EVALUATION OF PREMATURE FAILURE OF FLEXIBLE PAVEMENT IN HILLY AREA OF MIZORAM

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

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EVALUATION OF PREMATURE FAILURE OF FLEXIBLE PAVEMENT IN HILLY AREA OF MIZORAM

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SUBMITTED
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SUPERVISOR'S CERTIFICATE

This is to certify that the thesis entitled "**Evaluation of Premature Failure of Flexible Pavement in Hilly Area of Mizoram**" submitted to Mizoram University for the award of the degree of **Doctor of Philosophy,Civil Engineering** by **H Laldintluanga**, Ph.D. Registration No.**MZU/PhD/1389 of 01.08.2019**, is Ph.D. scholar in the Department of Civil Engineering, under my guidance and supervision and has not been previously submitted for the award of any degree in any Indian or foreign University. He has fulfilled all criteria prescribed by the UGC (Minimum Standard and Procedure governing Ph.D. Regulations). He has fulfilled the mandatory publication (Publication enclosed) and completed Ph.D course work.It is also certified that the scholar has been admitted in the Department through an entrance test, followed by an interview as per UGC Regulation of 2016.

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DECLARATION

I, **H Laldintluanga,** hereby declare that the subject matter of this thesis entitled **"Evaluation of Premature Failure of Flexible Pavement in Hilly Area of Mizoram"** is the record of work done by me, that contents of this thesis did not form basis of the award of any previous degree to me or to do the best of my knowledge to anybody else, and that the thesis has not been submitted by me for any research degree in any other University/Institute.

This is being submitted to the Mizoram University for the degree of Doctor of Philosophy in **Civil Engineering.**

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(H. LALDINTLUANGA)

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List of Abbreviations

ACV	=	Aggregate crushing value
ACWC	=	Asphalt Concrete Wearing Course
AIV	=	Aggregate Impact Value
ATL	=	Aizawl via Thenzawl via Lunglei
ASAs	=	Antistripping Agents
BC	=	Bituminous Concrete
BT	=	Bituminous
CRAM	=	Contained Rock Asphalt
CBR	=	California Bearing Ratio
CC	=	Cement Concrete
CR	=	City Road
CVD	=	Commercial Vehicle per day
CRM	=	Crumb Rubber Modifier
CSA	=	Cumulative Standard Axle
ER	=	Earthen Road
FWD	=	Falling Weight Deflectometer
Gb	=	Bulk Specific Gravity of Bituminous Mix
GSB	=	Granular Sub Base
GR	=	Gravel Road
Gt	=	Theoretical Specific Gravity of Bituminous Mix
HMA	=	Hot Mix Asphalt

IRI	=	International Roughness Index
LAAV	=	Los Angeles Abrasion Value
LDPE	=	Low-density polyethylene
MDD	=	Maximum Dry Density
MDR	=	Major District Road
MEPD	=	Mechanistic Empirical Pavement Design
MRAMS	=	Mizoram Road Asset Management System
MORT&H	=	Ministry of Road Transport and Highway
MSA	=	Million Standard Axles
MSS	=	Mix Seal Surfacing
NMAS	=	Nominal Maximum Aggregate Size
NNE-SSW	=	North North East- South South West
NH	=	National Highway
OGL	=	Original Ground Level
ODR	=	Other District Road
PB	=	Paver Block
PMC	=	Pre Mix Carpet
SBS	=	Styrene Butadiene Styrene
SDBC	=	Semi Dense Bituminous Concrete
SDI	=	Slake durability Index
SG	=	Specific Gravity
SH	=	State Highway
SMA	=	Stone Matrix Asphalt

STR	=	Satellite Town Road
TSR	=	Tensile Strength Ratio
Vb	=	Volume of Bitumen
VR	=	Village Road
VMA	=	Void in Mineral Aggregate
VDF	=	Vehicle damage factor
VES	=	Vertical Electrical Sounding
VFB	=	Void Filled with Bitumen
Vv	=	Volume of Void
WA	=	Water Absorption
WAV	=	Water Absorption Values
WMM	=	Wet Mix Macadam

CHAPTER 1

INTRODUCTION

1.1 Introduction

Transportation infrastructure is one of the backbones of economic development which transport agriculture products and industrial goods. The increasing traffic intensity, high tire pressure, increasing axle loads and rainfall etc. are causing early signs of distress to bituminous pavements throughout the world. The deterioration of the paved roads in tropical and subtropical countries differs from those in the more temperate regions of the world. This can be due to the harsh climatic conditions and sometimes due to the lack of good pavement materials and construction practices.

Pavement performance can be defined as the ability of the road to meet the demands of traffic and environment during its design life. The reduction in the performance level of the pavement with time is termed as deterioration. Flexible pavements deteriorate due to many factors, predominantly traffic, climate, materials, construction quality and time.

Road improvements bring immediate and at times dramatic benefits to road users such as improved access to hospitals, schools, and markets. Smooth roads also improve comfort, speed, safety and lower vehicle operating costs. Development of appropriate transportation policy and evaluation of the economic impacts also depend on the performance and interplay between the infrastructure facility and its user (traffic).

Premature failure of flexible pavement refers to the deterioration or breakdown of the pavement surface or structural layers earlier than expected. The premature failure of flexible pavement can be caused by a variety of factors, including inadequate pavement design, poor construction quality, insufficient pavement thickness, inadequate drainage, and excessive traffic loads. Other factors that can contribute to premature failure include poor materials quality, lack of maintenance, and environmental conditions such as freeze-thaw cycles or excessive heat. Proper design, construction, and maintenance practices can help minimize the risk of premature failure in flexible pavement.

Pavement failure may be considered to be either a structural, functional, or a combination of these types. Structural failure is the loss of load carrying capability, in which the total pavement collapsed or one or more component of the pavement layer had broken down and the pavement is no longer capable of sustaining the imposed load i.e. vehicular load.

Functional failure is a broader term, which may indicate the loss of any function of the pavement. In functional failure, the pavement will not carry out its intended function without causing discomfort to the road users or without causing high stresses to the vehicles plying on it due to its roughness. Unless timely proper maintenance work is done, the functional failure can become a structural failure.

1.2 Flexible Pavement

Flexible pavements consist of different types of materials such as soils, granular material, stabilised (cementitious soils), stabilised (cementitious of cemented) granular layers (subbase or base), bituminous layers (Surface, binder, base) – different types of mixes with different types of binders (unmodified and modified). Flexible pavements are designed to distribute traffic loads through the pavement structure to the underlying soil layers.

Pavement is the durable surface material laid down on an area intended to sustain vehicular load. Base on the structural behavior, pavements are generally classified into two categories, namely:

(1) Flexible pavement or bituminous pavement or black top pavement

(2) Rigid pavement or cement concrete pavement or white surface pavement.

This study focuses on flexible pavement because it makes up 99% of the road surface [1]. Flexible pavements are those which on a whole have low or negligible flexural strength and rather flexible in their structural action under load. A structure consisting of superimposed layers of processed materials above the natural soil subgrade, whose primary function is to distribute the applied load stress through a wider area on the soil sub-grade below.

Flexible pavement consists of several layers of materials, including a subgrade layer, base layer, and surface layer. The subgrade layer is the compacted soil that provides the foundation for the road which receives the stresses from the layers above. The primary functions of the sub-base course are to provide structural support, improve drainage, and reduce the intrusion of fines from the sub-grade in the pavement structure. The base layer is made of crushed stone or gravel, and provides support and stability for the surface layer. The surface layer is the top layer of the road and is made of bituminous concrete or other materials that provide a smooth and durable driving surface.

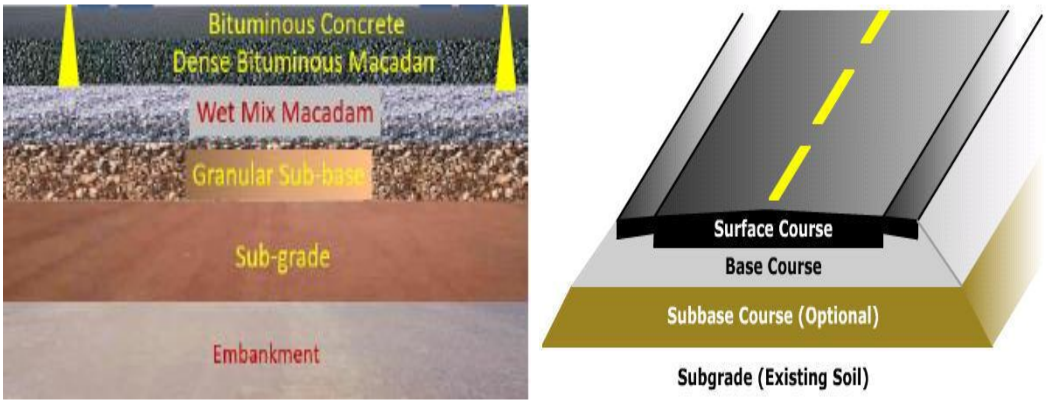

Fig. 1.1 Component of a typical Flexible pavement

(https://americangeoservices.com/flexible-pavements.html)

1.2.1 Types of flexible pavements

(a) Conventional layered flexible pavement

(b) Full - depth asphalt pavement

(c) Contained rock asphalt mat (CRAM).

- Conventional flexible pavements are layered systems with high quality materials are placed in the top where stresses are high, and low quality cheap materials are placed in lower layers.
- Full - depth asphalt pavements are constructed by placing bituminous layers directly on the soil sub-grade. This is more suitable when there is high traffic and local materials are not available.

- Contained rock asphalt mats are constructed by placing dense/open graded aggregate layers in between two asphalt layers.

Bitumen owing to its viscous nature allows significant plastic deformation. Additionally, bitumen as a material is also durable and cost effective. Because of all this reason, bitumen is commonly used material in pavement.

The function of a flexible pavement is to provide a smooth and safe driving surface for vehicles by distributing the weight of traffic loads over a wider area, reducing the stress on the underlying soil or subgrade. This is accomplished by using layers of different materials, such as bitumen and aggregate, those are designed to flex under traffic loads and distribute the stresses evenly. Additionally, flexible pavements have a higher degree of elasticity compared to rigid pavements, which allows them to absorb vibrations and impacts from passing vehicles. This reduces noise and improves ride quality for drivers.

The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Flexible pavement layers reflect the deformation of the lower layers on to the surface layer.

Load is transferred to the lower layer by grain to grain distribution as shown in the figure 1.2 and 1.3 given below;

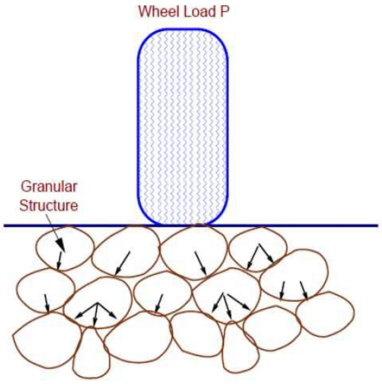

Fig. 1.2 Load is transferred by grain to grain contact

(https://www.civil.iitb.ac.in/tvm/1100_LnTse/401_lnTse/plain/plain.html

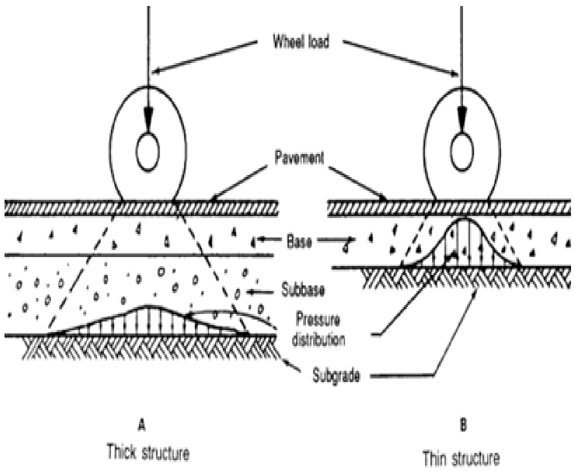

Fig. 1.3 Distribution of Pressure under wheel load

(https://theconstructor.org/transportation/factors-affecting-pavement-design/12849//306914416)

1.3 Need of Study

The road in Mizoram, a hilly region, could not last for very long despite significant expenditure in its construction. The materials used in road construction that fail quickly after being exposed to the weathering and wheel traffic are not suitable for pavement. Pavement that develops rutting, potholes, ravelling, and shoving on the pavement surface within three years has a major problem.

