri Formations comprises of light grey to crystalline white, medium to coarse grained, slight to moderately weathered, medium to thick bedded, massive and finely crystalline, highly jointed giving yellow to brown weathering color. The

Fig. 4.1 General geological map of the study area

dip directions of the rocks are almost towards the north and dip amount varies from 40^0 to 86^0. The mining of the rocks are being done along the right bank of the Jughe Khola. The lower contact between Chandragiri and Sopyang Formation was found at the right bank of the Jughe Khola and upper boundary between them lies near the Nagarjun Fence. Sopyang Formation is distributed in Ichangu and Narayanthan. It consists of dark, thin bedded calcareous slate, yellowish brown when weathered.

4.3 STRUCTURES

Within the study area, major structures pertaining regional geological significance and several minor structures having local geological significance were identified. Structurally the study area lies in the northern limb of the Thankot anticline and southern limb of the Nagarjun Syncline according to (Acharya and Dhital 2006). Except these anticline and syncline other major structure in the study area is a fault that exists between different Formation such as between Tistung and Chandragiri, Chandragiri and Sopyang etc. Small scale structures also bear pertinent significance since they can provide necessary information in portraying the general evolution of the area as a whole. The geological structures present in the study area are parallel lamination, cross-bedding, ripple marks, bedding planes, joints and folds.

Major distinguishing features of the bedding planes parallel lamination are dominant in the limestone beds of the study area. Laminas in the limestone beds are thin and continuous as in Fig.4.2a. Cross beddings were dominant in the calcareous sandstone beds of the Tistung Formation in the southern part of the quarry site.

Ripple marks are prominent features of the sandstones beds of the Tistung Formation. Bedding planes orients SE–NW in the quarry sites. Bedding planes are thin to thick bedded. General trend of the bedding planes varies from $86^0/76^0$N to $114/57^0$S.

(a)

(b)

(c)

Fig. 4.2 Minor structures in the study area, a) parallel lamination, b) Joint sets, and c) Fold

Joints are the major features of the quarry area. Two sets of joints are prominent. 1^{st} sets are perpendicular to the bedding planes i.e. vertical joints while others are randomly oriented as in the Fig.4.2b. Small scale folds are dominant in the sandstones beds of Tistung Formation of the study area. Small scale drag fold in sandstone beds of Tistung Formation is shown in Fig. 4.2c.

4.4 LITHOLOGY

The columnar section prepared at the quarry site shows that limestone is a dominant lithology with thin overburden of residual soil as in Fig. 4.3. Boundary between the Tistung and Chandragiri Formation at the southern most part of the columnar section shows highly weathered, coarse grained calcareous sandstone while the Chandragiri Formations at the quarry comprises of light grey to crystalline white, medium to coarse grained, slight to moderately weathered, medium to thick bedded, massive and finely crystalline, highly jointed giving yellow to brown weathering color. The dip directions of the rocks are almost towards the north and dip amount varies from 40^0 to 86^0. Total thickness of limestone bed is about 115 m.

4.5 ROCK MASS CHARACTERISTICS

It is universal that large volume of rocks are involved in the design and construction of structures and excavations in rocks such as dams, tunnels, underground power plants, road cuts and open pit mines. The stability and deformability of which is dependent on the strength and deformability of the rock masses. The most universally occurring anisotropic characteristics of all rock masses are the presence of distinct breaks or discontinuities in the physical continuity of the rocks. These include bedding surfaces, joints, faults and metamorphic foliation.

Fig. 4.3 Detailed columnar section from the outcrop on the road from Adeshwar–Kallabari to the Right bank of the Jughe Khola

Apart from the reduction in the strength from pervasive chemical weathering, the presence of discontinuities in a rock mass is the primary controlling factor of mass strength and deformability.

Thus, the evaluation of the engineering properties of rock mass includes knowledge of the intact rock properties, the occurrence and nature of discontinuities and the degree and extent of chemical weathering.

4.5.1 Discontinuities in rock masses

Bedding surfaces, joints, faults and well developed metamorphic foliation are the major discontinuities in a rock masses. Orientation, spacing, continuity, surface characteristics, the separation of the discontinuity surfaces, and the accompanying thickness and the nature of filling materials if present are the most consistently measured joint properties.

4.5.1.1 Orientation

The most readily apparent influence of the orientation of discontinuities of rock mass strength is evident in the failure of rock slopes along one or more discontinuities. The orientation of discontinuities is given in Table 4.2. The data in the table shows that bedding dip towards north with dip amount 79^0. Two sets of joints are prominent in the area but other sets are rare in the area.

4.5.1.2 Spacing

The spacing of discontinuities affects overall rock mass strength or quality. Even strongest intact rock is reduced to one of little strength when closely spaced joints are encountered. Conversely where the spacing is great, the behavior of the rock mass will be strongly influenced by the intact rock properties. Discontinuity spacing and intact rock strength have been used by Franklin et al (1971) to suggest excavation method for rock mass. Orientation and frequency of joints combine to influence the response of a rock mass to construction.

Table 4.1: Spacing, width and persistency of joints (Johnson and DeGraff, 1998)

Spacing	<5cm (L)	5–30cm (M)	0.3–1m (H)	1–3m (VH)	>3m (EH)
Width	<0.1mm (L)	0.1–1mm (M)	1–5mm (H)	5–20mm (VH)	>20mm (EH)
Persistency	Non–continuous	Few–continuous	Continuous no infill	Continuous thin infill	Continuous thick infill

L–low, M–medium high, VH–very high, EH–extremely high

The spacing of the discontinuity data obtained from the rocks of the study area shows the spacing ranges from 0.3 m to 3 m. This means that the rock in the study area falls within high to very high classes respectively.

4.5.1.3 Continuity or persistency

Length or continuity is measured on an exposed rock surface. The average measured continuity, persistence or size of joints in a given set may not be representative of actual conditions because of limited surface exposures. Except where joint is seen to terminate in the rock mass, all lengths measured are minimum lengths. With the exception of very continuous, through going discontinuities, some of the strength of the intact rock is transferred to the rock mass through the intervening rock or rock bridges.

In the field, persistency of the discontinuities was clearly observed and the tabulated in Table 4.1. Most of them are continuous and few of them contain thin to thick fillings. Continuity of discontinuities defines low stability of rock mass that means it is easy to excavate.

4.5.1.4 Surface characteristics

Three factors are involved when the surface characteristics of discontinuities are considered are:

- The waviness or undulation of the surface which results in variations in orientation or attitude along a given discontinuity.
- The smaller scale roughness of the surface, which provides friction between two adjacent blocks, and
- The physical properties of any materials that may fill the space between two bounding surfaces of the discontinuity.

Surface characteristics of rock mass were identified by compared with waviness and roughness profile (after Barton and Choubey 1977).

4.5.1.5 Separation and filling

The amount of separation or space between joint surfaces and the presence of filling materials may have a profound influence on the strength of a jointed rock mass. Combined with roughness they constitute the character of the joint. The separation of joint walls may result from the tensile stresses that created the joints, solution widening of joints or shearing movements that can separate the surfaces by generating gouge materials or by movement along wavy surfaces.

The joint separation ranges from 0.1 mm to >20 mm. The spaces may be empty, partially filled or completely filled with silty clay and vegetation.

4.5.2 Weathering grade and induration of rock

The weathering state of rock has a significant influence on the engineering properties of a rock mass. Physical weathering results in changes in size and number of discontinuities present in the rock mass. Chemical weathering of the rock mass is enhanced by the movement of groundwater through the networks of discontinuities present in the rock masses. Gradation from fresh to decomposed rock along the discontinuities will be relatively perpendicular to the orientation of the discontinuity and may bear no relation to the surface topography.

In the field, the weathering grade of rocks is classified according to weathering classification given by Geological Society London (1777) (Annex Table 2). The result of the field observation recorded in the table 4.3 shows that most of the rocks on the outer surface fall in grade II and III while inner excavating fall on grade IB. Therefore, the rock in study area is faintly weathered to moderately weathered.

Induration is the hardness of the rock and it is the indication of degree of freshness or degree of stiffness in terms of pressure or the introduction of cementing materials. The induration of intact rock specimen can be measured by means of scratch made by knife or a nail. Induration of the limestone ranges from H_3 to H_4 i.e. indurated to strongly indurated (Annex Table 3).

Table 4.2: Rock mass characteristics of Adeshwar area

Sample no.	Weathering grade	Seepage	Discontinuities	Dip direction/amount	Persistency	Spacing(cm)	Width(mm)	Roughness	Infilling materials
BR1	II	no	Bedding	207/79^0	Continuous	0.2–1m	0.1–1	8–10	Silty clay
			Joint 1	Vertical	Contd. no infill	0.5–1.3m	1–5	6–8	silty clay+veg
			Joint 2	100/83^0	Non. continuous	0.3–1m	0.1–1	10–12	silty clay+veg
BR2	II–III	no	Bedding	207/79^0	Continuous	0.3–1m	0.1–1	8–10	Silty clay
			Joint 1	115/83^0	Contd. no infill	5–30	1–5	10–12	silty clay+veg
			Joint 2	60/39^0	Non. continuous	1–3m	0.1–1	6–8	Silty clay
BR3	II	no	Bedding	357/76^0	Continuous	0.1–1m	<0.1	6–8	clay
			Joint 1	vertical	Contd. thin infill	5–30	5–20	8–10	silty clay+veg
BR4	IB	no	Bedding	13/56^0	Continuous	0.3–1m	0.1–1	6–8	silty clay+veg
			Joint 1	171/25^0	Contd. thick infill	1–3m	>20	10–12	silty clay+veg
			Joint 2	vertical	Non. continuous	5–30	0.1–1	8–10	clay
			Joint 3	280/83^0	Non. continuous	0.3–1m	<0.1	10–12	No infill
BR5	II	no	Bedding	207/79^0	Contd. no infill	0.3–1m	0.1–1	8–10	clay
			Joint 1	Vertical	Continuous	1–3m	1–5	10–12	silty clay+veg
BR6	II–III	no	Bedding	22/75^0	Continuous	0.1–3m	<0.1	6–8	No infill
			Joint 1	120/87^0	Contd. thick infill	1–3m	>20	8–10	silty clay+veg
BR7	IB	no	Bedding	340/59^0	Continuous	1–3m	0.1–1	10–12	clay
			Joint 1	vertical	Continuous	0.3–1m	1–5	8–10	silty clay+veg
			Joint 2	115/83^0	Non. continuous	1–3m	<0.1	6–8	No infill

Table 4.3: Rock mass condition of study area

Rock mass characteristics	Result
1. Discontinuities characteristics	
Orientation	NW–SE
Spacing	0.10mm –5m
Persistency	few continuous
Roughness of surface	6–8 to 12–14
Aperture	0.1mm–10mm
Infilling materials	silty clay and vegetation
2. Weathering grade	IB–III(faintly to moderately weathered)
3. Degree of induration	H_3–H_4(indurated to strongly indurated)
4. Colour	Dark grey to light grey

CHAPTER 5

TESTING OF CRUSHED – ROCK AGGREGATES

5.1 INTRODUCTION

Aggregate forms the major part of pavement structure and it is the prime material used in pavement construction. Aggregate has to bear load stresses occurring on the roads and runways and have to resist wear due to action of traffic. Aggregate is used in construction of pavements using cement concrete, bitumen and in water bound macadam. Thus, the properties of the aggregate are of considerable significance for road construction. The desirable properties of the aggregate can be distinguished separately as petrographic properties, physical properties, mechanical properties and chemical properties. Minimum testing frequency for a process control according to DOR (2001) is as in Annex Table 4.

5.2 PETROGRAPHIC PROPERTIES

Petrographic properties of the rocks can be categorized in to following two classes: microscopic and megascopic under the microscope and surface condition respectively.

5.2.1 Microscopic properties

Various rock samples collected in field were used in petrographic study. The thin sections of the different samples were prepared in the

Petrography Laboratory of the Central Department of Geology, Tribhuvan University, Kirtipur.

Altogether 20 representative samples were taken to prepare thin section. Among them thirteen samples were from stratified sequence and seven from bulk sampling points as in Fig. 5.1. Altogether 33 thin section were prepared 13 sections were colored so that pore spaces in the rocks can be studied. Thirteen stratified samples were prepared to study the lateral variation in the texture and composition of the rocks, and other seven thin sections were prepared from bulk sample area. Accept colored all other thin sections were stained for separating calcite and dolomite by Dickson method for staining carbonates. Massive purple to royal blue color on staining confirms that all the sections contain ferron calcite. No colored spaces in the thin sections except in some fracture shows that limestones have exceptional pore spaces and low porosity.

Generally in most of the thin sections the visual fields are covered by calcite and quartz grains with few grains of feldspar, mica and heavy minerals such as pyrite. Detailed description of thin sections of bulk samples is given as below in Table 5.1.

5.2.1.1 Analysis of composition

Compositional analysis of the thin section by using the petrographic microscope and using a scion image analyzer on the basis of percentage composition of calcite and siliciclastic sediments shows that in most of the section visual fields

Fig. 5.1 Sampling points of an aggregate in study area

are covered by calcite and quartz grains with few grains of feldspar, mica and heavy minerals such as pyrite. Percentage composition of the samples is given in Table 5.2.

Table 5.1: Detailed description of thin section of the samples

Sample No.	Description
BR1	Calcite dominant. Quartz grains and calcite shows uniformity of grain size and loss of original texture. Calcite grains show preferred linear arrangement. Grains are generally anhedral. Matrix supported i.e. quartz grains supported by calcite. Opaque mineral pyrite present. Mica shows linear arrangement. Only few pore–spaces present. Contact between quartz and calcite is sutured (Fig. 5.a, b).
BR2	Calcite dominant. Quartz grains are heterogeneously distributed and occur in cluster forms. Grains are of anhedral shape. Heavy mineral pyrite and other siliciclastic sediments such as feldspar and mica present in rare quantities (Fig 5.2c).
BR3	Calcite dominant. Uniformity of grain size and loss of original texture. Quartz grains are recrystallized and heterogeneously distributed. Grain shows preferred linear arrangement (Fig. 5.2d, e).
BR4	Analysis shows almost equal proportion of calcite and siliciclastic sediments. Sizes of the grains are comparatively equal to that of silt 0.002–0.060 mm. No linear structure is seen except of some mica. Grains show concavo–convex contact (Fig. 5.2f).
BR5	Siliciclastic sediments dominant. Quartz heterogeneously distributed and occurs in cluster. Under PPL quartz grains are polycrystalline. Grains are anhedral in shape. Opaque mineral pyrite is present (Fig. 5.2g).
BR6	Calcite dominant. Quartz grains are homogenously distributed and show preferred linear orientation. Some laminae contain higher percentage of quartz while others are dominated by calcite. Grains are anhedral in shape (Fig. 5.2h).
BR7	Calcite dominant. Quartz grains are heterogeneously distributed. Grain size ranges to the size of silt and grains are anhedral to sub–hedral. Contact between quartz and calcite grain is concavo convex. Beside calcite and quartz other minerals such as pyrite, feldspar and mica rarely present (Fig. 5.2h).

Table 5.2: Compositional analysis of thin section

Sample no.	% composition of $caco_3$	% composition of siliciclastic sediments				Classification
		Quartz	Feldspar	Mica	Heavy minerals	
BR1	78	14	2	5	1	Crystalline limestone
BR2	55	40	2	2	1	Siliceous limestone
BR3	85	10	1	3	2	Crystalline limestone
BR4	40	48	5	3	4	Calcareous siltstone
BR5	50	45	2	3	1	Calcareous siltstone
BR6	55	40	3	2	1	Siliceous limestone
BR7	52	45	2	1	1	Siliceous limestone

On the basis of percentage chemical composition (Table 5.2) and grain size parameters (Table 5.3) according to Clark and Walker (1977) limestone can be classified in to three sub groups.

- Crystalline limestone
- Siliceous limestone
- Calcareous siltstone or sandstones

Crystalline limestone contains more than 50% carbonate content, uniformity of grain size and loss of original texture (Fig. 5.3a). These are highly indurated and have unconfined compressive strength high. Sample BR1 and BR3 are crystalline varieties of limestone. These limestone's are thick bedded, light grey, coarse crystalline with well developed parallel lamination in field condition.

Siliceous limestone contains 50–70% total carbonate content and grain size is equal to that of the sand (Fig. 5.3b). These are moderately indurated and have unconfined compressive strength moderately strong to strong. Sample BR2, BR6 and BR7 are siliceous varieties of limestone. These limestones are thick to thin bedded, grey, medium grained limestone.

Calcareous siltstone contains 10–50% total carbonate content and grain size ranges from silt to that of the sand (Fig. 5.3c). These are moderately indurated and have unconfined compressive strength moderately strong to strong. Sample BR4 and BR5 are calcareous siltstone. But in the field these limestone are dark grey, fine grained, argillaceous, cross laminated.

Most thin section shows heterogeneous distribution of quartz grains interlocked with calcite. Quartz grains show polycrystalline nature under the plane polarized light. Polycrystalline nature of quartz and sutured contact also shows that quartz grains are partially recrystallized (Fig. 5.3d).

Detailed textures, structures, shape and size and classification on the basis of engineering point of view after Clark and Walker (1977), is given in Table 5.3.

Table 5.3: Textural analysis of thin sections

Sample No.	Size of grains /crystal	Shape of grains/crystal	Induration	Microstructures	
				Homogenity or heterogenety	Microfabric
BR1	0.032 –0.132	Subhedral	Highly indurated	Homogenous	Microlaminae
BR2	0.075–0.202	Anhedral	Moderately indurated	Heterogenous	Massive/ polycrystalline
BR3	0.082–0.130	Elongated and // to lamination	Highly indurated	Homogenous	Microlaminae
BR4	0.025–0.05	Anhedral	Moderately indurated	Heterogenous	Massive/polycrystalline
BR5	0.067–0.085	Anhedral	Moderately indurated	Heterogenous	Massive/polycrystalline
BR6	0.13–0.205	Anhedral to subhedral	Moderately indurated	Homogenous	Microlaminae
BR7	0.082–0.105	Anhedral	Moderately indurated	Heterogenous	Microlaminae

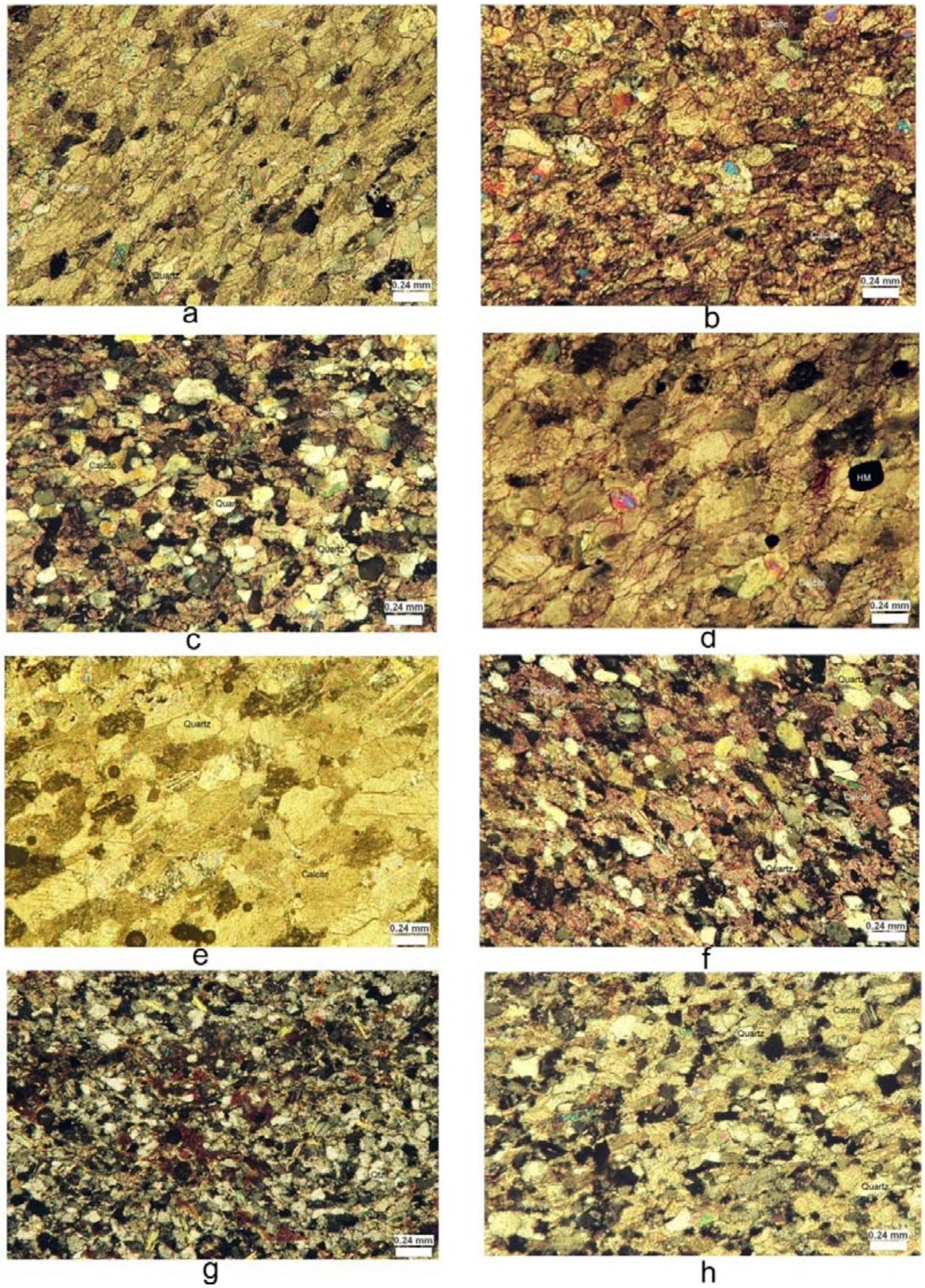


Fig. 5.2 Photomicrograph of limestone of various compositional and textural variation

Contd.

(a, b) Calcite dominant. Quartz grains and calcite shows uniformity of grain size and loss of original texture. Calcite grains show preferred linear arrangement and are generally anhedral. Mica shows linear arrangement. Only few pore–spaces present. Here b is stained thin section in which most of the part is stained which is calcite and blue color shows that the pore spaces are only few. Other minerals such as feldspar, mica also present.

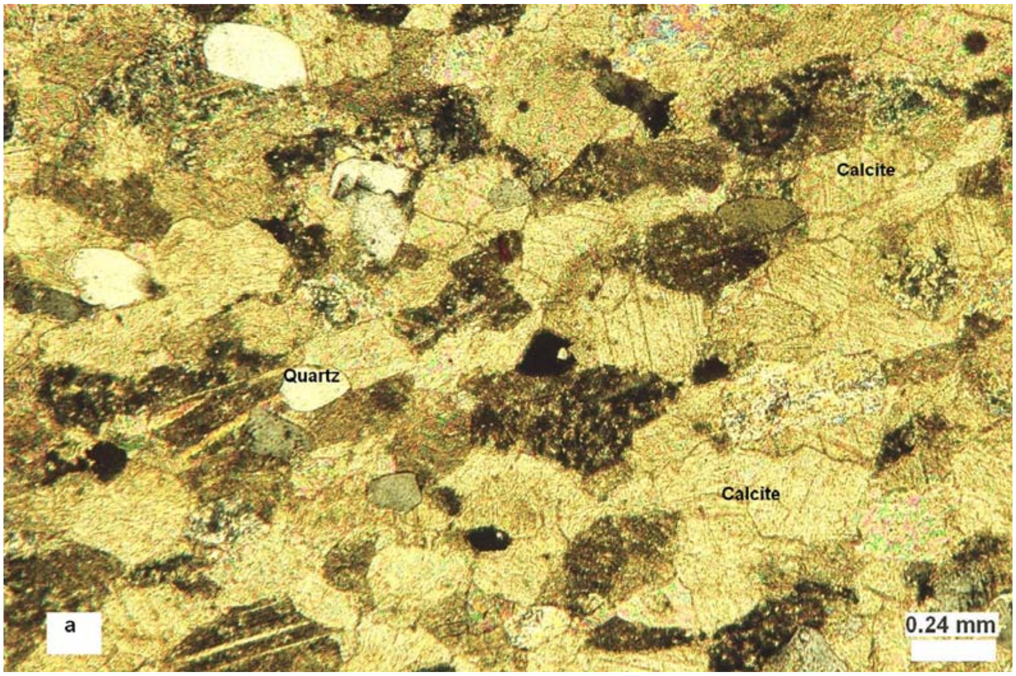
(c) Calcite dominant. Quartz grains are heterogeneously distributed and occur in cluster forms. Grains are of anhedral in shape and grain size ranges from that of silt (0.002–0.060). Quartz show polycrystalline nature and partially recrystallized.

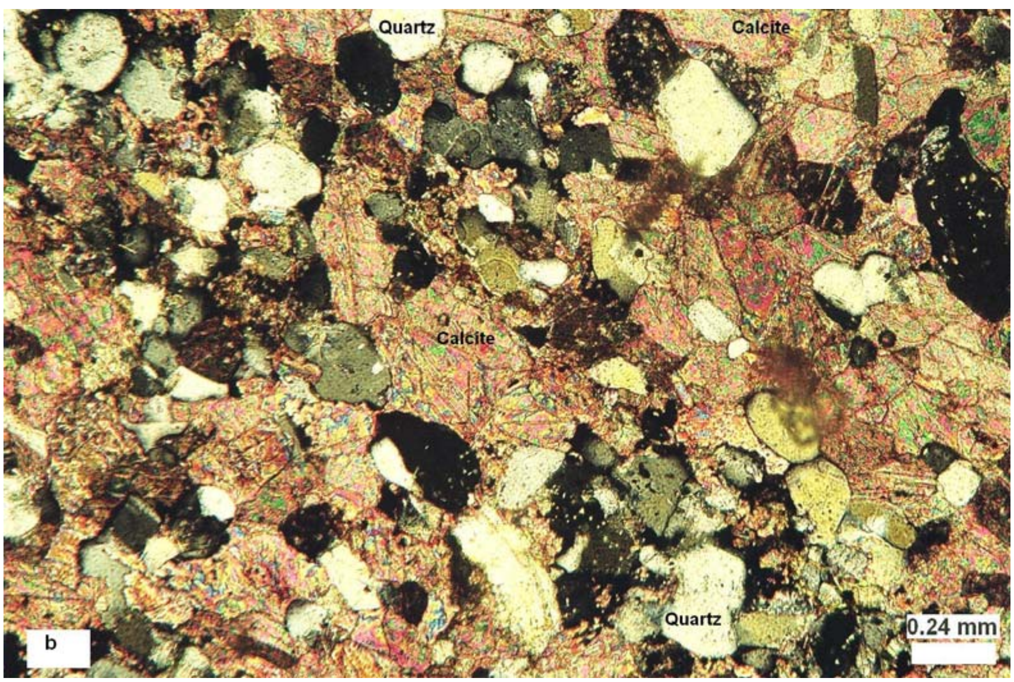
(d, e) Calcite dominant. Uniformity of grain size and loss of original texture. Quartz grains are recrystallized and heterogeneously distributed. Heavy mineral pyrite distinguished in stained thin section, Fig. d.