It has been long necessary to construct long lasting or durable road in a hilly area with challenging circumstances such as steep terrain, high intensity of rainfall and poor quality of road construction materials. Valid technical evidence based on research is much required to address this problem.

The rocks available in Mizoram are mostly shale, siltstone and silty sandstone. They are found intercalated with these stone. Due to scarcity of the strong aggregates, aggregates have to be taken from far away and even from outside the state which incurs a huge transportation costs. The majority of the sedimentary rocks in Mizoram are solid, but they can easily degrade when subjected to wheel loads and moisture. A study on subgrade, aggregate and bituminous concrete is needed in such a way that effect of materials, moisture, method and design of construction on the pavement structure can be known. In summary, the study of premature failure of flexible pavement is important for ensuring safety, reducing costs, minimizing environmental impact, and promoting sustainability in road construction.

1.4 Road condition of Mizoram

India total road network has the world's second longest road network spread over the entire states of the country. Among this, Mizoram has 7286.38 km of roads, which consists of 0.12% of India's total roadways. The large variation in terrain, climatic, environmental, soil and traffic characteristics of Mizoram roads, differentiate it from other roads in the country. In the case of Mizoram roads, flexible pavements contribute the major portion and are subjected to intense rain and traffic. The economic loss due to poor riding quality, early deterioration of pavement is huge in the state.

It is observed that about 65% (3172.5 km) of the road network excluding National Highway is paved i.e. Bituminous (BT), Cement Concrete (CC) and Paver Block (PB) and about 35% (1721.2 km) is still unpaved i.e. Earthen (ER) and Gravel (GR).

The distribution of road surface by each road category is presented in the Table 1.1. It is observed that higher category roads such as National Highway (NH), State Highway (SH), Major District Road (MDR), City Road (CR), Satellite Town Road (STR) have mostly paved surface i.e. 82% share of the respective road category length. However, for lower category roads such as Other District Road (ODR) and Village Road (VR), the distribution of paved and unpaved road network are almost equal i.e. 50% of the respective road category length.

Table 1.1 Distribution of Road Surface by Road Category

(MPWD, Mizoram Road Asset Management System (MRAMS), Data Completion Report June, 2020)

Road category	Road surface (km)					Total Length (km)
	BT	CC	PB	ER	GR	
NH (CPWD+BRO)	1322.5				88	2392.58
SH	233.3	2.3	0	7.8	58.1	301.5
MDR	437.5	11.2	0	20.2	96.5	565.4
ODR	614.4	12.4	0	38.9	202	867.7
CR	248.6	24.6	2.4	3.6	7.5	286.7
STR	541.8	87.7	6	77.8	56	769.3
VR	741.2	209.1	0	483.1	669.8	2103.2
Total	**4139.3**	**347.3**	**8.4**	**631.4**	**1089.9**	**7286.38**

Aizawl Road South, Aizawl Road North and Kolasib divisions have more than 80% paved road network. On the contrary, Champhai, Saiha and Lawngtlai divisions have more than 50% of unpaved roads.

The low traffic volume has been observed throuhout Mizoram. Majority of the road network in Mizoram carry less commercial vehicles ranging up to 100 CVD only, i.e. around 78% of the road carries a commercial traffic within a range of 100 CVD [1]. Only 5% of traffic has commercial vehicles more than 500 CVD. The Axle load survey data shows that average loading of goods vehicles are within the permissible limits for all the categories of commercial vehicle [1].

Only 4% of road length has roughness of <4 IRI which are in a good condition. Another 31% of the network is within 4 – 10 IRI which may need a resurfacing or overlay depending on traffic loading. Around 65% of the road length has IRI >10 which may need major maintenance or reconstruction.

It is observed that about 69% of the surveyed roads have deflection values less than 1.0 mm which are structurally sound. About 29% have deflection values ranging between 1-1.5 mm where a thinner overlay may be required to extend the pavement life. Only 2% of the road stretches has high deflection values (>1.5 mm) suggesting very weak pavement [1].

1.5 Study Area

The state highway-I in Mizoram (also called as ATL Road) is considered a priority road connecting Aizawl and Lunglei district of Mizoram with a length of 164 km. The climate of the study area is characterized by a humid tropical climate with cool summers and cold winters. In winter, the temperature varies from 8^0C to 24^0C, while in summer; it is between 18^0C to 32^0C. In Mizoram, the amount of rainfall is reasonably high. However, a shortage of water is often experienced in the post-monsoon season because most of the water available is lost as surface runoff. The study area receives heavy rainfall from June to September, with an average annual rainfall of 2,794 mm under the influence of the southwest monsoon [2]. Since the study areas encounter high intensity of rainfall over 2000 mm/year, which may be the contributing factor to the early deterioration of the pavement. Details of the different locations considered under study area including location coordinates, section condition, span of failure zone and distress type is tabulated in Table 1.2.

This study included Mel 5 a 'Good/ Sound Section' for comparison to other premature failure sites. The good section is included to provide an understanding of how good road conditions are influenced by geological and hydrological factors. Figure 1.4 shows the map of the study area.

Indian Remote Sensing Satellite Quickbird, (IRS-P6) LISS III data having spatial resolution of 23.5m and Cartosat-I stereo-paired data having spatial resolution of 2.5m were used as the main data to plot the geological map of study area. Survey of India (SOI) topographical maps and various ancillary data were also referred in the development of the map. The main road connecting Aizawl city and Lunglei (study area) i.e 160 km long was buffered 500 m on both sides. In a GIS environment, the geological lithology and structure were plotted.

Table 1.2 Location of the test site

Name of the sample	**Location coordinates**		**Section Condition**	**Span of failure**	**Distress Type**
	N	**E**			
Mel 5 (7km)	N23^{0}40'9.94"	E92^{0}43'31.41"	Good	Absent	-
Hualngo (12 Km)	N23^{0}39'23.81"	E92^{0}43'39.28"	Very Poor	90 m	Rutting, Crack, Pothole
Falkawn (16 Km)	N23^{0}37'12.67"	E92^{0}43'22"	Very Poor	120 m	Rutting, Crack, Pothole
MCON (17 Km)	N23^{0}37'02.04"	E92^{0}43'08.17"	Very Poor	100 m	Rutting, Crack, Pothole
MLT (19 Km)	N23^{0}36'03.81"	E92^{0}43'07.46"	Very Poor	85 m	Rutting, Crack, Pothole
Pukpui (150 Km)	N22^{0}57'48.45"	E92^{0}44'14.14"	Very Poor	200 m	Rutting, Crack, Pothole
Pukpui (154 Km)	N22^{0}57'59.27"	E92^{0}43'52.18"	Very Poor	180 m	Rutting, Crack, Pothole

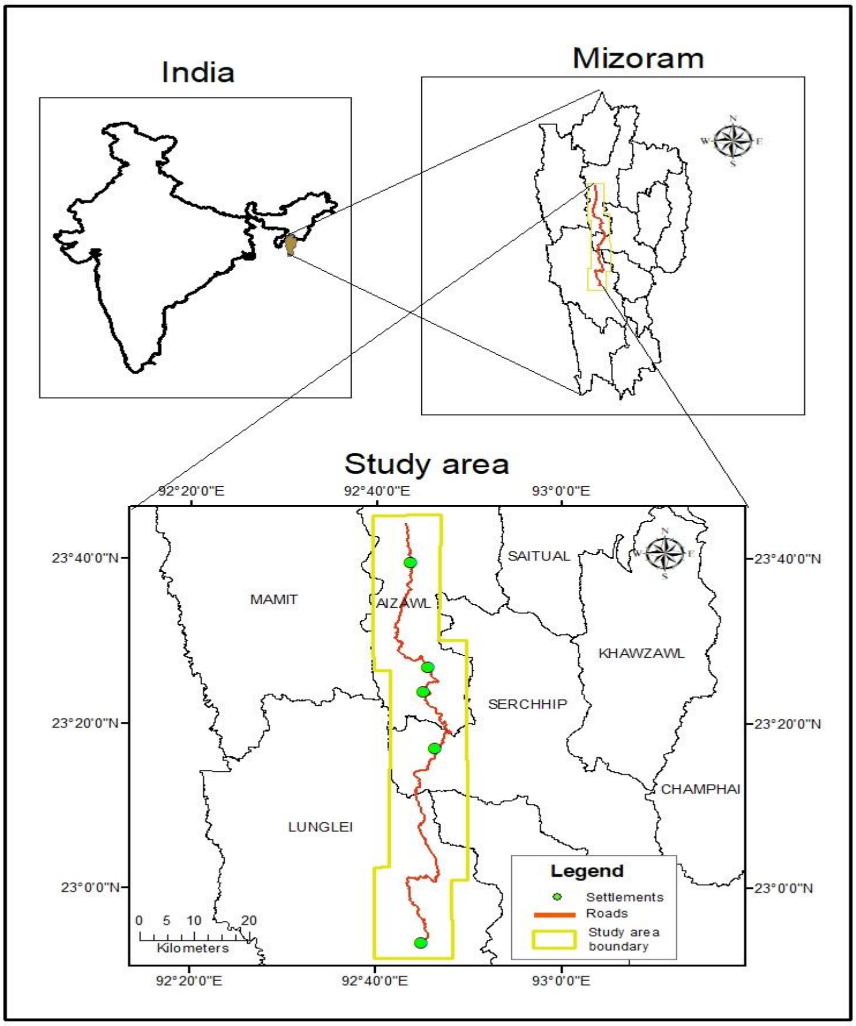

Fig. 1.4 Map showing the study area

1.5.1 Geology of the Study Area

The study area is within Bhuban Formation of Miocene age and comprises of shale, siltstone and sandstone as shown in the figure 1.5. The study area is occupied by shale, siltstone, and sandstone of Surma Formation of Miocene age. The average strike of the bedding is NNE-SSW, with dips varying from 40° to 50° towards both east and west. The sediments are folded into close to open asymmetrical anticlines and synclines along the N-S axis. Faults present in the area are longitudinal, transverse and oblique types affecting the folded sequence. The significant faults are longitudinal strike faults along the crest of the folded beds [3, 4].

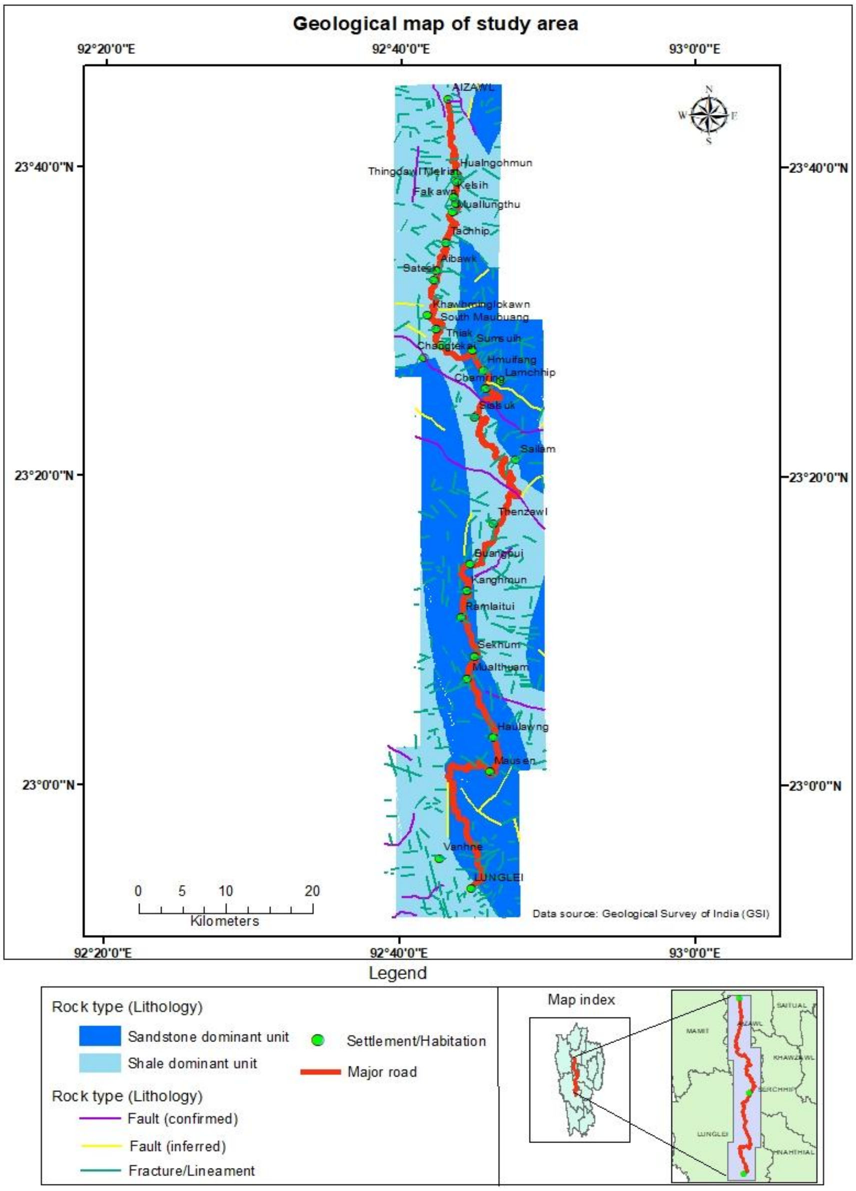

Fig. 1.5 Geological Map of the study area

The terrain in the study area is mountainous with prominent relief as shown in the figure 1.6. The topography and physiographic expression of the study area is represented by parallel to sub-parallel hill ranges trending North-South direction. Parallel to sub-parallel anticlinal hill ranges, synclinal narrow valleys form deep gorges. The western limbs of the anticlines are steeper than the eastern limbs. Basically, these are structural hills. The process of denudation and weathering is continuing in response to various natural forces. The hills are steep and separated by rivers that flow either to the north or to the south creating deep gorges. The terrain exhibit a very immature topography. Based upon lithology, relief, drainage, and structural pattern, the area has been divided into two major units, denudo-structural hills and valleys. Denudo -structural hills occupy a major portion of the study area, predominantly argillaceous comprising shale, siltstone and mudstone, fine-grained and compact sandstone with occasional limestone. The major form has been further divided: Low linear ridges, Moderate linear ridge and valleys. Moderate linear ridges occupy about 90% of the district. The main constituents are hard and compact sandstone, shale and siltstone, alterations of Bhuban Formation [5].

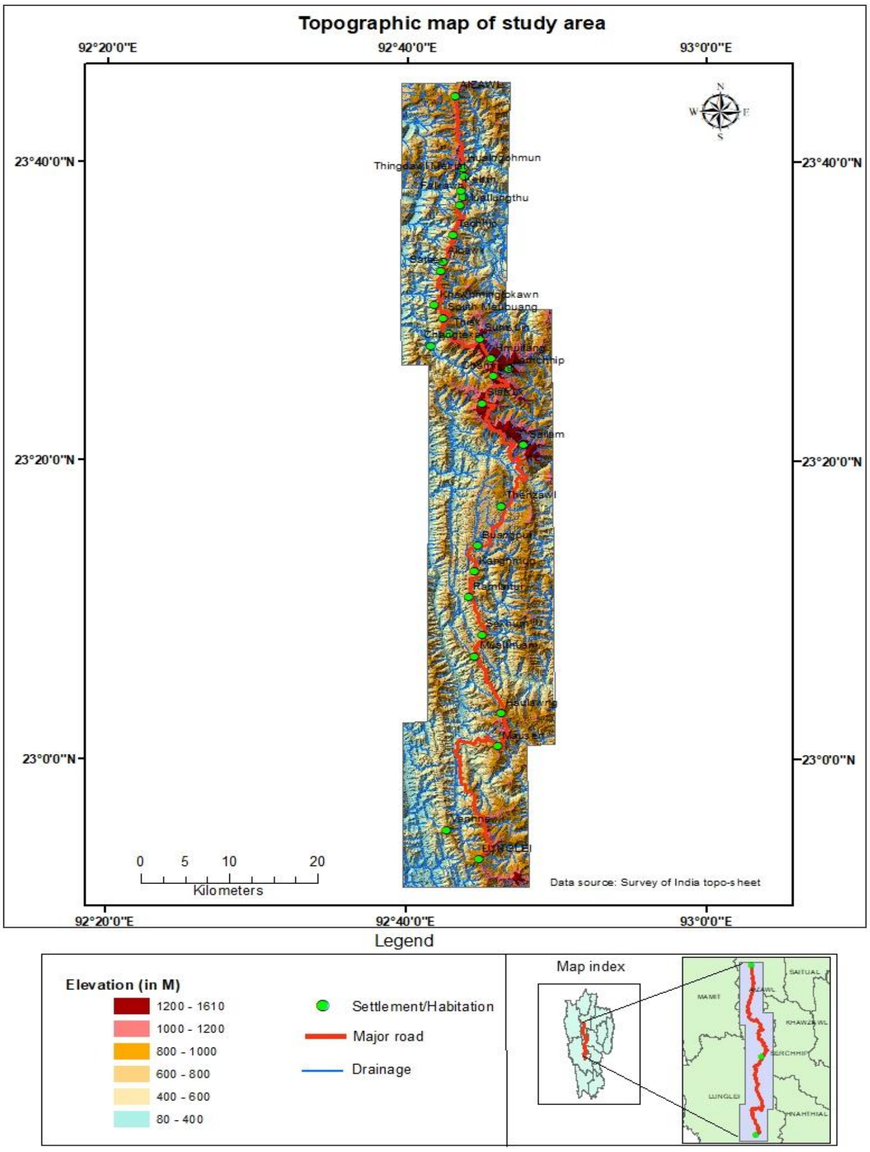

Fig. 1.6 Topographical Map of the study area

1.5.2 Condition of the flexible pavement in the study area

Aizawl - Thenzawl - Lunglei Road is the state highway constructed with the credit from the World Bank. Since its completion, maintenance works and resurfacing works had been done from time to time. However, portion of the road are found to have shown premature failure in spite of maintenance works done to the pavement.

Flexible pavements have been the preferred choice for pavement construction because of low initial cost as compared to the rigid pavements. Even though, the most severe damage to pavement is caused by axle loads, a very high percentage of pavement damage in Mizoram is water related distress during monsoon in bituminous road.

A pavement functional condition test was conducted in 2020 after 10 years of construction. It was observed that IRI values were more than 4 exceeding the permissible limit at different sections using Hawkeye 1000 Laser Profiler. Based on a traffic volume count, it was found that Cumulative Standard Axle (CSA) is 4.11 msa (million standard axles). FWD (Falling Weight Deflectometer) was used to assess the structural strength of the pavement by measuring deflection, layer thickness, and material properties. The average Deflection value obtained along the state highway is 0.77 mm. An axle-load survey is carried out to estimate the axle-load distribution of commercial vehicles using the road [6]. Vehicle damage factor (VDF) or equivalent standard axle load is calculated from axle load survey data and found to be 0.64 which is lower than the indicative value of VDF 1.7 for hilly terrain as per IRC 37-2018 [7]. Due to the low VDF and less number of traffic (4.11 msa) wheel load is not a factor causing premature pavement failure. The state highway was designed with Cumulative Standard Axle (CSA) 3 msa (million standard axles) and CBR 6% at the time of construction. There were numerous shady and damp stretches along the road length that received little or no sunlight at all. This posed a huge problem for the construction of pavement as sub-grade would remain damp even into the working season.

There are also stretches that show functional failure but would sooner than later advance to structural failure unless proper maintenance is given in time.

(a) Chainage 10 km

(b) Chainage 12 km

(d) Chainage 15 km

(e) Chainage 16 km

(f) Chainage 120 km

(g) Chainange 127 km

(h) Chainage 137 km

(i) Chainage 140 km

Fig.1.7 Some of the pavement condition in the study area (a), (b),(c),(d), (e),(f),(g),(h),(i)

1.6 Objectives

The main aim of this study is analysing the cause of the premature failure so that the pavement could last the design life. The following objectives are framed to achieve the aim:

1. To identify the location having premature failure of flexible pavement.
2. To characterize prematurely failed flexible pavement structure.
3. To analyse the causes of premature flexible pavement failure.
4. To develop suitable methods for preventing the premature flexible pavement failure.

1.7 Methodology for evaluation of premature failure of flexible pavement

Premature failure of flexible pavement can be caused by a number of factors, including inadequate design, poor construction practices, insufficient materials, improper quality control, and environmental factors such as temperature and moisture changes. A systematic approach should be taken to identify the root cause of the failure and develop a plan to address it. Here are the steps taken:

Data Collection: Data has been collected on the pavement construction history, traffic volume, pavement structural and functional condition, climate conditions, and other relevant factors that may have contributed to the premature failure.

Identification of premature failure location: The location of premature failure site has been identified using visual inspection of the pavement surface, looking for signs of distress such as cracking, rutting, and surface deformations. Take measurements of the distress and record their locations.

Conduct testing: Laboratory tests have been performed on samples of the pavement materials of the selected site to determine their properties and quality using Aggregate test, Bitumen test, Water sensitivity test, Electrical Resistivity and Asphalt Concrete test. The rock and bituminous concrete degradation under the influence of water had been also checked.

Analyze data: The collected data has been analyzed from the inspections and testing to identify the possible causes of the premature failure. Statistical analyses, hydrogeological survey, multilayer elastic program - IITPAVE, IP2Win software were used.

Develop a model: Based on the analysis of the data, model has been developed to address the root cause of the failure. These include modifications to the design or construction methods, or changes to maintenance practices.

1.8 Organization of the Thesis

Chapter 1 defines the role of road transportation infrastructure in country economy. It introduces the current road network of Mizoram and the condition of the study area. The various types of flexible pavement, the possible cause of premature failures have been highlighted. The need for finding the causes and analysis of the cause of the premature failure of flexible pavement are briefly discussed. The aim and objectives of the research study are given.

Chapter 2 highlights the factor which causes premature failure such as weak subgrade, inadequate drainage, stripping, dust coating of aggregate, moisture, axle load, improper design and weak materials. The failure of pavement influenced by geological and hydrological factors is also summarized. The prevention of premature failure pavement using anti stripping agents such as lime, modified binders and introduction of appropriate construction method, and construction materials is also presented.

In Chapter 3, presents geotechnical investigation on subgrade soil to determine the influence of subgrade soil on the premature failure of road in a hilly area where rainfall intensity is very high. The value of permeability, CBR value, type of soil, water saturation level and geometry of the pavement that affects in the performance of the pavement has been discussed. Effect of road cutting at a differential level on sloping terrain on soil strength across the transverse section is presented.

In **Chapter 4**, Weakening of subgrade, sub-base and base by ground water movement is one of the important factors leading to pavement failure. In relation to this, impact of hydrogeological and geotechnical factors on pavement performance has been examined in this study. The study area has experienced premature failure at multiple locations. Seven most critically affected areas having failure span length ranging from 85 m to 200 m were chosen and geophysical, geological survey and geotechnical tests were conducted at affected stretches to identify the cause of failure.

In **Chapter 5**, Sedimentary rocks from different quarries were evaluated to determine the influence of aggregate size on the degradation of materials. The effect of rock size on physical parameters of aggregate used in flexible pavement is investigated. The size-dependent performance of rock with regard to physical properties is the focus of the study. The possibility of adoption of larger size aggregate in the pavement base course is studied to withstand the effect of environmental action and wheel load, especially on the friable and quickly disintegrated rock. Development of aggregate degradation rating value and index is also presented.