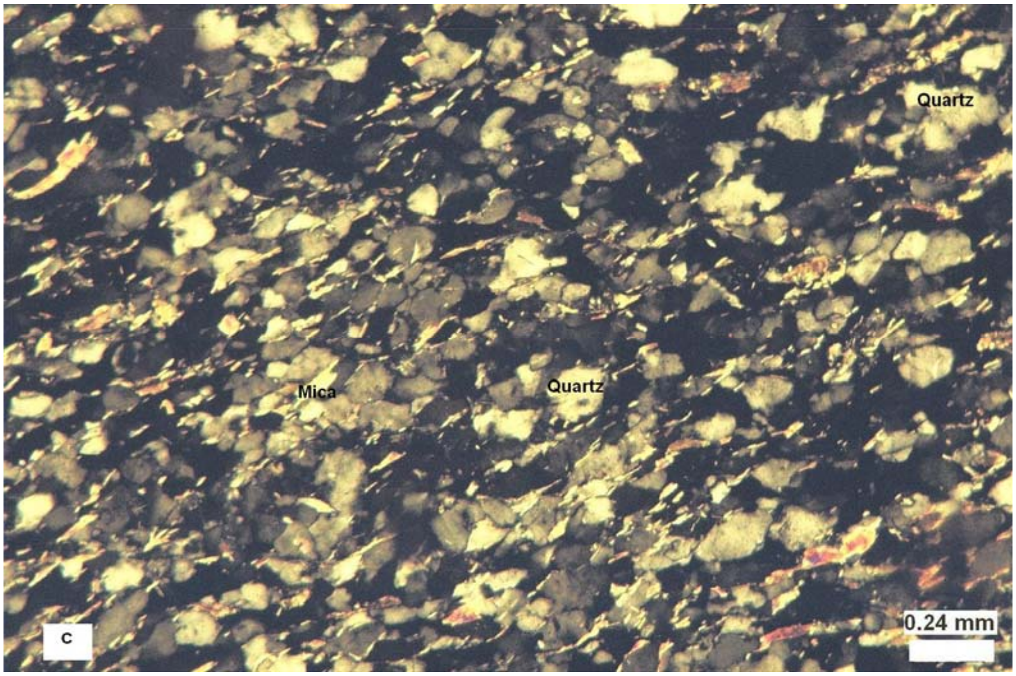
(f) Siliciclastic sediments dominant. Quartz heterogeneously distributed and occurs in cluster. Some lamina contains high percentage of quartz. Quartz grains are polycrystalline in nature and anhedral in shape. Size ranges from silt to sand (0.002–2.0 mm)

(g) Siliciclastic sediments dominant. Quartz heterogeneously distributed and occurs in cluster. Some laminae contain higher percentage of quartz while others are dominated by calcite.

(h) Calcite dominant. Quartz grains are heterogeneously distributed. Grain size ranges to the size of silt and grains are anhedral to sub–hedral. Contact between quartz and calcite grain is concavo convex.


Calcite
Quartz
Calcite
a
0.24 mm


Quartz
Calcite
Calcite
Quartz
b
0.24 mm



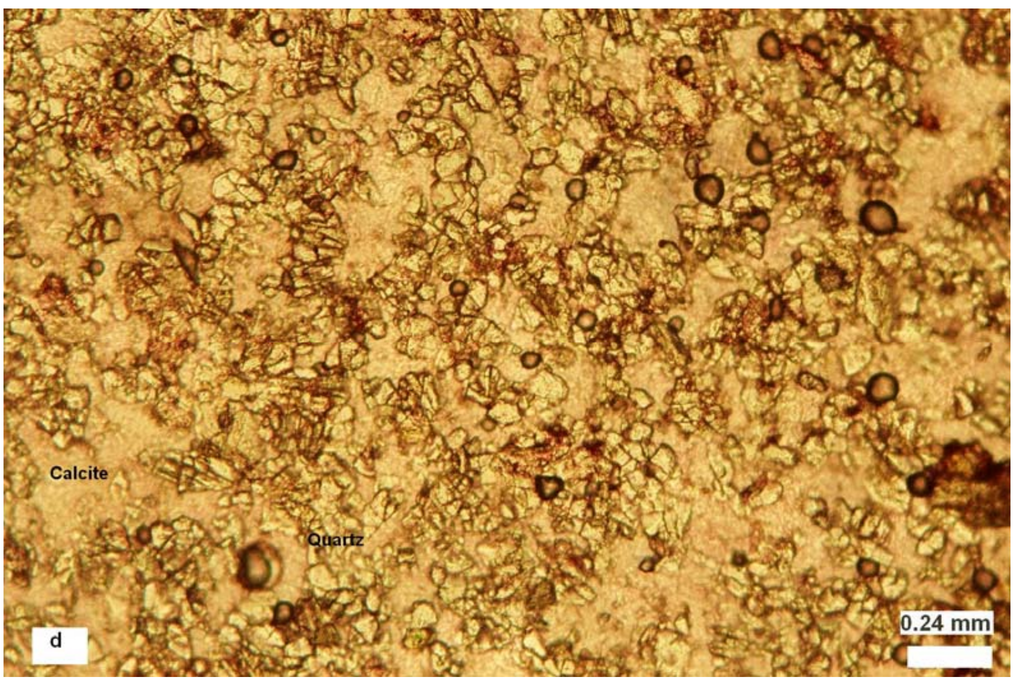


Fig. 5.3 Photomicrographs of different varieties of rocks; a) Crystalline limestone lacking original texture b) Siliceous limestone (stained) showing lamina c) Calcareous siltstone in which interlocked and elongate quartz grains d) Massive and polycrystalline nature of quartz

a b c

Fig. 5.4 Schematic diagram showing the spatial relationship of calcite and quartz grains in limestone, a) Crystalline limestone b) Siliceous limestone and c) Calcareous siltstone

5.2.2 Megascopic properties

5.2.2.1 Grading

Aggregate grading is the percentages of the different size fractions, after sieving, more scientifically determining the particle size distribution of the material. Gradation is the characteristics of a road aggregate on which the greatest stress is placed. So proper grading of an aggregate is important because of its direct influences on both the quality and cost of pavement component.

To obtain the different size fractions for weighing, the sample of aggregates is sieved on the appropriate sieve sizes for the particular material, and the retained aggregates amounts weighed. The total percentage method is very convenient for the graphical representation of a grading and is most widely used in graded specifications (Fig. 5.5). Since, the aggregates to be used are crushed rock aggregates and desired grading can be provided according to the requirements, here only one sample out of seven samples is graphically represented. The particle size distribution of crushed rock aggregate is given in Table 5.4.

Table 5.4: Grain size distribution of crushed rock aggregates

Sieve opening	Weight retained	Cumulative weight retained	Cumulative % retained	% passing
100mm	–	–	–	–
63mm	401	401	25.773	74.227
40mm	469	870	55.915	44.085
30mm	220	1090	70.055	29.945
20mm	105	1195	76.803	23.197
12.7mm	15.49	1210.49	77.799	22.201
9.52mm	39.31	1249.8	80.325	19.675
4.76mm	170.11	1419.91	91.259	8.741
2.00mm	38.34	1458.25	93.723	6.277
0.84mm	30.35	1488.6	95.673	4.327
0.59mm	4.33	1492.93	95.952	4.048
0.42mm	4.2	1497.13	96.222	3.778
0.297mm	6.08	1503.21	96.612	3.388
0.149mm	14	1517.21	97.512	2.488
0.074mm	38.4	1555.61	99.980	0.020
pan	0.31	1555.92	100.000	0.000

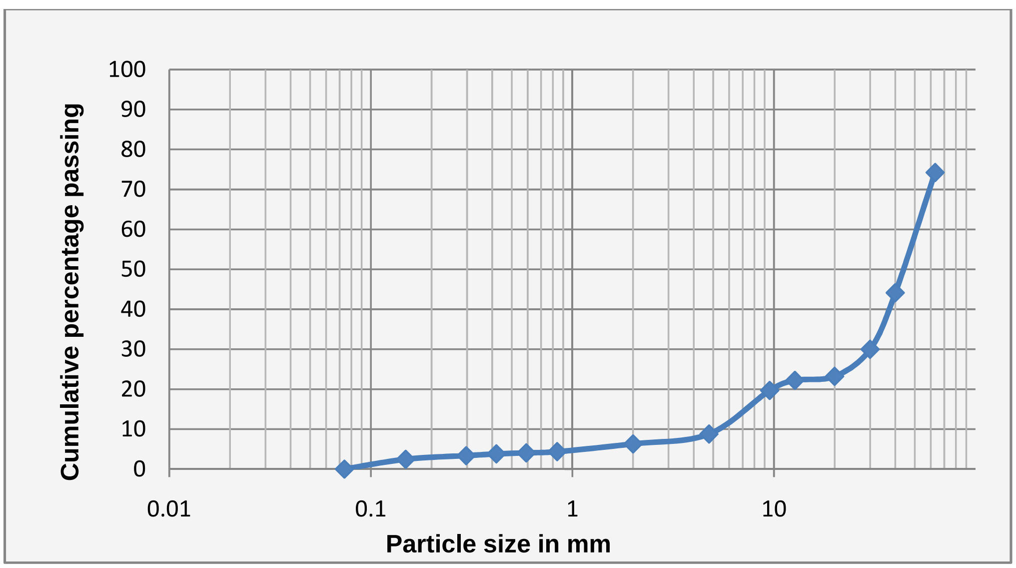


Fig. 5.5 Graph showing particle size distribution of crushed rock aggregates

5.2.2.2 Shape analysis of aggregate

The particle shape of an aggregate is determined by the percentages of flaky and elongated particles contained in it. For base course and construction of bituminous and cement concrete types, the presence of flaky and elongated particles are considered undesirable as they may cause inherent weakness with possibilities of breaking down under heavy loads. Rounded aggregates are preferred in cement concrete road construction as the workability of concrete improves. Angular shapes of particles are suitable for granular base course due to increased stability derived from the better interlocking.

The aggregates were studied for their shape factor (F), sphericity (ψ), roughness index (Ru), roundness index (Rn), flakiness index (FI), and elongation index (EI).

For shape analysis the length (a-axis), breadth (b-axis), perimeter, and area of 40 samples from the photographs taken in the laboratory were measured by the NH scion image analyzer (Fig. 5.6). The thickness (c-axis) was measured using a Verneir Caliper. Based on the longest (a), intermediate (b) and shortest (c) dimension of each grain, the shape was quantified in terms of flatness ratio (p=c/b) and elongation ratio (q=b/a). Shape factor (F) is the ratio of p/q.

The Aschenbrenner working sphericity (ψ) was calculated as

$$\psi = \frac{12.8 \times \sqrt[3]{p^2 q}}{1 + p(1+q) + 6\sqrt{1 + p^2(1+q^2)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5.1)$$

Roughness and roundness indices of Janno (1998) were measured using software NIH Scion image. Roughness index is calculated as

$$\text{Roughness index (Ru)} = \frac{p}{p_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (5.2)$$

where, p the perimeter of aggregate and Pc is is the convex perimeter.

The roundness (Rn) was calculated as

$$\text{Roundness index (Rn)} = \frac{4\pi A}{p^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (5.3)$$

where, A is the area of the clast and P is the perimeter of a clast.

The result of the shape analysis of gravel is shown in Table 5.5. Flatness ratio (p) and elongation ratio (q) ranges from 0.65 to 0.79 and 0.57 to 0.71 respectively. Shape factor ranges from 0.99 to 1.40 indicating their cubic to disc shape. High sphericity (0.82 to 90) and the plot of flatness and elongation ratios indicate that the most of the aggregates grains are cubic shape (Fig. 5.7).

For a smooth material, the roughness index is equal to unity. As roughness increases, the roughness index also increases. This shows that the aggregates are rough and show good workability (Kaplan 1961; Neville 1996).

The roundness index of a perfect circle is 100. As a material become angular, the roundness index decreases (Janno 1998). Rounded aggregates produce significantly higher permanent deformation than the angular ones. Internal frictional angle increases with the increasing angularity and surface roughness (Holtz and Kovacs 1981). Roughness index ranges from 1.28 to 1.40 whereas roundness index of the aggregates here varies from 52.65 to 54.77 which show that aggregates have high roughness index and moderate roundness value.