In **Chapter 6**, Plastic influence on different properties of laminated sedimentary aggregate (rock) and the bituminous mix in pavement has been examined. Plastic wastes (low-density polyethylene) can be used to strengthen the properties of laminated sedimentary aggregates (rock) and bituminous concrete as well as to prevent the early degradation of rock inside the bituminous concrete. The water susceptibility of the bituminous concrete has been also studied.

In **Chapter 7**, Attempt has been made to study the effect of material properties, loading conditions and pavement thickness on response value. The response of the materials in terms of stress, strain and deflection induced by wheel load has been calculated using IITPAVE. The pavement construction model has been developed for highly degradable rock that is susceptible to high intensity of rainfall and water seepage.

Finally in **Chapter 8**, a summary of the work and the conclusions of all the investigations that have been carried out in this thesis are presented. Some recommendations for future scope of research in this area are also given in this chapter.

CHAPTER 2
LITERATURE REVIEW

2. Introduction

The premature failure of flexible pavement refers to the deterioration or breakdown of the road surface and underlying layers sooner than expected, considering the design life and traffic loadings. Flexible pavement is a type of road structure commonly used in many areas, comprising multiple layers of materials that are designed to distribute traffic loads and provide a smooth and durable driving surface. Premature failure of flexible pavement can have various causes, and addressing these issues is crucial to maintain safe and functional roadways.

2.1 Premature failure due to moisture induced damage:

Moisture damage in hot mix asphalt pavements can be defined as the loss of strength and durability due to the effects of moisture [8] caused by loss of cohesion (strength) of the asphalt film, failure of the adhesion (bond) between the aggregate and asphalt, and degradation of the aggregate particles subjected to freezing [9]. Moisture damage is commonly manifested in the form of stripping as a result of detachment, displacement, spontaneous emulsification, pore pressure, and hydraulic scour [10-17].Moisture damage generally starts at the bottom of an asphalt base layer or at the interface of two asphalt layers. Eventually, localized potholes are formed or the pavement ravels or ruts. With hardened binders, localized fatigue cracking (longitudinal cracking that progresses to alligator cracking) may occur resulting in a weakened pavement structure [18].

Subsequent water intrusion into these localized water-damaged areas, coupled with traffic loading, further degrades the structural integrity of the pavement layer, and possibly the underlying layers which, if not repaired, can lead to substantial localized failure of the pavement structure. Surface ravelling or a loss of surface aggregate can also occur, especially with chip seals. Occasionally, binder from within the pavement will migrate to the pavement surface resulting in flushing or bleeding [19].

The effect of water on Marshall Stability of asphalt concrete wearing course (ACWC) made with a latex-bitumen binder. The results indicated that the addition of up to 4% latex to ACWC mix increased the retained Marshall stability, whereas the addition of latex above 4% decreased the retained stability of the mixture. The addition of 4% Crumb Rubber Modifier (CRM) significantly improved the retained stability of the mixture and was the best latex – ACWC mix [20].

Moisture damage can be minimal if stripping is restricted only to the coarse aggregate. If the fine aggregate strips, severe damage can occur because the fine aggregate constitutes the basic matrix of the mixture [21]. Moisture damage generally results in the loss of strength of the mixture because of two main mechanisms: the loss of adhesion between the asphalt binder and the aggregate and the loss of cohesion within the mixture. The combination of mineralogical constructed aggregates is very important in the occurrence of moisture based on their hydrophilic and hydrophobic properties and their relationship with the asphalt binder [22]. One of the common methods of improving the strength of the surface against moisture is to cover the surface by using materials with hydrophobic properties [23].

2.2 Premature failure due to axle load, improper design and weak materials:

Hot Mix Asphalt (HMA) mixtures manufactured with aggregates having soft particles, clay lumps, and excessive dust are particularly prone to moisture-related damage [24]. A number of researchers have noted that the vulnerability of unbound granular pavement performance is being heightened by increasing vehicle axle loads, limited material resources and loss of engineering design skills [25-27].

2.3 Premature failure due to inadequate drainage, stripping, dust coating of aggregate:

While stripping is often related to the presence of moisture in the HMA, there is general agreement that many moisture related issues can be solved through the use of suitable paving materials and by designing the pavement with drainage as a priority. Adequate pavement drainage must be employed to avoid stripping problems in asphalt pavements. Case studies have shown that stripping can be a localized phenomenon which occurs primarily in locations which are oversaturated with water/water vapour due to inadequate subsurface drainage conditions [28].

During production and design, it is important to ensure that the aggregates used are of good quality and do not exhibit weak and friable properties. It is also necessary that the aggregate is sufficiently dry as moist aggregate provides an excellent starting point for later stripping problems. Dust coated aggregate should be avoided as there is the possibility that if the dust is a clay it might act as an emulsifier. As clay tends to expand in the presence of water, there is the possibility that the asphalt film might be lifted off the surface of the aggregate due to the expansion of the clay [29].

Construction practices play an important role in moisture sensitivity. Inadequate compaction of the Hot Mix Asphalt (HMA) mat is probably the most common construction- related factor related to stripping. If the design air voids are not met during the compaction stage, excessive water may enter the pavement structure thus contributing to hydraulic pore- pressures build-up caused by traffic [30].

To prevent moisture susceptibility, proper mix design is essential. Many ways to prevent stripping in a pavement, the use of antistripping agents (ASAs) is the most common [31,32]. One of the most commonly used ASAs in the United States is hydrated lime. Others include liquid ASAs, such as amines, diamines, liquid polymers, and solids, such as, for example, Portland cement and fly ash [33].

2.4 Premature failure due to subgrade soil and fillers:

Research shows that the mineralogical composition and shape of filler materials can alter the behaviour of asphalt mixtures. Moisture damage, largely influenced by the filler part of aggregates is among the main factors affecting the durability and performance of asphalt pavements. Moisture damage contributes to the premature deterioration of flexible pavements [34].

Various studies have shown that the properties of mineral fillers (particles passing a 0.075-mm sieve) have a significant effect on the performance of asphalt concrete pavements [35,36]. Pavement engineers often improve the moisture susceptibility of asphalt mixtures by changing the filler part of aggregates or introducing antistripping modifiers to asphalt binders [37-40].

Expansive, high-plasticity soils exhibit significant mechanical property changes throughout seasonal drying and wetting cycles and cause different types of distress in the lightweight structures built over them. Low-volume roads, as a significant part of our transportation system, often face frequent maintenance and premature failure in expansive subsoil regions. Shrinkage-induced longitudinal cracking from moisture depletion is believed to initiate in the subgrade [41]. The upward propagation of cracks is due to the weak bond between the subgrade and the base and to the low tensile strength of the base layer. If the tensile strength of the hot-mix asphalt (HMA) layer is also inadequate, cracks may propagate to the surface [42].

Thicker or stronger pavement layers (e.g., increased layer thickness or modulus) do not provide better performance with respect to subgrade shrinkage cracking. Once the shrinkage crack initiates, a better structural design will not prevent cracks from propagating but may delay their progress. Increasing the strength and stiffness of a subgrade has a greater impact than making the pavement layers thicker or stronger. As such, consideration should be given to mitigating detrimental subgrade properties, improving subgrade strength and stiffness, and reducing subgrade moisture susceptibility [43]. Low volume road flexible pavements constructed on high plasticity index (high PI) clay subgrades fail prematurely primarily because of the highly variable properties of these clays due to moisture fluctuation throughout the year. The most prevalent cause of this premature failure is reported as longitudinal cracking. Unfortunately, most current pavement design procedures do not account for this mode of failure. It is therefore, important to improve the design and laboratory procedures to evaluate clayey subgrades and then design pavements accordingly to extend the life expectancies of these roads. [44].

In the summer months, the subgrade dries out with time. Such loss of moisture results in significant increase in the strength and stiffness (modulus) of the clay, which has a positive impact on the load carrying capacity of pavements. However, the increase in stiffness results in the increase in the brittleness of the clays. The loss of moisture also contributes to the shrinkage of these clays. This tendency to shrink, along with the increase in the brittleness, causes longitudinal cracks that will propagate to the surface of the road [45].

2.5 Prevention of premature failure with additives

Numerous additives have been shown to promote adhesion between asphalt binders and aggregate, these include:

- Liquid anti-stripping additives: These additives are added in liquid form to the asphalt cement, they act as surface activation agents which promote increased adhesion of the asphalt binder to the mineral aggregate [46].

- Lime additives: These additives are added to the aggregate, and have been proven to be very effective at mitigating moisture damage through improved adhesion of asphalt binder and aggregate [47].

- Modified asphalt binders: Polymer modifiers such as Styrene Butadiene Styrene (SBS) are typically used to modify the asphalt binder; such modifiers tend to improve the performance of the binder by increasing its elasticity and decreasing its susceptibility to permanent deformation. These binders typically yield greater film thicknesses, which have been shown to protect against moisture damage [48].

Coating of the aggregate surface with Zycosoil decreases this difference, and subsequently causes the mixture to be more resistant to moisture damage [49-52].

CHAPTER 3

EVALUATION OF SUBGRADE SOIL ON PREMATURE FAILURE OF FLEXIBLE PAVEMENT IN HILLY AREA

3.1 Introduction

When the pavement below gets saturated a result of migration of water by capillarity it results in a gradual loss of bearing capacity, which may eventually lead to pavement failure [53]. Increased moisture content reduces the resilient modulus of granular materials, their frictional strength and resistance to deformation [54]. Fine grained soils have a relatively smaller capacity in bearing a load than the coarser grained soils [55]. Once the moisture content reached or went over an optimum value the permanent deformation increased dramatically and the material collapsed [56]. The pavement structure loses significant structural capacity when the unbound material layers are saturated. The pavement structure rapidly regains strength once the subsurface water level dropped below the base course layer [57]. Failures may be due to either traffic (load associated) or environmental (non-load associated) influences. Destructive actions in the flexible pavement are quickly increased when surplus water is retained in the flexible pavement void spaces and increased moisture content in the soil can lead to increased pavement deflection [58, 59].

3.2 Study area

The state highway was constructed in 2000 and completed in March 2010 under the Mizoram State Road Project funded by World Bank with a length of 164 km. This SH has the shortest route link to the southern side of Mizoram as shown in the figure 1.4. It is considered as 'Priority Road' as it is the shortest route to connect north and south of Mizoram.

The road has been divided into four sections as shown in table 3.1. 'Section I' pavement condition is good except for the few locations which are addressed in this study. The 'Section II' condition of the pavement is good and structurally sound and performs after 10 years of construction.