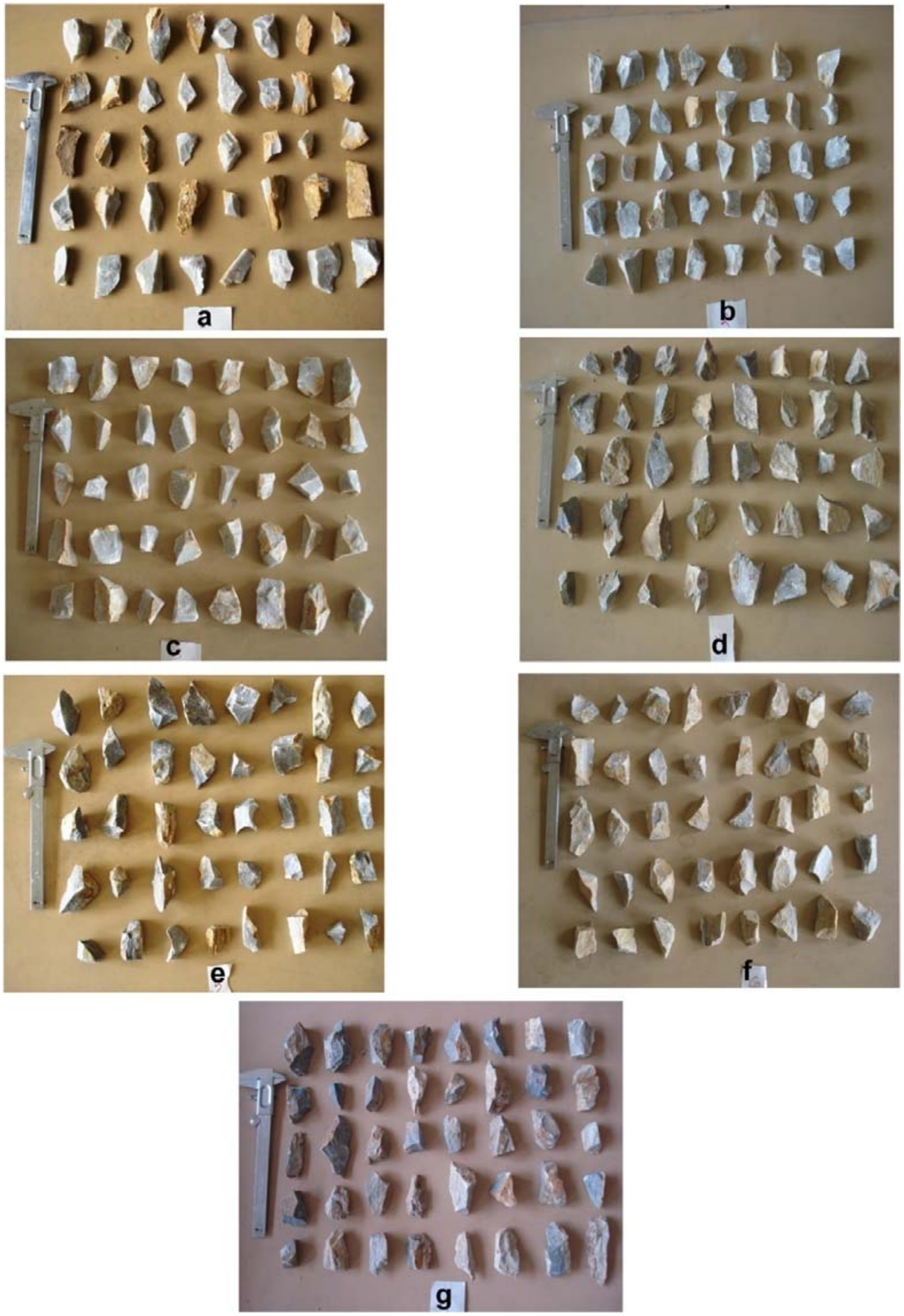


Fig. 5.6 Samples for the determination of shape analysis where, a) Sample BR1 b) Sample BR2 c) Sample BR3 d) Sample BR4 e) Sample BR5 f) Sample BR6 and g) Sample BR7

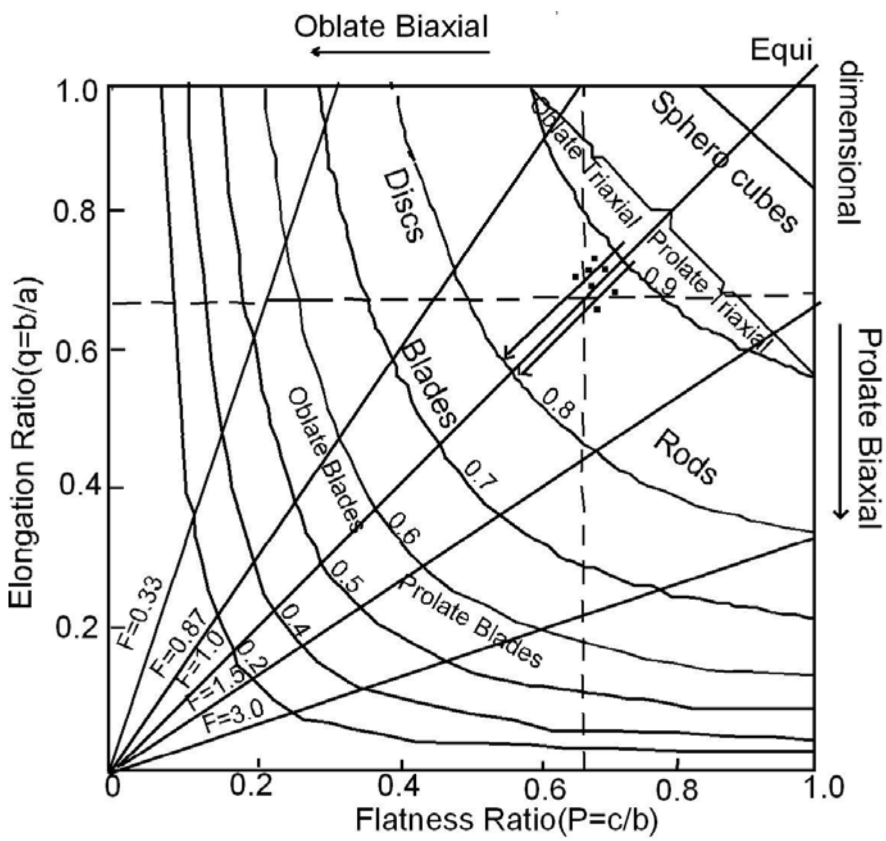


Fig. 5.7 Shape diagram indicating mean forms of aggregate samples

Table 5.5: Results of the shape analysis

Sample no.	Elongation ratio(q)	Flatness ratio (p)	Shape factor (p/q)	Sphericity (ψ)	Roughness (Ru)	Roundness (Rn)
BR1	0.69	0.73	1.1	0.86	1.36	54.47
BR2	0.66	0.77	1.34	0.86	1.28	54.77
BR3	0.7	0.79	1.16	0.9	1.28	53.33
BR4	0.71	0.65	0.99	0.85	1.31	52.81
BR5	0.65	0.69	1.1	0.85	1.32	53.3
BR6	0.7	0.72	1.06	0.87	1.32	53.51
BR7	0.57	0.72	1.4	0.82	1.4	52.65

The flakiness index is the percentage of that mass of aggregate whose least dimension is less than 0.6 times there mean dimension. This test is applicable to the aggregate of size larger than 6.3 mm. During this test aggregate sample was sieved through different sieve sizes, at least 200 pieces of each fraction were taken, weighed and allowed to pass through the selected slot size (Fig.5.8) which has width equal to 0.6 times the mean dimension.

$$\text{Flakiness index} = \frac{W_T}{W} \times 100\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (5.4)$$

where, W_T is the weight of the aggregate passing through the slot and W is the total weight before passing.

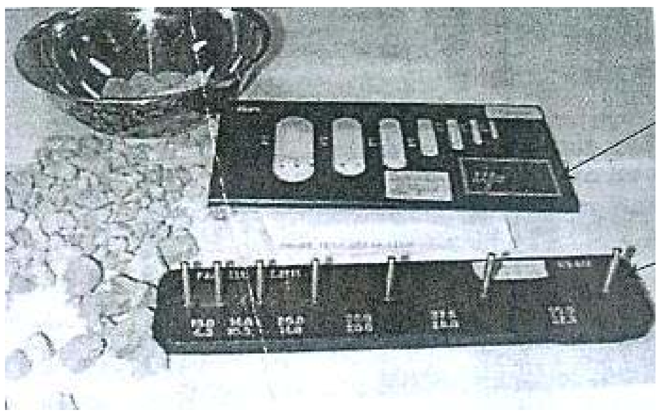


Fig. 5.8 Shape test apparatus for flakiness and elongation index

Elongation index is the percentage by weight of the particles, whose longest dimension is greater than 1.8 times their mean dimension. This test is not applicable to sizes smaller than 6.3 mm and sample size of 63–50 mm is neglected. Same procedure as in flakiness index was used except that length gauge (Fig. 5.8) was used instead of metal slot. Specified length of the length gauge is determined by using formula i.e. 1.8(X+Y)/ 2.

Where,

X=size of the sieve from which aggregate is passing

Y=size at which aggregate is retained

$$\text{Elongation index} = \frac{W_L}{W} \times 100\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (5.5)$$

where, W_L is the weight of aggregate retained in the length gauge and W is the total weight.

The result of the test is given in Table 5.6. Flakiness index of the test samples varies from 14% to 25% which is under the physical requirements of graded crushed stone for base and sub base course NRS (DOR 2001).This concludes that flaky and elongated particles are to be avoided because if flaky and elongated particles are present in appreciable proportions, the strength of the pavement layers would be adversely affected due to possibility of breaking down under loads.

Though the elongated shape of an aggregate also affects the compaction and the construction of a pavement, there are no specified limits of elongation index values as of flakiness index.

Table 5.6: Determination of shape of the test sample

Sample No.	Total weight (W)	Weight passing thickness gauge W_T	Weight retained on length gauge W_L	FI %	EI %
BR1	2277.5	570	1719.5	25.03	75.49
BR2	2108	471.5	1914	22.37	90.79
BR3	1673	369	1272	22.06	76.03
BR4	2205	461.5	1850.5	20.93	83.92
BR5	1951	287.5	1755	14.74	89.95
BR6	1363	22	1181.5	1.61	86.68
BR7	1595	255.5	1397.5	16.02	87.61

5.3 PHYSICAL PROPERTIES

5.3.1 Water absorption value and dry density

The water absorption value is usually accepted as a measure of porosity and sometimes as a measure of resistance to frost action. Particle size influences absorption values, so similar size aggregate is used for this test. Water absorption percentage is the soundness indicator; it was determined by measuring the increase in sample weight owing to the pore water expressed as percentage of dry weight (ISRM 1979). Core samples as in Fig. 5.9 were immersed in water for 24 hours, surface dried, weighed and percentage of absorption relative to the dry weight was calculated.

$$\text{Water absorption} = \frac{X_2 - X_1}{X_1} \times 100\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (5.6)$$

Where, X1 is the weight of oven dried sample and X_2 is the weight of wet sample after soaking.

Dry density test is defined as a mass per unit volume of the object in dry condition. Dry density was measured for the core samples whose diameter and length were measured by means of Vernier Caliper and then volumes of the cores were calculated. The dry weight was taken with the help of electronic balance. The dry density was calculated as follows.

$$\text{Dry density} = \frac{\text{Dry weight}}{\text{Volume of a cylindrical sample}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (5.7)$$

WAV ranges from 0.1 to about 3% for aggregate used in road construction (ASTM 1994).The measured WAV values in present core samples ranges from 0.19

to 0.67 which is quite low and lies with in the suggested range <1 of BS 1985. Overall aggregates should not have high porosity for pavement construction. Low value of test samples indicates Limestone is strong for pavement construction.

The dry density of aggregates normally used in road construction ranges from 2.5 to 3.0 with an average value of an about 2.68 (ASTM 1994). The measured dry density varies from 2.49 to 2.65 which indicate that the aggregates are suitable for pavement construction.

Fig. 5.9 Core samples for the determination of dry density and water absorption.

Table 5.7: Determination of dry density and water absorption value of different sample

Sample no.	Core length(cm)	Core dia.(cm)	Volume	Dry wt.(X_1) in gm	Saturated wt.(X_2) in gm	Dry density(gm/cm^3)	WAV%
BR1	4.33	2.543	21.981	54.85	55.05	2.495	0.365
BR2	3.89	2.532	19.577	51.25	51.4	2.618	0.293
BR3	5.78	2.531	29.066	77.25	77.4	2.658	0.194
BR4	3.76	2.526	18.833	47.9	48.15	2.543	0.522
BR5	2.96	2.531	14.885	39.2	39.3	2.634	0.255
BR6	3.7	2.53	18.591	48.5	48.65	2.609	0.309
BR7	4.54	2.536	22.920	59.15	59.55	2.581	0.676

5.8: WAV standard value for pavement layers

Serial no.	Types of pavement	Maximum value
1	Base course	4
2	Surface course	<6
3	Bituminous surface	10

5.4 MECHANICAL PROPERTIES

5.4.1 Aggregate crushing value

Aggregates used in road construction, should be strong enough to resist crushing under traffic wheel loads. The strength of course aggregates is assessed by aggregates crushing test. The aggregate crushing value (ACV) provides the relative measure of resistance to crushing under the gradually applied compressive load. To achieve a high quality of pavements, aggregate possessing low ACV should be preferred. About 6.5 kg of aggregate is required for preparing two samples.

The aggregate passing 25 mm sieve and retained on 20 mm sieve were selected for the test. The cylindrical measure was filled by the test sample of aggregate in three layers of approximately equal depth, each layer being tamped 25 times by the rounded end of the tamping rods. After the third layer was tamped, the aggregates at the top of the cylindrical measure were leveled off by using the tamping rod as a straight edge.

The surface of the aggregates was leveled and the plunger inserted so that it rest on this surface in level position. The cylinder with the test sample and plunger in position was placed on compression testing machine. Load was then applied through the plunger at a uniform rate of 4 tonnes per minute until the load is 40 tonnes, and then the load released (Fig. 5.10). Aggregates including the crushed portion were removed from the cylinder and sieved on a 4.75 mm IS sieve. The material which passes the sieve was collected. Then ACV was calculated by using an empirical formula given below.

$$ACV = \frac{W_2}{W_1} \times 100\% \ldots \ldots \ldots \ldots (5.8)$$

where, W_1 is the total weight of the dry sample in gram and W_2 is the weight of the aggregate passing 4.75 mm sieve

The results of the ACV are tabulated (Table 5.9). Thus the obtained ACV ranges from 20 to 30% which do not exceed the given standard NRS (DOR 2001). This indicates that aggregates are strong, as a crushed fraction is low. Therefore, as the stress at the base and sub-bas courses are low, aggregates with lesser crushing strength may be used.

Fig. 5.10 Determination of Aggregate Crushing Valve

Table 5.9: Results of aggregate crushing value

Sample No.	Total weight of dry sample W_1(gm)	Weight of sample passing through 4.75mm seive W_2(gm)	ACV %
BR1	3000	725	24.1667
BR2	3000	775	25.8333
BR3	3000	755	25.1667
BR4	3000	795	26.5
BR5	3000	790	26.3333
BR6	3000	785	26.1667
BR7	3000	794	26.4667

5.4.2 Aggregate impact value

Toughness is the property to resist impact. Due to traffic loads, the road stones are subjected to the pounding action or impact and there is possibility of stone breaking in to smaller pieces. The road stones should therefore be tough enough to resist fracture under repeated condition. Thus, a test designed to evaluate the toughness of the stone i.e., the resistance of the stones to fracture under repeated impact is called an impact test for road stones. The aggregate impact test is considered an important test to assess the suitability for use in pavement construction.

In this test, sample consists of aggregates passing 12.5 mm sieve and retained on 10 mm sieve. The aggregates were filled in a cylinder one-third at a time and tamped 25 times with tamping rods. This process was repeated three times. Test sample was then fixed in position on the base of the machine.

The hammer was raised until its lower face was 38 cm above the upper surface of the aggregates in the cup, and allowed to fall freely on the aggregates

(Fig. 5.11). The test sample was subjected to a total of 15 such blows, each being delivered at an interval of not less than one second. The crushed aggregate was then removed from the cup and the whole of it sieved on the 2.36 mm sieve until no further significant amount passes. The mass of an aggregate passing on 2.36 were taken for calculation of aggregate impact value (ASTM, 1979).

$$\mathrm{AIV} = \frac{\mathrm{W_3}}{\mathrm{W_1}} \times 100\% \quad \ldots\ldots (5.9)$$

where, W_1 is the total weight and W_3 is the weight of an aggregate passing on 2.36 mm.

The results of AIV are given in Table 5.10. The results shows that the AIV lies between 10 to 20%, which is under the standard value of ASTM, 10-20%, BS < 20% and NRS < 30%. Therefore the entire sample indicates strong enough to resists impact and is appropriate for both base and sub base course.

Fig. 5.11 Determination of aggregate impact value

Table 5.10: Results of aggregate impact value

Sample No.	Total weight ,W_1 (gm)	Weight retained, W_2 (gm)	Weight passing ,W_3 (gm)	AIV%
BR1	500	449.69	50.31	10.062
BR2	500	444.59	55.41	11.082
BR3	500	439.11	60.89	12.178
BR4	500	439.37	60.63	12.126
BR5	500	429.73	70.27	14.054
BR6	500	429.31	70.69	14.138
BR7	500	442.75	57.25	11.45

Table 5.11: Aggregate properties with AIV test standards

AIV(ASTM,1979)	Remarks
<10%	Exceptionally strong
10–20%	Strong
20–30%	Satisfactorily for road surfacing
>35%	Weak for road construction

5.4.3 Los Angeles Abrasion test

Resistance to wear or hardness is an essential property for road aggregates especially when used in wearing course. Thus, road stones should be hard enough to resist the abrasion due to the traffic. Hardness and toughness of aggregates associated together are often carried out in the Los Angeles Abrasion test. The principle of the test is to find the percentage wear due to the relative rubbing action between the aggregates and the steel balls used as abrasive charge. This test has been standardized by the ASTM (1981).

Los Angeles abrasion test consists of hollow steel cylinder closed at the both the ends and mounted on stub shaft about which it rotates on the horizontal axis. An opening was provided in the cylinder for the introduction of the test sample. A removable cover of the opening was provided in such a way that when closed and fixed by bolts and nuts, it was dust-tight and interior surface was perfectly cylindrical. Abrasive charge, consisting of cast iron spheres approximately 4.8 cm in diameter and 390 to 445 gm in weight were used. The number of sphere and weight of the abrasive charge used depending on the gradation of the aggregate test (Table 5.12).

During our test "A-grade" sample were prepared for the test. Sample weighting 5 kg was placed in the cylinder of the machine and twelve number of abrasive charge sphere according to gradation of aggregate were also placed in the cylinder containing test sample. The cover was then fixed dust tight. The machine was rotated at a speed of 30 to 33 revolutions per minute. The machine was rotated for 500 revolutions. After the completion of revolution the material was discharged from the machine. By using the sieve size of 1.75mm, sample was sieved and sample retained in the sieve was weight which helps in the calculation of Abrasion Value (Fig. 5.12).

$$\mathrm{LAA} = \frac{\mathrm{W}_1 - \mathrm{W}_2}{\mathrm{W}_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (5.10)$$

where, W_1 is the original weight of aggregate and W_2 is the weight of aggregate retained on 1.70mm IS sieve after test.

Fig. 5.12 Determination of abrasion value

The los angles abrasion value varies from 26 to 30%. This means it is under the acceptable limit of NRS (DOR 2001), ASTM (1979). According to the results, the aggregates can be used road aggregates in base and sub base courses.

Table 5.12: Grading of test samples (materials finer than 37.5mm) (ASTM Designation, C131–69).

Seive passing-retaining	Grade of sample(Wt.of fraction by seiving in gm)			
	Grade A	Grade B	Grade C	Grade D
40–25	1250 ± 25			
25–20	1250 ± 25			
20–2.5	1250 ± 25	1250 ± 25		
12.5–10	1250 ± 25	1250 ± 25		
10–6.3			1250 ± 25	
6.3–4.75			1250 ± 25	
4.75–2.36				5000 ± 10
Total	5000 ± 10	5000 ± 10	5000 ± 10	5000 ± 10

Table 5.13: Result of Los Angeles abrasion test

Sample No.	Total weight W1	Wt.retained W2	Loss in weight(W1–W2)	LAA%
BR1	5000	3465	1535	30.7
BR2	5000	3500	1500	30
BR3	5000	3580	1420	28.4
BR4	5000	3540	1460	29.2
BR5	5000	3490	1510	30.2
BR6	5000	3600	1400	28
BR7	5000	3660	1340	26.8

5.4.4 California Bearing Ratio test

This is the method of classifying and evaluating soil sub grade and base course materials for flexible pavements. It is a measure of resistance of materials to penetration of standard plunger under controlled density and moisture conditions. The test procedure may be conducted in re-moulded or undistributed specimen the laboratory.

Re-moulded soil specimen may be compacted either by static compaction or dynamic compaction. During this test dynamic compaction was used in which about 45 kg of materials was dried and sieved through 20 mm sieve and sample retained on 4.75 mm sieve was taken. The optimum moisture content and maximum dry density of the soil were determined by adopting IS light compaction (Proctor compaction) (Annex Table 5).

After compacting the last layer, the collar was removed and the excess soil above the top mould was evenly trimmed off by means of the straight edge .The such compacted specimen were prepared for the CBR test. About 100 gm of soil samples were collected from each mould for moisture content determination from trimmed of portion. The clamps were removed and the mould with the compacted soils was lifted leaving below the perforated base plate and the spacer disc which was removed. The mould with the compacted soil was weighed. A filter paper was placed on the perforated base plate, the mould was inverted and placed in position over base plate and the clamps of the base plate were tightened. Another filter paper was placed on the top surface of the sample and the perforated plate with adjustable stem placed over it. Surcharge weights of 2.5 or 5.0 kg weights were placed over the perforated plate and the whole mould with the weight was placed in the water tank for soaking such that water can enter the specimen both from top and bottom. The swell measuring device consisting of the tripod and dial gauge were placed on the top edge of the mould and the spindle of the dial gauge was placed touching the adjustable stem of the perforated plate. The initial dial gauge reading was taken and the test set up was kept undisturbed in water tank to allow soaking of the soil specimen for four full days or 96 hours. Final dial gauge was noted to measure the swelling.

The mould with the specimen was clamped over the base plate and the same surcharge weights were placed on the specimen centrally such that the penetration test could conducted. The mould with the base plate was placed under the penetration plunger of the loading machine. The penetration plunger was seated at the centre of the specimen and is brought in contact with the top surface of the soil sample by applying a seating load of 4 kg. The dial gauge for load reading and the penetration dial gauge were set to zero. The load was applied through the penetration plunger at a uniform rate of 1.25 mm/min. The load readings are recorded at the penetration readings of 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0, and 12.5. After the final readings, the load was released and the mould from the loading machine. The proving ring calibration factor was noted so that the load dial values can be converted in to load in kg.

Calculation,

The swelling or expansion ratio was calculated from the observation the swelling test using a formula

$$\text{Expansion ratio} = \frac{100(D_f - D_i)}{H} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (5.11)$$

where, D_f is the final dial gauge reading after soaking, D_i is the initial dial gauge reading before soaking and H is the initial height of the specimen.

Here, D_f = 25 mm

D_i = 10 mm

H = 120 mm

Therefore, expansion ratio is equal to12.5.

The load values noted for each penetration level are divided by the area of the loading plunger (19.635 cm^2) to obtain the pressure or unit load values on a loading plunger. The load penetration curve is than plotted in natural scale for each specimen (Fig. 5.13). Load values corresponding to 2.5 and 5.0 mm penetration values are found from the graph.

The CBR value was calculated from the formula;

$$\text{CBR} = \frac{\text{Unit load carried at defined penetration level}}{\text{unit load carried above penetration level}} \times 100\% \dots\dots\dots (5.12)$$

Therefore,

$$\text{CBR at 2.5 mm} = \frac{42.78}{70} \times 100\% = 61.114$$

$$\text{CBR at 5 mm} = \frac{64.17}{105} \times 100\% = 61.114$$

Thus, CBR of the soil sample is 61.

According to NRS (DOR 2001) minimum CBR value for base course is 80% and sub base is 60%. Here, CBR values thus obtained is 61% which is under the acceptable value for base and sub base course. This means that a material is suitable for sub grade and granular sub base course.

Table 5.14: Results of the CBR test value

Penetration (mm)	Load on plunger in sample	Unit/load kg/cm2
0.5	150	7.639
1	220	11.204
1.5	340	17.316
2	550	28.011
2.5	840	42.781
3	940	47.874
4	1040	52.967
4.5	1150	58.569
5	1260	64.171
5.5	134	6.825
6	181	9.218
7	221	11.255
7.5	3000	152.788
8.5	3450	175.707
10	4220	214.922
11	5160	262.796
12.5	5550	282.659

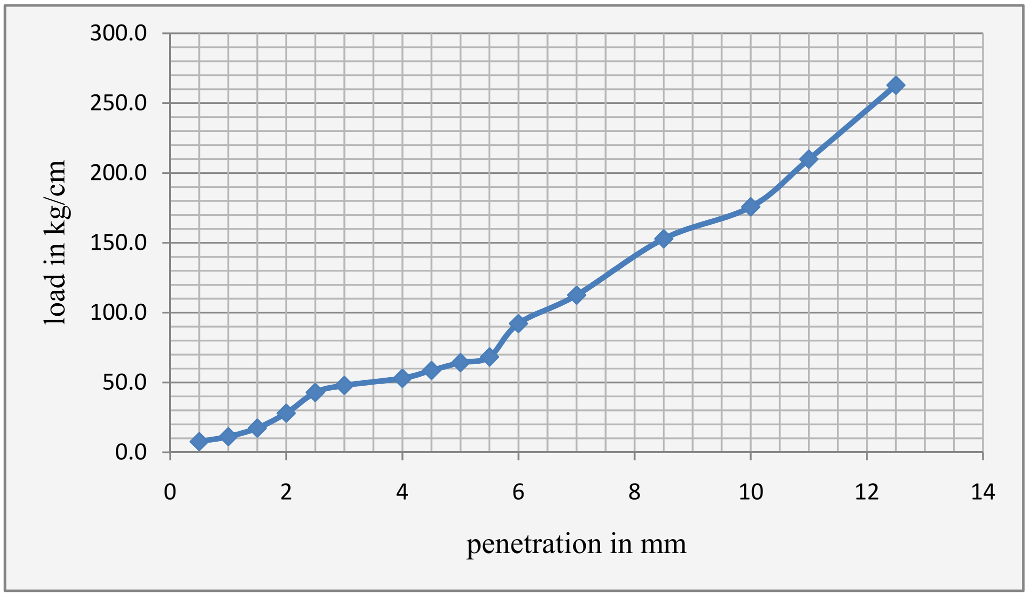


Fig. 5.13 Test load penetration curve

5.5 CHEMICAL PROPERTIES

5.5.1 Chemical durability (sodium sulphate soundness test)

Soundness test is measures of how resistant an aggregate is to chemical weathering. Soundness test determine the resistance to disintegration of aggregates due to alternate cycle of dry and wet condition.

Sample of size 10–14 mm and mass 455 gm for the test washed with distilled water and oven dried at 105–110^{o}c. A saturated solution of sodium sulphate was produced with the density of 1.32 gm/ml. The test specimen were then subjected to five 48 hours immersion and drying cycle as in Fig. 5.14, in which they were Immersed in saturated solution for 16 –18 hours

Drained for two hours

Oven dried at 105–110^{0}c for 24 hours

Cool for five hours at lab temperature

After completion of five cycles the sample were washed to free it from the salt, oven dried and hand sieved on the same 10 mm sieve used in the preparation of the sample. Then sample retained on the sieve was weighted and thus the sodium sulphate soundness value was calculated as:

$$SSV = \frac{W_1 - W_2}{W_1} \times 100\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.(5.13)$$

where, W1 is the initial weight of the sample and W_2 is the weight retained on 10 mm after the test.

Calculated SSV are listed in Table 5.15. The result of the test a sample varies from 6 to 12% which is under the acceptable range of NRS (DOR 2001) for base and sub base course. SSV less than 12% means the aggregates samples are sound and resistant against chemical weathering and frost susceptibility.