'Section IV' is the worst section where failure occurs repeatedly even after frequent repair and overlay. The pavement composition along the state highway (ATL Road) is as below:

Table 3.1 Pavement Composition of State-Highway (ATL) Road

Pavement Composition	Section I (0.57 - 28.50 km)	Section II (28.50 - 52 km)	Section III (52-98 km)	Section IV (98-164 km)
Wearing Course	20 mm MSS	30 mm BC	20 mm MSS	30 mm BC
Binder Course	50 mm BM	-	-	-
Base Course	150 mm WMM	225mmWBM	225mmWMM	225mmWMM
Sub-base Course	150 mm GSB	150 mm GSB	150 mm GSB	200 mm GSB
Pavement Thickness	370 mm	405 mm	395 mm	455 mm
Avg IRI (mm/km)	7.7	8	8.5	9

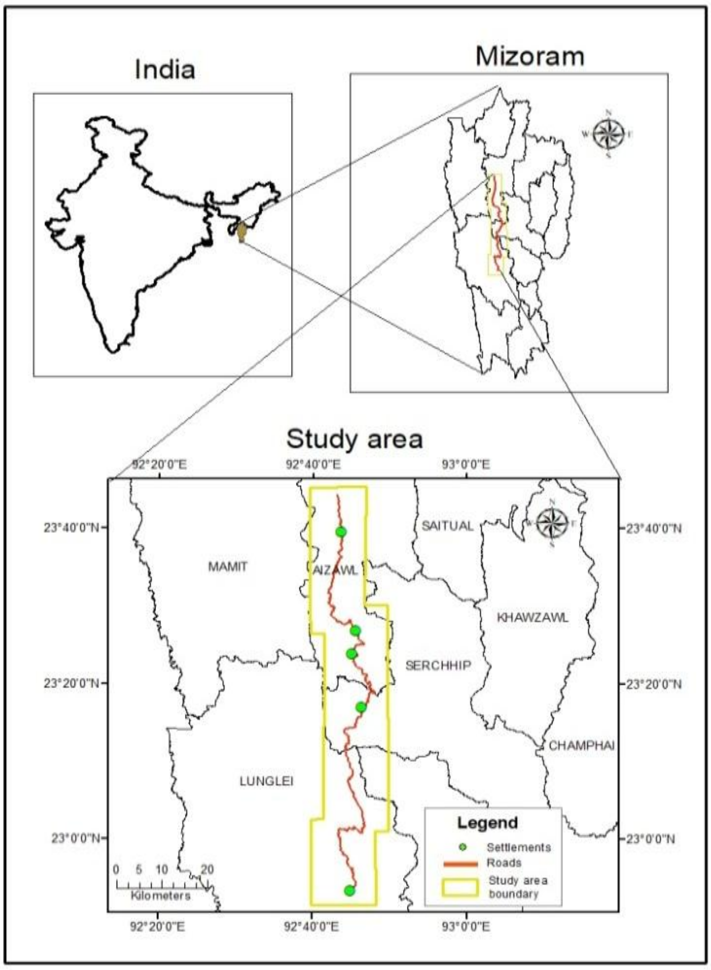

Fig. 3.1 Study Location

The section locations selected in this study are as follows:

1. Chainage 14-20 km (Muallungthu Area)
2. Chainage 10-14 km (Falkawn Area)
3. Chainage 100 -164 km (Pukpui Area)

Details of the different locations considered under study area including location coordinates, section condition, and type of distress is tabulated in table 3.2

Table 3.2 Sample Location Details

Name of the sample	Location coordinates		Section Condition	Distress Type
	N	E		
Falkawn (1H&1V)	N23^{0}37'12.67"	E92^{0}43'22"	Very Poor	Rutting, Crack, Pothole
Falkawn (2H&2V)	N23^{0}37'02.04"	E92^{0}43'08.17"	Very Poor	Rutting, Crack, Pothole
Muallungthu (3H)	N23^{0}36'03.81"	E92^{0}43'07.46"	Very Poor	Rutting, Crack, Pothole
Mel 5(Good Section)	N23^{0}40'9.94"	E92^{0}43'31.41"	Good	-
Kelsih	N23^{0}38'06.66"	E92^{0}43'24.37"	Moderate	Crack
Hualngo	N23^{0}39'23.81"	E92^{0}43'39.28"	Very Poor	Rutting, Crack, Pothole
Pukpui 5	N22^{0}57'48.45"	E92^{0}44'14.14"	Very Poor	Rutting, Crack, Pothole
Pukpui 6	N22^{0}57'59.27"	E92^{0}43'52.18"	Very Poor	Rutting, Crack, Pothole

3.3 Condition of the pavement

There were numerous shady and damp stretches along the road length that received little or no sunlight at all. This posed a huge problem for the construction of pavement as sub-grade would remain damp even well into the working season.

The subgrade soil is examined to determine the causes of premature failure of the pavement as the pavement condition along the highway is in bad condition at a few stretches despite using the same road construction materials. The premature failure occurred frequently at some locations even after many repairs. Since all sections of the pavement are constructed from the same material, except for the foundation soil, the subgrade soil has been focused on.

3.4 Materials and Methods

3.4.1 Materials

A total of 10 soil samples were collected from 8 different places on State Highway (ATL Road) of Mizoram. 8 samples out of 10 were taken from ‘poor section’ where premature failure frequently occurs. The locations are at Falkawn (1V, 1H, 2V, 2H), Muallungthu (3H), Hualngo, Pukpui 5 and Pukpui 6. Two soil samples were taken from ‘moderate section’ at Kelsih and ‘good section (pavement failure not observed for a long period time)’ at Mel 5. Subgrade soil sample was collected not only in a failed section; it is also taken from the ‘moderate’ and ‘good section’ also to make a comparison.

A CBR test was conducted on the soil samples which were collected from Melnga site of World Bank road to assess the effect of moisture content on the strength of soil under different duration of soaking.

3.4.2 Methods

Samples of soil collected from the site were tested by Atterberg limits, Wet sieve analysis, Permeability test, Compaction test and California Bearing Ratio Test (CBR) to determine the physical properties of the soil, compaction characteristics and strength of the soil. A soil swell test was conducted using the free swell index and hydrophilic coefficient measurements.

3.5 Results and Discussion

3.5.1 Classification and physical properties of soil:

Soil classification is conducted to find out the type of soil using Highway Research Board, Unified Soil Classification and Group Index Method as shown in Table 3.3. The soil quality in terms of drainage, volume change characteristics is poor as seen from the table below. The Good section (Mel 5) and Moderate sections (i.e Kelsih) are found as suitable for subgrade material.

Table 3.3 Classification of Soil

Name of the Sample	Soil Classification			Type of Soil	Suitability as a subgrade	Drainage	Volume Change
	HRB	USC	GI				
1V	A-7	CL	11.63	Clay	Poor	Very poor	High
1H	A-4	ML	5.34	Silt	Fair	Poor	Medium to High
2V	A-7	CH	13.42	Clay	Poor	Very poor	Very High
2H	A-5	ML	3.63	Silt	Fair	Poor	Medium to High
3H	A-4	CL	8.00	Silty clay	Fair	Very poor	Medium
Kelsih	A-4	ML	4.09	Silt	Fair	Poor	Slight to Medium
Hualngo	A-6	CL	3.17	Plastic Clay	Very Poor	Very poor	Medium
Pukpui 5	A-4	CL	2.72	Silt	Poor	Very poor	Medium
Pukpui 6	A-6	CL	9.94	Clay	Very Poor	Very poor	Medium
Mel 5	A-4	ML	3.29	Silt	Good	Fair to Poor	Slight to Medium

Table 3.4 Physical Properties of Soil

Name of the Sample	Specific Gravity	Permeability (cm/sec)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	CBR (%)	MDD (g/cm^3)	OMC (%)
1V	2.34	$8.05x10^{-7}$	44	26.93	17.07	4.68	1.44	25
1H	2.55	$5.6x10^{-6}$	25.6	20.95	4.65	5.12	1.75	18
2V	2.59	$7.34x10^{-6}$	53	34.01	18.99	5.53	1.53	18
2H	2.52	$9.5x10^{-6}$	40	30.73	9.27	5.84	1.74	16
3H	2.57	$1.69\ x10^{-6}$	34.7	26.43	8.27	5.58	1.69	19
Kelsih	2.57	$1.34\ x10^{-6}$	29	19.34	9.66	6.57	1.73	16
Hualngo	2.6	$3.52x10^{-6}$	35	21.8	13.2	3.51	1.89	13
Pukpui 5	2.53	$2.35x10^{-6}$	35	27.1	7.9	5.2	1.73	16
Pukpui 6	2.53	$3.6\ x10^{-7}$	40	22.98	17.02	4.2	1.8	17
Mel 5	2.484	$4.31\ x10^{-5}$	30	25.98	4.02	7.88	1.76	14

As shown in table 3.4, the good section (Mel 5) soil is selected to compare the soil condition with the poor section of the road. The good section of soil is having a low PI value of 4.02%, a high MDD of 1.76 kg/cm^3, a high CBR value of 7.88%, and lesser silt and clay content (46.95%). The poor section of the road soil samples are having high plasticity, low CBR (CBR <5.5%), MDD value less than 1.75 kg/cm^3 and a high value of silt and clay content (>70%). Pavement sections such as Hualngo, Pukpui 5 and Pukpui 6 have a bottom-up failure as a result of water seepage from the hillside during rainy season, which lead to the failure of the subgrade.

There is low permeability in the entire stretches at the level of subgrade as shown in the table 3.4. Most of the pavement section fails on the hilly side first in the study area, maybe due to the slow rate of water dissipation in the downward and transverse directions. A significant amount of rainfall infiltrates into the subgrade through the shoulder on both sides of the pavement. As the valley side has an open slope terrain characterized by more permeable soil, it can easily drain out water horizontally, while the hilly side has a relatively less permeable rocky soil, which slows down drainage of water horizontally and downward direction. In fact, the hill side does not have an open surface to drain out water.

Based on the properties of the soil, soil by itself is not the primary reason for pavement failure; rather, failure happens when the soil becomes significantly saturated with water over an extended period of time.

3.5.2 Influence of silt and clay in the subgrade soil

Wet Sieve Analysis test is conducted to find out the particle size of the soil. Silt and clay are soil particles passing 75microns sieve. From table 3.5, we observed that soil from locations of 1H, 2V, 3H, and Pukpui 6 are having silt and clay content of more than 70%. A higher percentage of silt & clay content leads to low permeability and infiltration which results in poor drainage. Poor drainage causes early pavement distresses such as rutting, cracking and shoving.

Table 3.5 Particle Size Distribution

Sample Name	Particle Size Distribution		
	Gravel%	**Sand %**	**Silt & Clay %**
1V	1.3	19.49	79.21
1H	0.073	38.205	61.712
2V	3.2	24.733	72.067
2H	6.5	41.2	52.3
3H	3.1	19.329	77.571
Kelsih	8.71	35.825	55.465
Hualngo	18.767	35.342	45.891
Pukpui 5	10.6	40.8	48.604
Pukpui 6	5.2	24.129	70.671
Mel 5	13.3	39.75	46.95

3.5.3 Influence of the road design level from original ground level

As the depth of the pavement level differs from the original ground level (OGL), soil samples were taken from hill and valley sides of the pavement (sections 1 and 2) to determine the differences in their physical properties. Soils are able to hold more loads with increasing depth [60]. Due to the gradient of the slope terrain, the level of the road cutting varies from hilly to flat terrain. The depth of cutting depends on the degree of terrain. Steep terrain has resulted in a higher cutting height and vice versa.

Table 3.6. Comparison between Hillside and Valley Side of Subgrade Soil

Name of the Sample	Permeability (cm/sec)	Plasticity Index %	CBR %	MDD in kg/cm^2	OMC %	Silt& Clay %	Avg. Height of cutting 'm'
1(Valley)	$8.05x10^{-7}$	17.07	4.68	1.44	25	79.21	1.7
1(Hill)	$5.6x10^{-6}$	4.65	5.12	1.75	18	61.712	4.5
2(Valley)	$7.34x10^{-6}$	18.99	5.53	1.53	18	72.067	1.2
2(Hill)	$9.5x10^{-6}$	9.27	5.84	1.74	16	52.3	3.9

The valley side of the pavement section has a lower plasticity index, CBR, and MDD than the hillside as shown in table 3.6. The pavement normally starts to fail from the hillside which is caused by seepage from hillside and delayed of water on the wearing course. There is inconsistency in the density of soil on the cross section of the road due to varying depths of pavement levels from original ground surface. In other word, uniformity of the soil strength does not exist on the transverse section of the road.

3.6 Influence of moisture level in pavement subgrade

3.6.1 Effect of groundwater level

The construction of pavement at numerous shady, damp and seepage-prone stretches presented another challenge. Even during the dry season (March), there is still water seepage on the surface of the pavement at Pukpui, Hualngo as shown in the figure 3.2. Since the pavement is fully saturated even during the dry season, it will weaken the pavement structure as soil tends to lose strength when fully saturated with water. It is necessary to lower the water table in this section and to provide good drainage so that the water can drain off quickly.

(a) **(b)**

Fig.3.2 High water table (a) Pukpui (b) Hualngo

The Pukpui stretches have a layer of shale rock that is considered impermeable. A layer of impermeable material prevented the water from further propagating downward. Impermeable layers prevent water from flowing further downward, making an unconfined aquifer that leads to pavement collapse.

3.6.2 Effect of water saturation level

Saturation level of any soil with water can be understood by the total amount of void in the soil that has been filled up by water. The failure of a pavement is more often dependent upon the underlying subgrade. Saturation mainly affects the subgrade layer of the pavement. Under the influence of the southwest monsoon, the study area experiences significant rainfall from June to September, with an average annual rainfall of 2,794 mm. The high intensity of rainfall leads to the saturation of subgrade soil if proper geometric design is not incorporated. There were numerous shady and damp stretches along the road length which received little or no sunlight at all. This posed a huge problem to the construction of pavement as sub-grade would remain damp even well into the working season.

The CBR tests were performed at different water saturation levels to investigate the variation of CBR value with respect to different duration of soaking. The saturation period of soil in water is divided into number of days such as unsoaked, 4,7, 14,21,61, 90, 102 and 150 days. The soil classified as is silt based on the value of A-4 (HRB), 3.29 (Group Index) and ML as per USC.

It is observed that the CBR value of the soil sample prepared at a particular density decreases rapidly after 4 days of soaking. The rate of decrease of CBR value beyond 4 days soaking is minimal. It is also observed that there are not much significant variations in CBR values from 90th day of soaking. The soil strength reduces as the water saturation period of soil increases as shown in the figure 3.3. Subsurface moisture reaches lower layers of pavement from different sources like pavement surface, seepage from shoulders and seepage from adjoining hills and capillary rise of moisture from the ground. If water is able to enter the pavement, there must be a way for it to depart. Soil resistance to compression may drop substantially if its moisture content increases. Increased moisture content reduces the load-carrying capacity of the road and causes premature pavement failure, resulting in shorter pavement life.

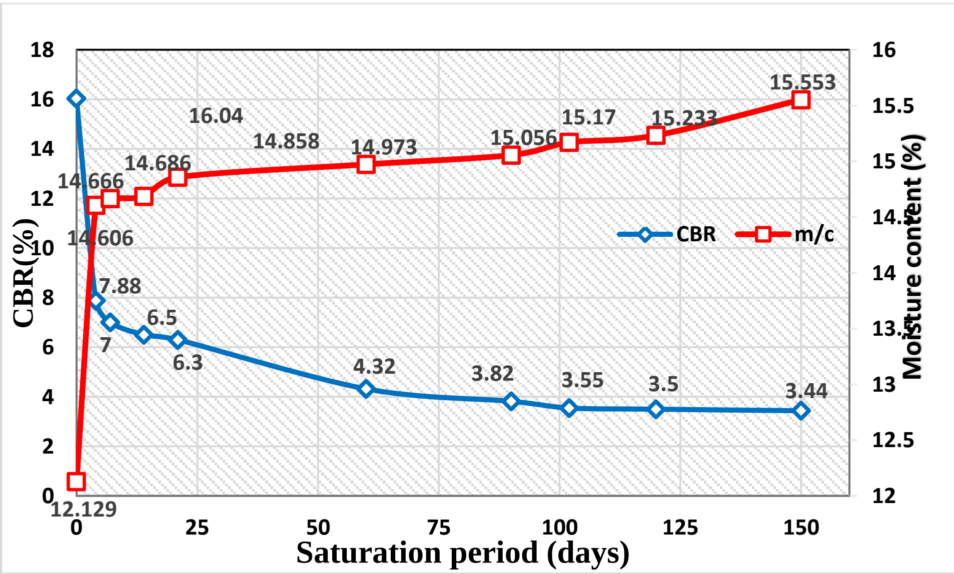

Fig.3.3 Effect of moisture saturation period in soil strength

It was conceivable that the build-up of moisture in the subgrade would ultimately lead to its degradation under vehicular load and this was evident from the localized appearance of rutting and depressions at several locations. In a few cases, substantial accumulation of water had led to the puncturing through of the sub-base layer and complete failure of the subgrade.

3.6.3 Soil hydrophilic coefficient and Free swell index

Surface wettability plays a significant role in how fluids interact with materials. Water has a strong affinity for hydrophilic surfaces, while hydrophobic surfaces have a weaker affinity for water [61]. The hydrophilic coefficient is the ratio of volumes after 72 hours of sedimentation of equal dry-volumes of dry and high specific surfaced soil in water and paraffin. The hydrophilic coefficient can be defined as follows:

$$\eta = V_{water} / V_{paraffin}$$

Where

η : hydrophilic coefficient

V_{water} : volume of soil in water after 72 hours sedimentation

$V_{paraffin}$: volume of soil in paraffin after 72 hours sedimentation

The hydrophilic soils have a higher volume in water than they do in paraffin; therefore, their hydrophilic coefficient will be high. If the coefficient is greater than 1, the soil has a hydrophilic property. If it is under 1, the soil has a hydrophobic property.

The Free-swell index is the increase in the volume of soil, without any external constraints, upon submersion in water. The possibility of damage to structures due to swelling of expansive clays needs to be identified. All the soils are hydrophobic having non reactivity with water as shown in table 3.7.

Table 3.7 Swelling Properties of Soil

Name of the Sample	Free Swell Index %	Hydrophilic coefficient
1V	0	1.01
1H	0	1.06
2V	0	0.98
2H	0	1.01
3H	0	1
Kelsih	0.56	1.01
Hualngo	1.69	1.12
Pukpui 5	1.35	1
Pukpui 6	0.75	1.03
Melnga	0	1.01

The soil is non-reactive with water since they do not change in volume when in contact with water. Moreover, a small change in volume of soil is observed at Hualngo, Pukpui 5 and Pukpui 6 locations.

3.6.4 Geometry of the pavement

Pavement geometry includes carriage width, alignment, sight distance, shoulder, side drain, curve, camber, and gradient. The 35 year average rainfall of Mizoram is over 2000 mm which has a significant impact on subgrade softening if surface and sub-surface drainage is poor. The rainy season (monsoon) lasts for at least four months (June-September) and contributes about 70 percentage of total annual rainfall. The road with a high shoulder delays the surface runoff of water due to poor surface drainage as shown in figure 3.4. Delay water saturates the subgrade by entering the shoulder from the edge of the pavement.

The alignment passing via a Box cut (through cut) normally creates drainage issues mainly due to the water sources coming from both sides of the hill slope.

Fig.3.4 Raised shoulder and tall grass blocking surface drainage

The performance of embankment sections at Thenzawl is good and there is no pavement distress found along the road till recently. With the pavement raised above the ground, an embankment provides an advantage of preventing ground water and allowing for improved drainage.

3.7 Summary

It is found that pavement failures occur due to oversaturation of soil caused by poor drainage at Hualngo and Pukpui stretches. Pavement stretches in Hualngo and Pukpui have a layer of shale rock that prevents water from penetrating further downward, causing unconfined aquifers and pavement failure.

The varying amount of saturation level on the soil increases (i.e. the degree of exposure of soil to water/moisture), the overall load bearing strength of the soil decreases considerably. Based on the properties of the soil, soil by itself is not the primary reason for pavement failure; rather, failure happens when the soil becomes significantly saturated with water over an extended period of time.

The hillside of the road has a higher chance of subgrade degradation despite its higher soil density owing to the higher possibility of dampness and water intrusion from the hillside. One of the major causes of premature failures is due to the emanation of water seepage in the form of spring along the hilly side of the road. Water flow needs to be channelized and controlled.

Road cutting at a differential level on sloping terrain results in non-uniform soil strength across the transverse section. The deep cutting of the road is not suggested due to the possibility of encountering water seepage. Replacement of weak subgrade with granular material was found to be ineffective as the new material would get saturated in a short period of time. Although not ideal, the technique of applying a layer of stone soling at subgrade was determined to be the most effective.

CHAPTER 4

HYDROGEOLOGICAL EFFECTS ON PREMATURE FAILURE OF FLEXIBLE PAVEMENT IN HILLY AREA

4.1 Introduction

Road cutting on the hill slope is inevitable in the hilly area for the construction of pavement. The roads are constructed on geologic materials, and these materials properties impact their functioning as transport medium [62]. Performance of a road pavement can be severely affected by the existence of geological features and engineering characteristics of the underlying geologic sequences [63]. The proper design of a highway requires adequate knowledge of subsurface conditions beneath the highway route. Road failures can also arise from inadequate knowledge of residual soils geotechnical characteristics and behaviour on which the roads are built and non-recognition of the influence of geology and geomorphology during the design and construction phases [64]. Poor geotechnical characteristics of the soils such as insufficient bearing capacity, low maximum dry density, high liquid limit, high plasticity index, low California bearing ratio and high compressibility are typical geotechnical parameters responsible for road failures [65]. Other factors such as hydrogeology, climate and drainage are among the most critical factors contributing to pavement instability. Lack of proper drainage in pavement can enhance the ingress of water into the pavement structure causing significant disintegration of the underlying materials. As a result there is a considerable loss of strength in pavement and shoulders material leading to pavement failure in many occasions [66,67]. The use of substandard base and sub-base materials could also result in road failure. Depth of water table is considered to be one of the most important climatic, topographic and drainage factor affecting road performance [68]. Relative position of groundwater table with respect to ground level has a significant effect on the overall bearing capacity of the pavement structure and the stiffness of the unbound layers. The effects of moisture are more damaging on materials with a high proportion of fines. [69]. Excess moisture presence in pavement unbound layers can result in lower structural stiffness and reduce the service life of road systems [70]. In flexible pavement, permeability of the surface (hot mixed asphalt, HMA) layer controls the flow of water through the HMA layer to the underlying layers. However, floodwater can also make its way through surface distresses and hence for a specific section of the roadway, the intensity of cracks and joints will also be an important factor affecting water ingress [71]. In inundation conditions, the surface and the subsurface drainage areas are filled with water and do not contribute to any drainage from the pavement [72].

The objective of the present study is to identify the cause of premature failures observed in multiple locations along state highway-I in Mizoram. In light of the discussions made above regarding different factors related to pavement failure, an in-depth geophysical and geotechnical investigation was conducted. The results of geotechnical and geophysical tests coupled with geological survey data obtained from site were evaluated one by one to ascertain the cause of failure. The investigation results implied the more prominent role of geological and geotechnical features.

4.2 Study Area

Since the study areas encounter high intensity of rainfall over 2000 mm/year, which may be the contributing factor to the early deterioration of the pavement. Details of the different locations considered under study area including location coordinates, section condition, span of failure zone and distress type is tabulated in Table 4.1.

This study included a Good/ Sound Section of Mel 5 site for comparison to other premature failure sites. It is included to provide an understanding of how good road conditions are influenced by geological and hydrological factors.

4.3. Methods

4.3.1 Geotechnical Method

The soil samples were collected from the selected sites as shown in Table 4.1 to study the geotechnical properties. Soil samples were taken at the subgrade level. The excavation was carried out at the edge of the pavement until the subgrade level was reached. Laboratory tests such as Atterberg limit test, particle size analysis, permeability, standard compaction test, shear test, and California Bearing Ratio (CBR) test were conducted for the collected soil samples. To have a comparison of geotechnical properties and to serve the purpose of a controlled sample, soil samples from a sound stretch (Mel 5- ch.7 km) were also collected along with the soil samples collected from failed stretches.

4.3.2 Geophysical Method

As a part of geophysical method, Electrical resistivity method using Vertical Electrical Sounding (VES) technique was used to study groundwater condition and lithology of the selected stretches under consideration. Electrical Resistivity Method is one of the geophysical techniques used to investigate the nature of the subsurface formations. The Vertical electrical sounding is used to estimate the resistivity and thickness of various subsurface layers at a given location. The method is based on the estimation of the electrical conductivity or resistivity of the medium. Schlumberger configuration is adopted in which all four electrodes are kept in a line. The outer electrode spacing is kept large compared to the inner electrode spacing, usually more than five times.

The lithology types and thickness of the individual layers from the vertical electrical sounding locations were anticipated by referring lithology unit. The electrical resistivity data were collected using the Resistivity meter SSR MP-ATS model for Vertical Electrical Sounding (VES). The estimation is performed based on the measurement of voltage of electrical field induced by the distant grounded electrodes [73]. Vertical electrical soundings using Schlumberger arrays were carried out at five (5) stations to find the resistivity of the subsurface layers. The apparent resistivity (ρ_a) data were then plotted against the electrode spacing (AB/2) to obtain a resistivity-depth model at IP2win software.