Table 5.15: Results of the soundness test

Sample No.	Initial wt.W1(gm)	Final weightW2(gm)	SSV%
BR1	455	415	8.79
BR2	455	408	10.33
BR3	455	416	8.57
BR4	455	409	10.11
BR5	455	424	6.81
BR6	455	426	6.37
BR7	455	402	11.65
BR7+BR2+BR5	455	411.9	9.47
BR4+BR7	455	399.8	12.13
BR3+BR6	455	401	11.87

Fig. 5.14 Samples at different cycles during the test a) Initial sample before testing b) First cycle c) Second cycle d) Third cycle e) Fourth cycle f) Fifth cycle

CHAPTER 6

EVALUATION OF CRUSHED ROCK AGGREGATES FOR ROAD MATERIAL

As already mention that sub grade, sub base and base courses are the major structural layers of the unbound pavements. Sub grade being upper layer of earthwork, it includes treatment of formation, addition of layers of selected materials. So during our work CBR is performed for the evaluation of sub grade. Since the work is based on crushed rock aggregate our evaluation is mainly focused on sub base and base course. The result of the analysis of an aggregate is summarized in Table 6.1.

According to field studies Limestone can be categorized into three classes according to surface texture, grain size and color as light grey course crystalline limestone, medium grained siliceous limestone and fine grained argillaceous limestone but these varieties when classified petrographically on the basis of total carbonate content and grain size in engineering classification can be categorized in to crystalline limestone, calcareous siltstone and siliceous limestone respectively.

These three categories of samples can be evaluated with NRS (DOR 2001) on the basis of petrographic, physical, mechanical and chemical properties (Table 6.2, 6.3 and 6.4).

Table 6.1: Obtained value of different parameters

Sample no.	WAV%	Dry density (gm/cm^3)	LAA %	AIV %	ACV%	SSV %	FI %	EI %	CBR %	Compositional group
BR1	0.364	2.594	30.30	10.04	24.16	8.79	25.02	75.49	61	Crystalline limestone
BR2	0.292	2.615	30.00	11.08	25.83	10.32	22.36	90.79		Siliceous limestone
BR3	0.199	2.652	28.40	12.17	25.16	8.57	22.05	76.03		Crystalline limestone
BR4	0.521	2.536	29.20	12.12	26.50	10.10	20.29	83.92		Calcareous siltstone
BR5	0.255	2.626	30.20	14.05	26.33	6.81	19.73	89.95		Calcareous sandstone
BR6	0.309	2.605	28.00	14.12	26.16	4.37	16.25	86.68	61	Siliceous limestone
BR7	0.676	2.567	26.80	11.45	22.00	11.64	16.01	87.61		Siliceous limestone

Table 6.2: Evaluation of crystalline variety with NRS (DOR 2001)

Properties		Obtained value	NRS (DOR 2001) Base course	NRS (DOR 2001) Sub base course	Remarks
Petrographic	FI %	22.05–25.05	25	30	Appropriate for both base and sub base course
	EI %	75.49–76.03			Good road aggregates
Physical	WAV %	0.199–0.314	<4	<4	Low effective porosity and acts as good road aggregates
	DD %	2.594–2.652	2.622	2.622	Average density for aggregates
Mechanical	ACV %	24.16–25.16	<30	<30	Strong as crushing value is low
	AIV %	10.04–12.17	20-25	30	Strong enough to resist impact for both course
	LAA %	28.40–30.30	30-35	40	Best for road and concrete
	CBR min. %	61	80	60	Appropriate for sub base course
Chemical	SSV %	8.57–8.79	12	12	Chemically sound and good for both courses

Table 6.3: Evaluation of Siliceous variety with NRS (DOR 2001)

Properties		Obtained value	NRS (DOR 2001) Base course	NRS (DOR 2001) Sub base course	Remarks
Petrographic	FI %	16.01–22.36	25	30	Appropriate for both base and sub base course
	EI %	86.68–90.79			Good road aggregates
Physical	WAV %	0.292–0.676	<4	<4	Low effective porosity and acts as good road aggregates
	DD %	2.567–2.615	2.622	2.622	Average density for aggregates
Mechanical	ACV %	22.03–26.16	<30	<30	Strong as crushing value is low
	AIV %	4.8–14.12	20-25	30	Strong enough to resist impact for both course
	LAA %	26.80–30.00	30-35	40	Best for road and concrete
	CBR min. %	61	80	60	Appropriate for sub base course
Chemical	SSV %	4.37–11.64	12	12	Chemically sound and frost susceptibility

Table 6.4: Evaluation of calcareous variety with NRS (DOR 2001)

Properties		Obtained value	NRS (DOR 2001) Base course	NRS (DOR 2001) Sub base course	Remarks
Petrographic	FI %	19.73–20.29	25	30	Appropriate for both base and sub base course
	EI %	83.92–89.95			Good road aggregates
Physical	WAV %	0.255–0.521	<4	<4	Low effective porosity and acts as good road aggregates
	DD %	2.536–2.626	2.622	2.622	Average density for aggregates
Mechanical	ACV %	26.38–26.50	<30	<30	Strong as crushing value is low
	AIV %	12.12–14.05	20–25	30	Strong enough to resist impact for both courses
	LAA %	29.20–30.20	30–35	40	Best for road and concrete
	CBR min. %	61	80	60	Appropriate for sub base course
Chemical	SSV %	6.81–10.10	12	12	Chemically sound and frost susceptibility

Though limestone differ in percentage carbonate content and grain size their physical, mechanical and chemical properties do not varies exceeding the NRS (DOR 2001) which suggest that crushed limestone aggregates are suitable for unbound pavements and can make performance better than those having uniform composition.

Overall evaluation of crushed rock limestone aggregates for road aggregates can be made by comparing the obtained value with different standard (Table 6.5). Weathering grade of the rock of quarry area varies from IB to II which means that the rock is faintly to slightly weathered and can be better for road aggregates.

Since the materials to be used are natural crushed aggregates it should be screened to contribute to the uniformity, workability and plasticity of the materials as it is mixed.

The surface features such as angular shape and rough crystalline texture are the valuable guides relative to the internal frictional properties of an aggregate which resists the movement of aggregates past each other and considered as excellent road aggregates.

On the basis of characters such as spacing of joints, persistence (Rock mass strength classification of John 1962 and Bieniawski 1973) the rocks of the study area are sound and medium strength rock mass which can be only extracted by blasting and fracturing.

According to NRS (DOR 2001), the shape of the aggregates i.e. flakiness index should not exceeds 25%. The FI values of all the samples are < 25 % which is under the acceptable limit of the entire standard. FI should not exceeds the given range of value 25–30 as rounded aggregate are preferred in cement concrete road construction as the workability of concrete improves and angular shape of particles are desirable for granular base course due to increased stability derived from better interlocking.

WAV value of the limestone obtained varies from 0.19–0.67. This means that the limestone has very low effective porosity. WAV is under the accepted value of ASTM C33, 1994 i.e. < 3% and < 1% according to BS 812, 1975 for road surfacing, while values of up to 4% may be accepted in road bases.

ACV ranges from 22–26 which means that limestone is strong enough to resists fracture under an applied compressive load. And materials with values greater than 35 are very weak to be utilized in a pavement.

LAA values of the limestone thus obtained varies from 28 to 30 which means that the aggregates are acceptable for road base and sub base courses which is under the NRS (DOR 2001) i.e. < 40%.

AIV, the toughness value of an aggregate obtained varies from 10–14 which is under the acceptable limit of NRS (DOR 2001). AIV value for base and sub base course is < 30% according to ASTM C33, 1994 and BS 812, 1975. Sodium soundness value, useful to access the resistance of an aggregate to weathering is with in the acceptable range of 12% for road base and sub base after 5 cycles (ASTM C33, 1994) so the aggregate samples are sound and resistance against chemical weathering and frost susceptibility.

CBR value is 60 which is an acceptable value for road aggregate according to NRS (DOR 2001).

The specific gravity of aggregate normally used in road construction ranges from about 2.5 to 3.0 with an average value of about 2.68 (Khanna and Justo 2004). Obtained value of dry density varies from 2.4–2.6 which is under the acceptable range.

Table 6.5: Comparison of different standards and their remarks with obtained values of aggregates test

Properties		Obtained value	ASTM C33, 1994	BS 812,1985	NRS (DOR 2001)		Remarks
					Base course	Sub base course	
Petrographic	FI %	16.06–25.02			25	30	Appropriate for both base and sub base course
	EI %	75.49–90.79					Good road aggregates
Physical	WAV %	0.199–0.676	<3	<1	<4	<4	Low effective porosity and acts as good road aggregates
	DD %	0.536–0.652	2-3.1		2.622	2.622	Average density for aggregates
Mechanical	ACV %	22.00–26.50			<30	<30	Strong as crushing value is low
	AIV %	10.04–14.12	10-20.5	<20	20-25	30	Strong enough to resist impact for both course
	LAA %	26.80–30.30	<30		30-35	40	Best for road and concrete
	CBR min. %	61			80	60	Appropriate for sub base course
Chemical	SSV %	4.37–11.64	<10	<10	12	12	Chemically sound and good for both courses

CHAPTER 7

GRADE ANALYSIS AND RESERVE ESTIMATE

Limestone of Adeshwar deposits are within the Chandragiri Formation of Phulchowki group. The quality assessment and reserve estimates are described as:

7.1 SAMPLING

Sampling is an essential tool to evaluate the limestone deposits at every stage of its study from a prospecting to operating a mine. Sampling is done to ascertain the characteristics of mineral deposits such as their grade. Sampling basically helps in evaluating the deposits. The type of sampling depends upon the nature of study whether it is preliminary or detailed. The methods of sampling during an exploration of deposits are as follows,

- Grab sampling
- Chip sampling
- Channel sampling

During the present study, grab sampling method applied.

7.2 CHEMICAL ASSESSMENT

The quality of limestone is essential for manufacturing cement because certain chemical constituent control the grade. But present study is considered for using limestone as a road aggregate. Though the grades do not play any role in the construction of unbound pavement the chemical composition should be with in the certain limit to make the limestone usable for cement production.

The collected samples were tested at Bridge cement industries Pvt. Ltd.Bhairahawa. There results are given in Table 7.1

Table 7.1: Chemical analysis of bulk sampling

Sample NO.	CaO %	M gO %
BR1	47.38	3.2
BR2	44.28	5.4
BR3	48.8	1.2
BR4	38.6	8.4
BR5	42.8	4.4
BR6	49	0.8
BR7	45	3.2

7.3 ESTIMATION OF RESERVE

The estimation of reserves is the ultimate goal of a certain stage of exploration to determine quantity and quality of deposits. Ascertaining the spatial distribution of grade in deposit as a whole, and in its separate blocks and provisional determination of economic importance of the estimated reserves are also objective of reserve estimation.

7.3.1 Classification of Reserve

Reserve estimation are placed under three categories,

- Proved reserve estimation is based on the sufficient data such that, it will not vary much from the actual tonnage and grade when mined.
- Probable reserves are known as indicated reserves which carry a lesser degree of assurance and are based on a limited data of sampling and core logging.
- Possible reserve estimation is done from extrapolation of sampling data to areas where there is no data of sampling available is termed possible reserves (inferred reserves).

7.3.2 Reference data for reserve estimation

The reserves of deposit ‘Q’ in terms of tonnage can be obtained by multiplying the volume of ore body ‘V’ or the volume of the block of the deposit ‘V’ with the tonnage factor ‘T'.

$Q = V \times T$

Total reserve, $Q = \sum (V \times T)$

Where $V= St$

S= cross sectional area, m^2

t= strike length, m

T = Tonnage factor (the total volume in cubic meters multiplied by the specific gravity of the minerals gives directly the tonnage in metric tones. And this specific gravity here is called the tonnage factor).

The accuracy of the reserve calculation depends upon the extent to which detail works were carried out. Dividing the ore body into different blocks with respect to the sampling locations and computing the tonnage of each block separately can give the desired accuracy.

Vertical section method is used for most of the steeply dipping deposits, and for the nonmetallic and less precious minerals like limestone. In this method different cross sections are prepared across the strike of deposit with respect to the sampling point and then the cross sectional area of the deposit with desired grade is calculated and then multiplied with the strike length of the area that comes into the influence of the sampling point.

For the reserve calculation of the deposits three vertical cross section as in Fig. 7.1 were prepared oblique to the strike of the bedding planes of the deposits on the reserve estimate map of scale 1:100 of the deposit area.

Table 7.2: Reserve calculation of the Adeshwor limestone deposit

Cross– section	Cross sectional area of the limestone (m^2)	Strike length (m)	Volume of limestone (m^3)	Tonnage factor	Minable reserve (tonnes)
AA’	1552.42	100	155242	2.6	403629.2
BB’	3381.28	100	338128	2.6	879132.8
CC’	5228.66	100	522866	2.6	1359451.6
Total					**2642213.6**

As this reserve calculation has been done on the basis of few grab samples and existing geological data from the deposit area the reserve can be categorized as ‘probable or inferred deposits’.

CHAPTER 8

MINING STATUS, SUGGESTED PLANS, METHODS AND ENVIRONMENTAL MANAGEMENT

8.1 MINING STATUS

Adeshwar limestone quarry lies at latitude $27^0$42'30" to $27^0$44'00'' and longitude $85^0$15'00' to $85^0$16'43''. Adeshwar deposit lies at the top of the hill of 1606m with hill slope of 30–65^0. Northern slope of the hill is steeper than the southern slope, which bear scattered bushes whereas southern slope bear contour. Outcrops are prominent on northern slope. Around Adeshwar deposit, there are steep mountains and many small valleys, scarcely flat area. Overburden to be removed seemed to be thin (Fig. 8.1).

Geologically, this deposit contains not only thick to thin bedded, crystalline white to dark grey limestone but also very low volume of brown residual soil which can also be used as a construction purpose as a infilling materials. For getting limestone efficiently, residual soils should be removed initially before the extraction of limestone.

The Adeshwar deposit has been worked out in local scale but it will be worked with well developed technique in near future. Mining is continued at present at the road from Sitapaila to Ichangu, near the northern edge of the deposit on the right bank of the Jughe Khola.