4.3.3 Geological Survey

A physical survey employing compass survey was undertaken to gain insight of features related to geological structures such as fractures, folds, joint sets, bedding, dip amount and dip direction of rock layers found alongside hill slopes at selected locations as explained in Table 4.1. The purpose for such study was to characterize the surface run off and water movement into or below the pavement structure. The geological features are not only relevant to the landslide problem, but also to the pavement structure.

4.4 Results and Discussion

Hill road alignment may follow alignment at valley bottom or on a ridge depending on the feasibility of the road. The first is called River route and the second is called Ridge route. In the study area, roads passed through hills and a few valleys, necessitating slope cutting throughout the alignment. In almost all the cut slopes studied in the present work, exposed soil and rock strata was observed. The road which crosses soil and rock strata throughout its alignment and is susceptible to groundwater flow needs geological assessment of the exposed rock strata for evaluation of its effect in pavement performance.

In order to identify the cause of premature failure of pavement at selected locations, a range of methods involving geotechnical test, geophysical tests, groundwater survey and geological structural survey were carried out. The results of various methods as mentioned above in relation to premature failure occurrence have been discussed in subsequent sections.

4.4.1 The geological structure and geotechnical properties

Proximity of ground water in relation to subgrade layer and ground water movement direction plays a significant role in pavement performance. In the present context, given the geology of the study area, it is pertinent to mention that groundwater movement is affected by the dip angle of the bedding plane and the direction of rock/soil strata toward (dip slope), against (anti-dip slope), and parallel to the road alignment. The favorable situation for a well performing pavement structure is when rocks/soil strata dip against the slope (anti-dip slope) resulting flow of water away from pavement, while the most unfavorable condition will be when rocks dip towards the slope. The natural disposition of rocks may be horizontal, vertical and inclined. The strike and dip of bedrocks/soil strata in relation to road alignment is a crucial aspect concerning the performance of pavement. If rocks are inclined, the strike direction of rocks/soil strata will be parallel, perpendicular, or diagonal to the road alignment. Rock/soil strata dip against the hillside slope is more favorable than rock bed dip towards the slope (road alignment). Geological condition of the affected locations along with dip and strike direction in relation to pavement alignment is presented in Table 4.1. Geotechnical properties of soil samples collected from different stretches are tabulated in Table 4.2.

From the results presented in Table 4.2, it may be observed that highest value of CBR (design parameter in flexible pavement) is observed at location Mel 5. Further, geological structure of hill slope at location Mel 5 is that road cut is parallel to strike and bed is dipping against the slope (anti-dip slope), which is the most favorable condition for pavement performance as long as water movement is concerned. The road alignment at this stretch has a dip direction of rock/soil strata bedding against the slope (road alignment) with a dip angle of 41^0. Thus, sound performance of pavement structure at Mel 5 may be attributed to moderate CBR value and favorable geological structure of hill slope.

Lowest value of CBR was observed for stretch at location Hualngo (12 km). Notably, geological structure of hill slope at location Hualngo (12 km) is unfavorable for pavement performance. Thus, it may be reasonably assumed that pavement distress developed at location Hualngo (12 km) owing to poor CBR value with an unfavorable geological structure of hill slope. It may be observed that for all the remaining stretches except for MCON (17 km) and MLT (19 km) wherein no geological structure is observed, road cut is parallel to dip direction and bedding is dipping along the road alignment. Nonetheless, CBR values at those locations are above 5% except for location at Pukpui (150 km) wherein a CBR value of 4.2% is observed. In the absence of soil-related failure, subsurface moisture can cause premature failure.

Table 4.1 Geological condition along the alignment of State Highway
(Actual measurement at the site)

Location	Geological Structure of hill slope w.r.t road alignment	Dip angle	Strike	Dip Direction	Depth of cutting
Mel 5 (7 km)	Road cut is parallel to strike, bed dip against the slope (anti-dip slope)	42^0	$N65^0E$	$S15^0$ W	6 m
Hualngo (12 km)	Road cut is parallel to strike, bed dip toward the slope (dip slope)	41^0	$N67^0W$	$S56^0E$	5 m
Falkawn (16 km)	Road cut is parallel to dip direction, rock bed dip along the road alignment	30^0	$N52^0E$	$S55^0$ W	6 m
MCON (17 km)	Structural features is not yet formed	-	-	-	7 m
MLT (19 km)	Structural features is not yet formed	-	-	-	5.5 m
Pukpui (150 km)	Road cut is parallel to dip direction, rock bed dip along the road alignment	11^0	$N44^0E$	$S46^0$ E	6.2m
Pukpui (154 km)	Road cut is parallel to dip direction, bed dip along the road alignment	13^0	$N23^0E$	$S67^0$ E	8m

Table 4.2 Geotechnical Properties of subgrade

Name of the Location	Specific Gravity	Permeabi-lity (cm/sec)	LL %	PL %	PI %	CBR %	Soil Classification		Compaction characteristics	
							HRB	USC	MDD (g/cc)	OMC%
Mel 5(7 km)	2.55	4.31×10^{-6}	30	25.98	4.02	8.59	A-4	ML	1.76	14
Hualngo(10 km)	2.6	3.52×10^{-7}	35	21.8	13.2	3.51	A-6	CL	1.89	13
Falkawn (16 km)	2.55	5.6×10^{-6}	25.6	20.95	4.65	5.12	A-7	CL	1.75	18
MCON(17km)	2.52	9.5×10^{-6}	40	30.73	9.27	5.84	A-5	ML	1.74	16
MLT(19 km)	2.57	1.69×10^{-6}	34.7	26.43	8.27	5.58	A-4	ML	1.69	19
Pukpui (150 km)	2.53	2.35×10^{-6}	35	27.1	7.9	5.2	A-4	ML	1.73	16
Pukpui (154 km)	2.53	3.6×10^{-7}	40	22.98	17.02	4.2	A-6	CL	1.8	17

The main cause of failure in premature failure zones is groundwater, which causes the subgrade to fail and the rock to disintegrate. The amount and direction of groundwater flow are determined by geological conditions such as dip amount, dip directions/orientations, fractures, folds, strike, joint sets and bedding. The flow of water through fracture contributes to the failure of the pavement structure when the bedding is dip slope.

4.4.2 Interpretation of resistivity value

The subsurface formations at the Vertical Electrical sounding (VES) locations are composed of three and four layers. Table 4.3 shows that when the subsurface formations consist of three layers or four layers, four types of curves and eight types of curves are possible respectively in various combinations and permutations of resistivities; the layer cases represent the situation with resistivities ρ1, ρ2, ρ3 and ρ4 [74].

Table 4.3 Various combinations and permutations of resistivities

Various combinations of resistivity	Type of Curves
ρ1>ρ2<ρ3	H type
ρ1>ρ2>ρ3	Q tpye
ρ1<ρ2<ρ3	A tpye
ρ1<ρ2>ρ3	K Tpye
ρ1< ρ2 < ρ3< ρ4	AA Type
ρ1 > ρ2< ρ3 <ρ4	HA Type
ρ1 > ρ2< ρ3> ρ4	HK Type
ρ1< ρ2< ρ3 > ρ4	AK Type
ρ1< ρ2 > ρ3< ρ4	KH Type
ρ1< ρ2 > ρ3> ρ4	KQ Type
ρ1 > ρ2>ρ3 < ρ4	QH Type
ρ1 >ρ2 >ρ3 >ρ4	QQ Type

Table 4.4 shows that possible layers of Hydrogeological sections at VES 01 to VES 05. VES-01(Mel 5- 7 km chainage) has a three-layer earth model; its inferred lithological units are silty gravel, sandstones, silty clay. The sounding curve is shown in figure 4.1. The sandstone rock dips against the road at the outcrop area. The top layer of soil at this location has good physical properties. The lithological units at VES-02 include clay, clayey sand with water, clay and soft rock/shale as shown in figure 4.2. The water presence is found at a depth of 1.454 m. The rate of infiltration of water is low due to the low hydraulic conductivity of clay. Clay layer acts as impermeable layers that prevent water from flowing further downward, making an unconfined aquifer that leads to pavement collapse.VES-03 lithological units are clayey silt, clay with groundwater, and soft rock/shale, as shown in figure 4.3. Seepage water exists at a depth of 4.33 m. Because of its low infiltration of

clay, the lithological layer can trap water for a long time.

Table 4.4 Possible layers of Hydrogeological sections

Name of Station	Layers no.	Resistivity (Ohm-m)	Thickness (m)	Curve Type	Lithology
VES 01 (Mel 5 - 7 km)	1	305	0.482	K	Silty Gravel
	2	1242	1.47		Sandstone
	3	72.6	0		Silty Clay
VES 02 (Hualngo-10 km)	1	36.49	0.41	KH	Clay
	2	113.9	1.044		Clayey Sand with fresh water
	3	43.95	4.26		Clay
	4	167.3			Soft rock/Shale
VES 03 (Falkawn 12 km)	1	149	4.33	H	Clayey silt
	2	0.762	17.9		Clay with groundwater water
	3	120			Soft rock/Shale
VES 04 (MCON-13km)	1	27.6	0.59	KQ	Clayey Silt
	2	380	0.709		Sand/soft rock with fresh water
	3	64.5	8.68		Silty Clay
	4	6.7			Clay with groundwater
VES 05 (MLT-15km)	1	28.65	0.652	K	Clayey Silt
	2	361.2	2.15		Sand/soft rock
	3	0.392			Clay with groundwater

VES-04 lithological units are clayey silt, sand with freshwater and clay with groundwater. Water seepage from the adjoining hillside is encountered at a depth of 1.299 m. VES-05 lithological units are clayey silt, sand, silty clay and clay with groundwater. At VES-04 and VES-05, groundwater is found at a depth of 9.97 m and 2.8 m with electrical resistivity values of 6.7 Ωm and 0.392 Ωm, respectively as shown in figure 4.4 and 4.5. Pukpui stretch area has the outcrop appears along the full stretches. The road was cut through shale rock, which is considered impermeable. The seepage water is attained at the surface level of the road in Pukpui area due to seepage from the adjoining hillside, capillary rise from the shale bedrock. The pavement structure traps water because water cannot further infiltrate the shale bedrock.

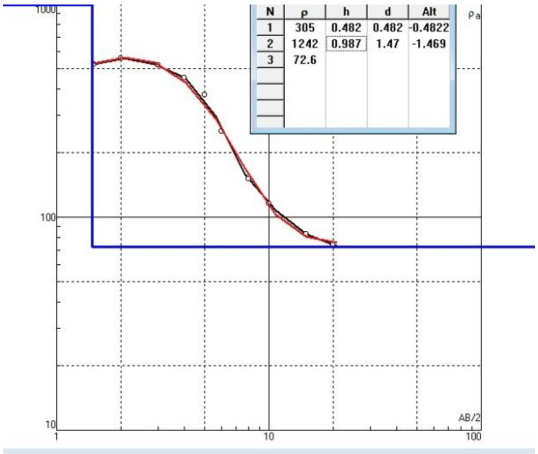

Fig. 4.1 VES Station 01-Mel 5 (7 km)

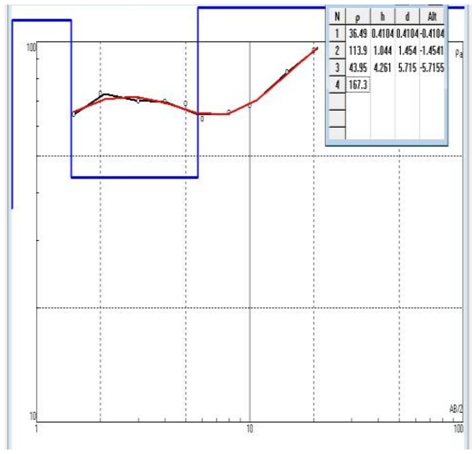

Fig. 4.2 VES Station 02Hualngohmun (10km)