Fig. 8.1 Out crop of the limestone deposit facing north

Fig. 8.2 Present condition of mining in the deposit at local scale facing north

Mining in Adeshwar is haphazard in present status. Traditional mining tools such as chisel, hammers, and shovel have been used to extract the blocks of stones from the quarry. Rock fragments are randomly thrown from the uphill slope to the bank of Jughe Khola with the help of gravity without taking any safety measures which can be seen in the Fig. 8.2. Many people give the desired grading to the aggregates depending on the requirements. Such activities continue by crushing the fragments using hammers on the right bank of the Jughe Khola or on the different places of Sitapila VDC.

Two sets of joints prominently present in the limestone which have their joints plane near vertical and forms wedges with the intersection of bedding planes. North dipping bedding planes with attitude $86^0/76^0$N, nearly vertical joints with attitude $10/83^0$S and formation of wedges make the extraction of the limestone by manual operation easy.

8.2 SUGGESTED MINE PLAN AND DESIGN

Considering the consent of land owners, land use pattern, human settlement and topography of the area only about 45000 sq.m of total surveyed area was taken for stone quarrying.

The lower limit of quarry (QLL) will be kept at 1405 m, which is about 5 m above the level of the Jughe Khola. The upper limit of the mining will be kept at the altitude of 1515 m towards the south of the section CC' Fig. 7.1.

The layout of the quarry is designed as such the topmost bench level will be at a height of 1505 m. Excavation will commence from this level and proceed downwards forming one bench at a time resulting horizontal slices.

Other corresponding bench levels will be developed at a 3 m bench height and 3 m bench width so that the vehicles can move in all levels with ease and the transport of materials will be done with no difficulty. The strike of the bench will be almost perpendicular to the bedding direction of rock, which is almost northeast to southwest. The general advance of the bench will be from the southeast to northwest and from the top to the bottom.

The dump yard has been proposed at the lower part of the quarry area so that back filling can be done whenever desired. A narrow drainage channel around the dump yard as well as along the access to every bench levels will be made in order to drain out the water during rainy season. Also, one siltation pond has been proposed below the level of dump yard towards the north of the dump yard.

8.2.1 Purposed mining method

Opencast mining with the benches moving toward bottom from the top is the most appropriate method for quarrying the stone deposit with the less overburden as well as less commercial value. This deposit will be mined by open-cast stripping from top to bottom in successive layers or benches as in Fig. 8.3.

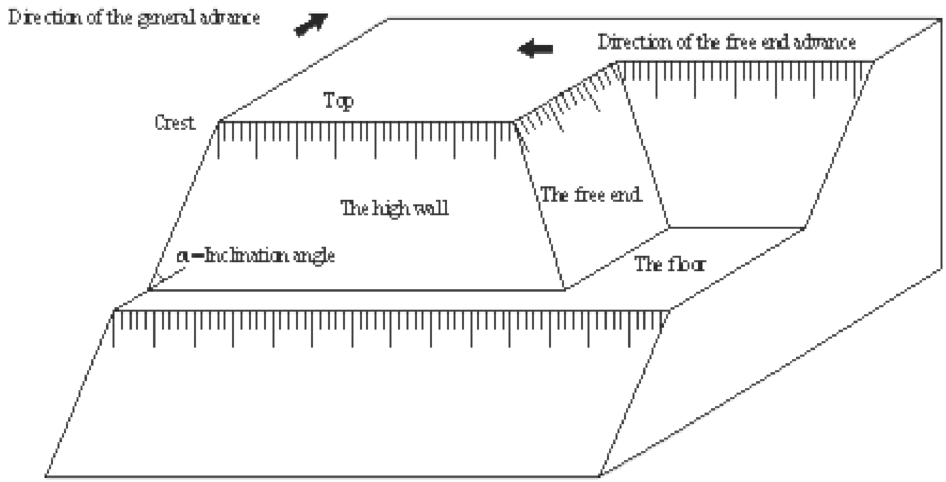


Fig. 8.3 Basic elements o f benches in open cast mining

Opencast mining method will be adopted for the extraction of stone deposit with the use of traditional mining tools such as chisels, hammers, crowbars, wedges, hand drilling machine, hydraulic splitter and rock breaker with the combination of back hoe shovel. Overburden and the stones are loaded manually or by shovel into trucks and kept separately.

8.2.2 Mine development work

The motorable road already exists near by the proposed quarry site, which passes along the Jughe Khola just below the lowest mining level. Also another road already exist the quarry to the Kallabari which works as a haulage road. The road will be of 4 m wide and will be provided with several wider sections to enable on coming vehicles to pass by. The alignment of the approach road has been set in such a way that there will be minimum change in it when the quarry level moves down. Access to all bench levels and exit road for the dumpers will be made.

Before quarrying all the bushes, soil mass and any permanent features will be removed from the bench faces where the area is to be quarried.

8.2.3 Rated production versus mine life

Number of working days in a year = 300

Number of working shifts in a days =1

Number of working hours in a shift = 8

Daily production of the quarry is proposed to be 156.72 m^3 (60 Trips) in average.

(Considering the volume of the truck = 2.612 m^3)

Annual Output = 300 X 156.72= 47016 m^3

(Note: Number of working days in a year is considered to be 300 days only because there will be no wok in rainy season as the approach road to the site will be blocked and during festivals, national holiday workers are not willing to work.)

Thus, mine life =2642213.6×1.2/47016×2.6

= 26 years.

where, 1.2 is the coefficient of expansion in volume due to fragmentation and 2.6 is tonnage factor.

8.2.4 Mining parameters

Mining is done considering the following factors:

Bench height = 3 m

Bench width = 3 m

Working bench slope = 60^o

8.2.5 Method of excavation

Hand held tools such as shovel, crowbars, wheelbarrow, hammer, peak, spade, chisel etc will be used for the extraction of the boulders. A splitter, crowbar will be inserted as wedge inside such joints so that rocks pieces are broken down. Broken pieces are still broken to the suitable size to carry on wheel-barrows. The 3 m wide benches will be used as working platform. Benches are excavated from outer wall and advance towards the inner side. However the distance between two consecutive working walls should not be less than 3 m.

Drilling can be also used for the purpose of pre-splitting of the rock mass with the combination of hammer and chisel method. In this method first a 2–3 feet long drill hole is drilled in to the rock mass with the help of hand held drill machine in the square pattern. Then in every hole, a wedge is inserted with the help of hammer. With this process splitting occurs in the rock mass.

In case of massive rock mass, shovel with rock breaker or hydraulic rock splitter with the combination of secondary drilling will be used for hard rock excavation depending upon the site conditions. And for big boulder hand held drilling is done. After that splitting the stone is done by inserting splitter into the drilled holes.

8.2.6 Quality control

Quality control is mainly related to the separation of the limestone and top most residual soil. Since, in our quarry site topmost residual soil is removed initially during stripping there is no need of separate any mix up mass.

8.2.7 Transportation system

Broken rock and waste material can be transported from the bench face by hired tipper or truck up to the desired location. Materials can be loaded into the vehicle either manually or by any means of mechanical support, depending upon site situation.

8.3 MINE ENVIRONMENT MANAGEMANT

Quarrying in any area certainly generates some negative impacts. Quarrying is such a process in which the topsoil has to be removed first and the rock is excavated only after that. Both types of work may affect the area up to some extent. But miners should not be concentrated only in excavation work. Consideration will also be given to minimize the negative impact in the vicinity for the smooth and long running of the quarry.

8.3.1 Careful mining and drainage management

Mining will be done by stripping layers form top to bottom in benches. This minimizes many negative environmental impacts and provides enough safety to the workers. The bench parameters like bench height and bench width will be maintained 3 m each. Although the mining operation will not be carried out during rainy season, a narrow drainage channel around the dump yard as well as along the access to every bench levels will be made in order to drain out the water so that the loose materials generated in earlier works may not be easily washed out into the Jughe Khola. The running water both from the quarry and the dump yard will be drained into the siltation pond which has been proposed to construct towards the north of the quarry on the right bank the Jughe Khola. It will provide the sufficient time for the sedimentation. The sediments from the pond will be dredge out regularly.

8.3.2 Overburden and Waste Management

The dump yard has been proposed at the northern lowermost part of the proposed quarry. The height of the gabion wall and also the length will be maintained as per requirement The dump yard will be protected either by dry wall or by gabion wall about 50 m long to prevent wash out of the soil by rain water along with the drainage canals. A siltation pond will be also made towards the north of the dump yard to allow the sedimentation of the run off water from dump yard and the quarry before entering into the Jughe Khola. At the end of mining, soil from dump yard will be spread over the flat area at the level of 1405 m created after the quarry work for reclamation work.

8.3.3 Soil conservation

Soil erosion and fines from freshly excavated benches and dumps will be carried out by the rain water out by the provision of well networking of drainage. Special local stone paved chutes and channels will be provided to allow controlled descent of water. Gully formation will be prevented by filling with local stones or sand filled bags. Inactive dump slopes will be planted with bushes, grass, shrubs and trees to prevent soil erosion.

8.3.4 Other measures

- Water sprinkling on the main haulage road will be carried out.
- Dust extraction system will be used in drill machines. Water injection methods are also proposed for dust suppression at the generation point.
- Dense tree belts will be developed around the mine
- Proper maintenance and repairing of machinery and transport vehicles will be ensured.
- Turfing is highly recommended over the non-working slopes, the area where future excavation is not called for. The worked out quarry will be reclaimed physically and trees will also be planted and thus land will be rehabilitated.

8.3.5 Mitigation measures for social, economical and cultural environment

The local mud road in the proposed quarry site exists just above the river level. This local mud road may be impacted adversely during the construction of the quarry road and during the transportation. However, the leaseholder has done agreement with local people and the VDC to compensate and construct the alternatives for the system if any damage or interruption occurs during the mining interrelated activities. The others measures as per agreement area as follows:

- Chronological maintenance and repairing of the existing mud road will be done.
- Compensation will be provided for affected families for the loss of cultivated land as per the rules of the Government of Nepal.

- Priority will be given to the local people for the employment as per the qualification.

8.4 MINE SAFETY MANAGEMENT

Mining activities are carried out semi-mechanically. Accidents like rock-fall, down-fall of workers, fly rock, slope failure, hammering on body, and accident due to drilling or other are still possible to occur during mining. So it is deemed to observe measures mentioned below:

- Quarrying is recommended from top to bottom with the view of minimizing negative environmental impacts and providing safety to workers.
- Safety helmets, safety boots, individual dust masks, safety belts, air plugs (cotton piece may be used) will be provided to the specific workers, as per site conditions.
- Audible and visual signals will be used to give warning of falling stone while excavation process and some guards will be employed with red flag to control the situation in and around the safe distance of the quarrying area.
- Regular inspection of mine by technicians, experts will be organized.

8.5 POST MINING AND RECLAMATION

Mining is only a temporary activity of land use and therefore a rehabilitation program consisting of concurrent and post mining reclamation is compulsory. The reclamation of land is necessary to restore the land, to avoid sudden saturated dump slide, to maintain the aesthetic beauty and to avoid siltation and sedimentation in the downstream of the area. Reclamation will be done in the mined-out, dump and subsidence areas where stones have been excavated completely according to the mining scheme.

The reclamation of above mentioned area will be carried out in two stages by landscaping or site preparation after its filling to optimum capacity, slope correction, terrain formation and toe protection in the first stage as a physical reclamation and secondly, soil amelioration by reusing or spreading fertile top soil over the areas, vegetation and plantation should be done after physical reclamation because biological reclamation takes time. The related experts will select the species of plant for the reclamation depending upon the local conditions. However the plantation over the slope and cultivation over the flat land that will be developed at the bottom of the proposed quarry.

Therefore, mining and reclamation will go side by side and we eco-friendly mining will be maintained in the quarry.

CHAPTER 9

CONCLUSION

Study area comprises of mainly two rock types sandstones and limestone, while going from south to the north from outcrop on a road from Adeshwar–Kallabari to the right bank of the Jughe Khola. Sandstone is the rock type of the Tistung Formation while limestone is the main rock type of Chandragiri Limestone. Basically in the study area consist of dominantly well bedded, highly jointed, light grey to dark grey limestone giving yellow to brown weathering color. Within the outcrop at the quarry site rocks can be further subdivided into coarse crystalline, medium grained and argillaceous limestone. Beds generally dip towards north but disturbed slightly on the top of the Adeshwar–Kallabari ridge.

Petrographically on the basis of composition and texture, three different varieties of rocks have been distinguished. These are crystalline limestone, siliceous limestone and calcareous siltstone. Crystalline limestone contains more than 50% carbonate content, uniformity of grain size and loss of original texture. Siliceous limestone contains 50–70% total carbonate content and grain size is equal to that of the sand (0.060–2.00 mm). Calcareous siltstone contains 10–50% total carbonate content and grain size ranges from silt to that of the sand (0.002–2.00 mm).

Flatness ratio (p) and elongation ratio (q) ranges from 0.65 to 0.79 and 0.57 to 0.71 respectively. Shape factor ranges from 0.99 to 1.40 and high sphericity (0.82 to 90) indicates that aggregates are oblate biaxial to prolate–triaxial. Roughness index greater than 1 (1.28 to 1.40) and Roundness index moderate (52.65 to 54.77) and FI indicating only few flat grains shows that aggregates have a good workability as a road and concrete aggregates.

Water absorption value and dry density ranges from 0.19 to 0.67 and 2.5 to 3.0 gm/cm^3 which is quite low and lies with in the suggested ranges of BS, ASTM and NRS standard which indicates that aggregates have low effective porosity and strong for pavement construction as well as concrete aggregate.ACV, AIV and LAA ranges from 22 to 26%, 10 to 14% and 28 to 30%respectively which means aggregates are strong enough to resists fracture under the compressive load as the crushed fraction is low.SSV with in the acceptable range of $<$12% for road base and sub–base after 5-cycles indicates that the aggregate samples sound and resistance against chemical weathering and frost susceptibility.

Total reserve of limestone deposit at the quarry was found 2642213.6 tonnes. Semi mechanized opencast mining method will be adopted depending upon the size of the deposits and its topographic position rather than extracting haphazardly.The deposit is estimated to run for 26 years with the production rate 122241.6 tonnes per annum, considering the mining parameters of bench height 3 m bench width 3m,working bench slope 60^0 ultimate pit slope 60^0.

Highly fractured, slight to moderately weathered, medium to thick bedded rock masses with prominent 2 sets of joint parallel and perpendicular to the bedding planes make the extraction of rock easy during mining.

The existing environmental condition of Adeshwar area is not disappointing and the deposit is being quarried at local scale so, employment of EIA facilitates the mine to run smoothly.

Therefore, the total deposit of the aggregates in the study area is about 10 times more than that of the present quarry site which indicates that aggregates when extracted efficiently improving the mining techniques much more aggregates can be produced to fulfill the increasing demand of the market.

REFERENCES

Acharya, K.K. and Dhital, M.R., 2006, Geology and structure of Raniban–Champadevi area, Kathmandu Nepal, Journal of Nepal Geological Society, vol. 33, pp. 59–71.

Anon, 1991, Rock color chart, Geological society of America (Munsell color).

ASTM, 1979, Standard method of test for triaxial compressive strength of undrained rock core specimen without pore pressure measurements. American Soc. Test material, Annual Book of ASTM Standards, 632p.

ASTM, 1981, Test for resistance to degradation of large size coarse aggregate by abrasion and impact in the Los–Angeles Machine.

ASTM, 1994, Standard specification for transportation materials and methods of sampling, Washington D.C.

ASTM, 1996, Annual Book of ASTM Standards,' Concrete and aggregates", vol. 4.02.

Bajarnason, G., Petursson, P., and Erlingsson, S., 2000, Aggregate resistance to fragmentation, weathering and abrasion. Unbound Aggregates in Road construction (UNBAR). Reykjavik, 62p.

Barton, N. and Chouby, V., 1977, The Shear of Rock Joints in Theory and Practice, Vol. 10, pp. 1–54.

Bieniawski, Z.T., 1973, Engineering classification of Joint Rock Mass, vol. 15, pp. 335–343.

BS 812, 1985, Specification for constituent materials and mixes.

Clark, A.R. and Walker, B.F., 1977, A purposed scheme for the classification and nomenclature for use in the Engineering description of Middle Eastern Sedimentary rocks. Geotechnique Vol. 27, pp. 93–99.

Croney, D. and Croney, P., 1991, The design and Performance of Road Pavements, London, pp.12–246.

Dhakal, G.P., Kodma, J., and Goto, J., 2006, Freezing–Thawing effect and slake durability of some rocks from cold regions of Nepal and Japan, Nepal Geological Society, vol.33, pp. 45–54.

DOR, Nepal Road standard, 1970 (2027), Report of Ministry of Road and Transport, pp. 1–15.

DOR, 2001, Standard Specifications for Road and Bridge works, Reports of Ministry of Physical Planning and Works, pp. 600–1200.

DOR, 2006/07, Statistics of Strategic Road Networks, pp. 3–48.

Folk, R.L., 1959, Practical Petrographic Classification of Limestones, Bulletin of the American Association of Petroleum Geologist, vol. 33, pp. 2–38.

Franklin, J.A., Broch, E. and Walton, G., 1971, Logging the Mechanical Character of Rocks, Vol. 80, Bull. 770, pp. A_1–A_9.

Geological Society Engineering Group working party report, 1977, The description of rock masses for Engineering purposes. Quaterly Journal of Engineering Geology, Vol. 10, pp. 355–388.

Holtz, R.D. and Kovacs, W. D., 1981, An introduction to Geotechnical engineering, Eaglewood Cliffs, New Jersy, Prentice–Hall, Inc.

ISRM, 1979, Suggested methods for determining water content, porosity, density, water absorption and related properties and swelling and slake –durability index properties. Intl. Soc. Rock, Mech. Min. SCi. and Geomech. Abstr., Vol. 16, pp. 141–156.

Janno, V.C., 1998, Quantification of Shape, Angularity, and surface texture of Base Course Materials, US Army Crops of Engineers, Special Reports 98–1, pp. 1–22.

Johnson, R.B. and Degraff, J.V., 1998, Principles of Engineering Geology, John Wiley and sons, Inc. Canada, pp. 497.

Kaplan, M.F., 1961, Crack propagation and fracture of concrete, Journal American concrete Institute, vol.58, pp. 509–610.

Khanna, S.K. and Justo CEG., 2004, Highway and Material Testing a Laboratory Mannual, Nemchand and Bros, Roorkee Press, pp. 2–94.

Maharjan, D.K. and Tamrakar N.K., 2003, Quality of Siltstones for Concrete Aggregates from Nallu Khola Area, Kathmandu Nepal, Geological Society, vol. 30, pp. 167–176.

Maharjan, S. and Tamrakar N.K., 2007, Study of Gravel of the Narayani and Rapti River for Construction Materials, Bulletin of Department of Geology T.U., vol. 10.

Nepal Standard, 1994, Specification for aggregates (NS–297–2050) U.D.C N620, pp. 113.

Neville, A.M., 1996, Properties of concrete, Fourth edition, Addision Wesley Longman, Essex, England.

Smith M.R. and Collis L., 1993, Sand, Gravel and Crushed Rock Aggregates for Construction Purpose, Geological Society London, special publication, vol. 9, pp. 5–263.

Stocklin, J. and Bhattarai K.D., 1980, Geology of Nepal and Its Regional Frame, Journal of the Geological society of London, vol. 137, pp. 1–34.

Tamrakar, N.K., Yokota, S. and Nakamura, M., 1999, Some mechanical properties of the Siwalik sandstones and their relation to petrographic properties, Geoscience Report of Shimane University, vol. 18, pp. 41–54.

Tamrakar, N.K. Yokota S. and Shrestha S.D., 2002, Physical and geomechanical properties of the Siwalik sandstones, Amlekhjung–Suparitar area, Central Nepal Himalaya, Journal of Nepal Geological Soceity, vol. 26, pp. 59–71.

Tamrakar, N.K., Yokota, S. and Shrestha, S.D., 2007, Relationship among mechanical, physical and petrographic properties of Siwalik sandstones, Central Nepal Sub–Himalayas, Engineering Geology, Vol. 90, pp. 105–123.

Thapalia, A., 2005, Study of the quality assessment, reserve estimation and mining techniques of dimension stones of Khorpa deposits, Central Nepal. Dissertation submitted to Central Department of Geology, Tribhuvan University, pp. 65.

Transportation Research Record, 1990, Design and evaluation of Rigid and Flexible Pavements, Record number 1286, pp. 7–11.

ANNEX

Table 1: NRS classification of roads based on traffic flow

Classification	Types of carriage way	Types of Topography	Transportation Units(T.U)/day
Class IAA	Four lanes divided by 2×2×3.5 meters with central median; Asphalt concrete or cement concrete (complete control of access)	Level Rolling Mountainous	7000 5000 3000
Class IA	Two lanes 2×3.5 meters; Bituminous premixed wearing course(partial control of access)	Level Rolling Mountainous	3000 2500 1500
Class I	Two lanes 2×3.5 meters; Surface treatment	Level Rolling Mountainous	1500 1000 300
Class II	Single lane 3.5 meters; Surface treatment	Level Rolling Mountainous	300 150 75
Class III	Single lane 3.5 meters; Gravel	All topography	75

Table 2: Weathering and alteration grades after Geological Society (1977) and after Fookes et al (1971)

Term	Grade	Description	Material characteristics
Fresh	I	No visible sign of weathering	Aggregate properties not influenced by weathering. Mineral constituents of rock are fresh and sound.
Faintly weathered	IB	Discoloration on major discontinuity surfaces	Aggregate properties not significantly influenced by weathering. Mineral constituent are sound.
Moderately weathered	II	Discoloration indicates weathering of rocks materials and discontinuity surfaces. All the rock materials may be discolored by weathering and may be some what weaker than in its fresh condition.	Aggregates properties may be significantly influenced by weathering. Strength and abrasion characteristics show some weakening. Some alteration of mineral constituents with micro constituent with micro cracking.
Moderately weathered	III	Less than half of the rock material is decomposed and/or disintegrated in to a soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.	Aggregate properties will be significantly influenced by weathering .soundness characteristics markedly affected. Alteration of mineral constituents' common and much micro cracking.
Highly weathered	IV	More than half rock material is decomposed and/or disintegrated in to a soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.	Not generally suitable for aggregate but may be suitable for lower part of road pavement and hardcore.
Completely weathered	V	All rock material is decomposed and /or disintegrated in to soil. The original mass structure is still largely intact.	Not suitable for aggregate, or pavement but may be suitable for select fill.
Residual soil	VI	All rock materials is converted to soil .The mass structure and material fabric are destroyed .There is a large change in volume but the soil still as not been significantly transported.	May be suitable for random fill.

Table.3: Definition of degree of induration (Larsen et al.1995)

Induration	Description	Properties
H1	Unindurated, loose, soft	The materials can without difficulty be removed with fingers. Grain rich material fall apart in dry condition.
H2	Weakly indurated	The material can easily be cut by a knife and scratch by a nail. For grain rich material, grain can be detached with a knife.
H3	Indurated	The materials can be shaped by a knife, but cannot be scratch by a nail. From grain rich materials, grain can be detached with a knife.
H4	Strongly indurated	The materials can be scratched by a knife, but single grains can not be detached with a knife. Fractures follow the gain contact.
H5	Very strongly indurated	The materials can not be scratched by a knife. Fractures passes through the grains.

Table.4: Minimum testing frequency for a process control according to standard specification of road and bridge work (2001)

Tests	One test in every	Minimum no.of tests per section
Materials		
Gradation	$200m^3$ or part of it and change in source	2
Plasticity index	" " " "	2
Flakiness index	" " " "	2
Maximum dry density and Optimum moisture content	$1000m^3$ " " "	2
Los Angeles abrasion	$200m^3$ " " "	-
Aggregate impact value	" " " "	-
Sodium sulphate soundness	$500m^3$ " " "	-
Crushing ratio	$200m^3$ " "	-
CBR	$500m^3$ " " "	-
Field density and moisture content	$500m^2$	2

Table 5: Compaction test for determination of OMC and MDD

Volume of the mould (v_m)cc =15733.824

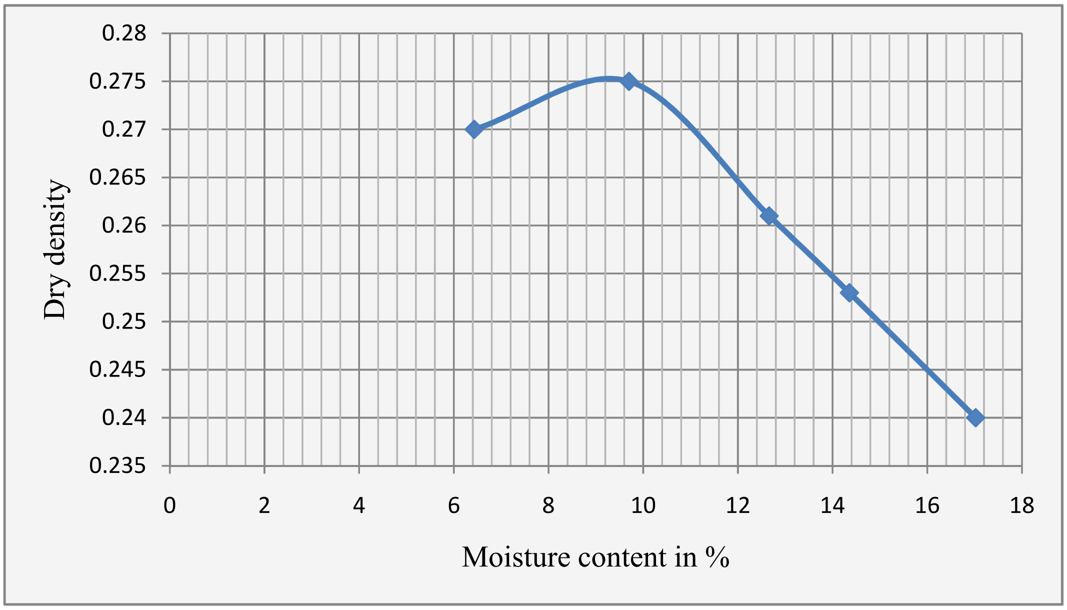
Weight of hammer =2.6 Kg

Amount of compaction =Light

Determination no.	Sample no.1	Sample no.2	Sample no.3	Sample no.4	Sample no.5
Wt. of mould(w_m)	2885	2885	2885	2885	2885
Wt.of mould+compacted soil(w)	7410	7630	7523	7450	7310
Wt. of moisture container(w_1)	26.984	27.52	34.00	23.909	28.489
Wt.of container+weight soil(w_2)	121	125.14	144.61	122.7	120.4
Wt.of container+dry soil(w_3)	115.30	116.52	132.18	110.62	107.03
Wt.density γ_m=w-w_m/v_mg/cc	0.2875	0.3015	0.2947	0.2901	0.2812
Moisture content W%=w_2-w_3/w_3-w_1	6.430	9.697	12.661	14.355	17.023
Dry density γ_d=γ_m/1+w/w_m g/cc	0.2702	0.2750	0.2617	0.2536	0.2403

Plotting on the graph,

Moisture content on the x-axis and dry density on the y-axis



From the graph,

Maximum dry density = 0.275 g/cc

Optimum moisture content = 9%