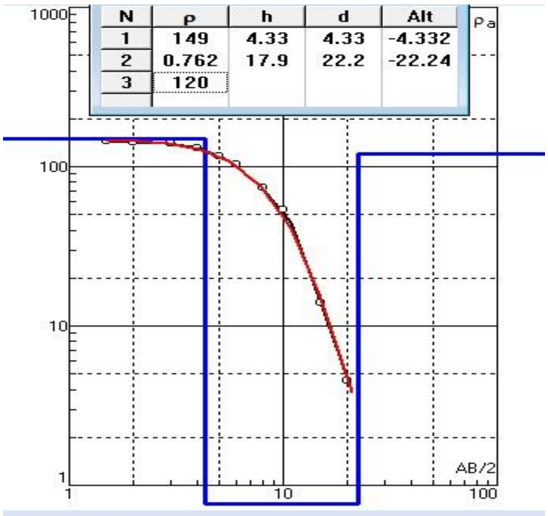

Fig. 4.3 VES Station 03- Falkawn (16 km)

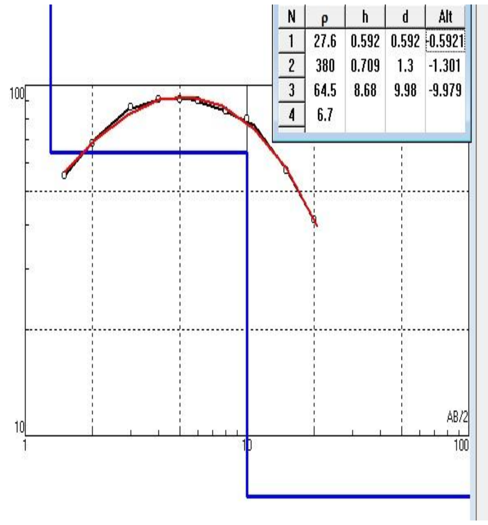

Fig. 4.4 VES Station 04- MCON (17 km)

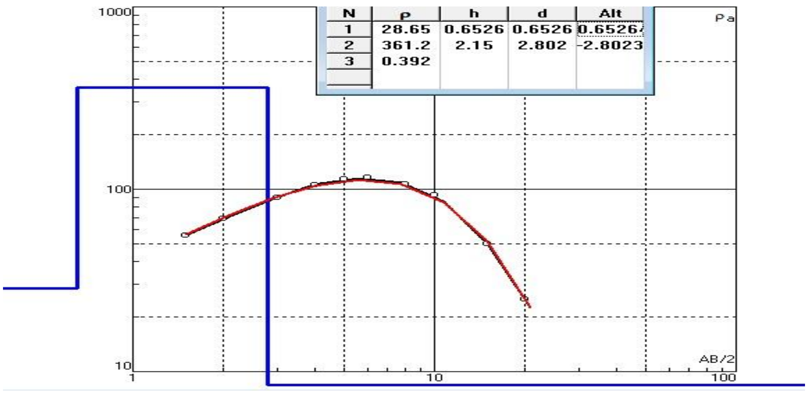

Fig. 4.5 VES Station 05- MLT (19 km)

Soil samples were obtained from subgrade level at affected locations and laboratory tests were conducted to find out properties pertaining to permeability, compaction and plasticity behaviour of soil. A careful evaluation of geophysical and geotechnical test results revealed that geological features were crucial factor affecting the pavement performance. Factors such as dipping of rock strata towards the cut-slope, presence of impermeable bedrock strata at shallow depth below subgrade, emanation of groundwater in the form of springs along the hill slope were identified to be the contributing factors resulting in premature failure at locations considered in the study.

4.4.3 Cross-sectional element of pavement

Pavement cross-sectional element includes carriage width, shoulder, side drain, camber, and formation cutting. The road with a high/elevated shoulder obstructs the surface water to drain off from the pavement and force water to flow on the pavement as shown in figure 4.6 (a) & (b). Delay/standing water seeps to lower layers and accelerates the process of stripping caused by tall grass at the shoulder and elevated shoulder.

(a) (b)

Fig. 4.6 (a) & (b) Elevated shoulder & tall grass obstructing surface drainage

The absences of the side drain also lead to quick water recharge into the pavement structure. It is found that the road on the box cutting (through cut) has given a faster rate of deterioration than one side hill cut. There is a possibility of water seepage from both sides of the hill slope in box cutting. There are some badly damaged roads at Pukpui stretches at the box cutting portion, where seepage may occur from both sides of the hill. Water seeps through under the pavement layer, resulting in loss of aggregate to aggregate contact and premature pavement failure.

4.4.4 Hydrogeological condition and subsurface water

The study area is occupied by semi-consolidated formations of denudostructural hills belonging to the Bhuban Formation of Miocene age. The low linear ridges are characterized by low permeability and infiltration capacity. It acts as a runoff zone. The moderate linear ridges, which occupy the major portion of the study area, comprise hard and compact sandstone, shale, siltstones and alternations of Surma Group of rocks. This unit is also characterized by very low permeability and infiltration capacity that acts as a runoff zone. In general, discharges of the springs are meager in high altitudes, which progressively increase down a slope [75].

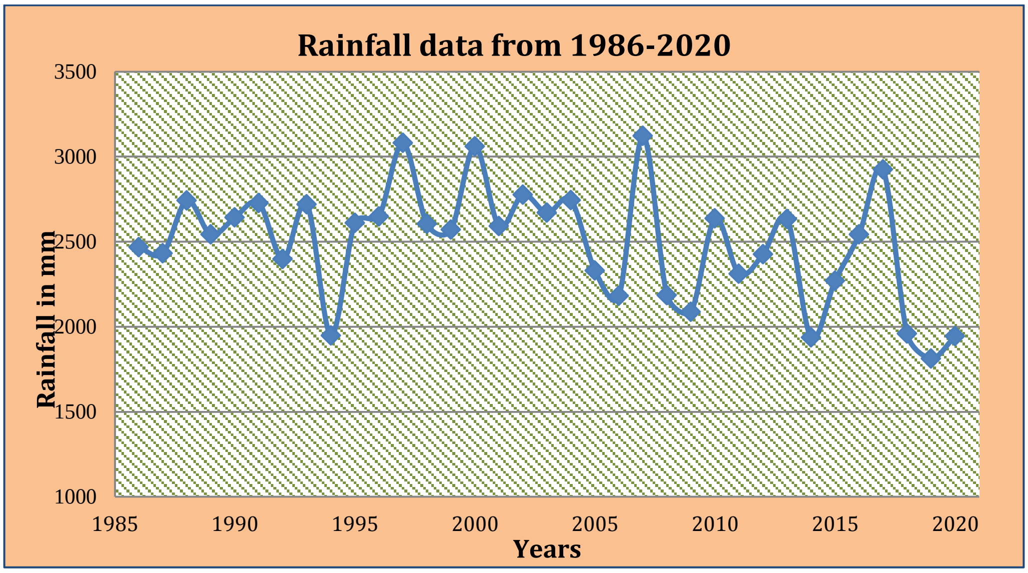

Fig. 4.7 Rainfall Data from 1986-2020

The 35-year average rainfall of Mizoram is 2379 mm as, shown in figure 4.7, which significantly impacts pavement softening by entering pavement structure through the shoulder, seepage and capillary rise. The rainy season (monsoon) lasts for at least four months (June-September) and contributes about 70 per cent of total annual rainfall. Subsurface moisture reaches lower layers of pavement from different sources like pavement surface, seepage from shoulders and seepage from adjoining hills and capillary rise of moisture from the ground. There must be a path for water to exit the pavement if it is able to enter. Soil resistance to compression may drop substantially if its moisture content increases. An increase in moisture content results in a reduction of the load-carrying capacity of the road and premature failure and distress of the pavement resulting in a reduction in the design life of the pavement [76].

Water can cause early deterioration of a road, if its surface and sub-surface drainage conditions are poor. The road is susceptible to early damage due to the prolonged presence of water if the drainage system is not sufficiently effective.

Subsurface moisture is mainly contributed by rainfall which is also the source of spring. Springs emerge as a result of recharge from precipitation through the overlying weathered zone and groundwater. A large number of springs along the highway are perennial. The weak planes such as fractures and joints have served as conduits for the movement and storage of groundwater and creating seepage conduits, which are sources of springs as shown in the figure 4.8

The presence of groundwater will result in lowering of shear strength of rocks, increased pore pressure and deterioration of foundation material. The water seepage is observed even during the dry season (November to April) at Pukpui stretches (between 150 km and 154 km) and Hualngo stretch (12 km), as shown in figure 4.9 and 4.10.

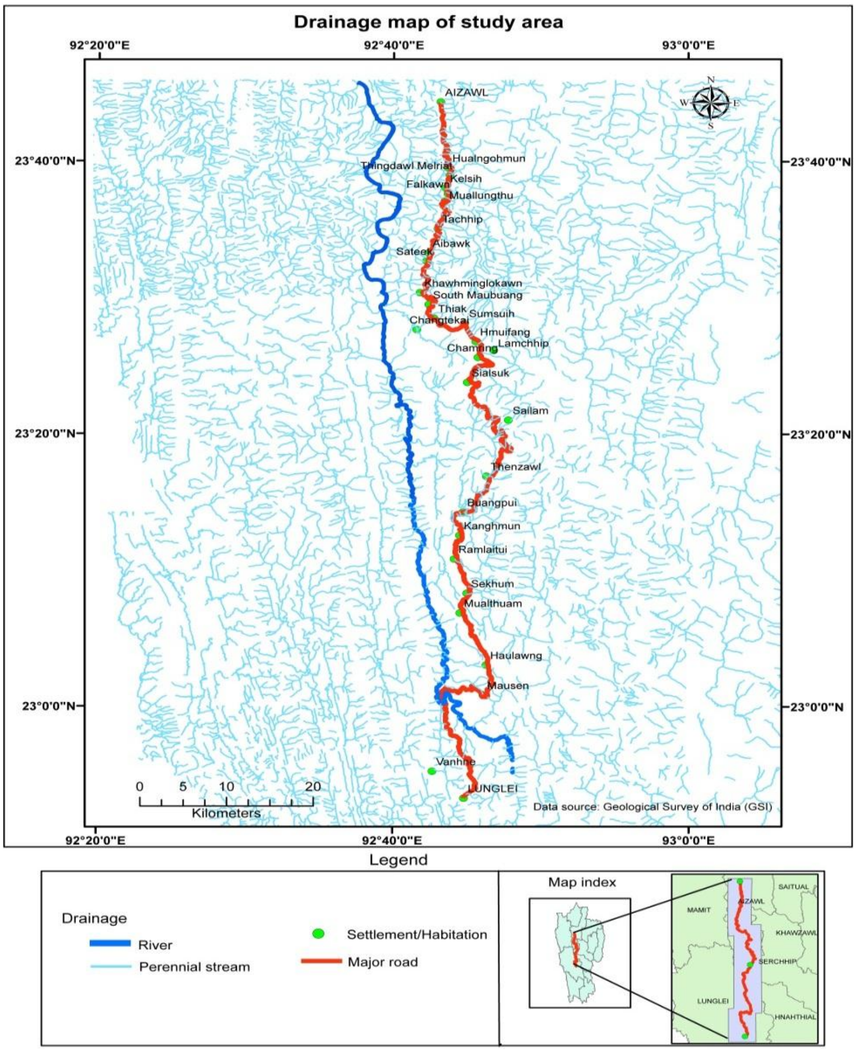

Fig. 4.8 Drainage map of the study area

Fig. 4.9 High water table   **Fig. 4.10** Emanation of groundwater from the hill slope

When moisture is trapped in the subgrade, pavement behaves like a spring, resulting in cracks, settlements, and ruts. Water trapped in the pavement layers reduces the structures load-bearing capacity.

4.5 Summary

The road condition deteriorates rapidly at some locations along the state highway despite using the same pavement design, construction method and construction materials. The road performs very well except for the selected study stretches.

The types of rock and geological structure features encountered in the alignment of the hill road section influence the pavement performance. In contrast to the embankment type of road construction, the road is constructed on the excavated slope of a hilly area. In hilly terrain, road-cut sections expose different rock features with springs at various locations. It has been demonstrated that hydrogeological and geotechnical factors affect road performance. As a result, the following conclusions have been drawn:

Road failure is associated with hydrogeological features such as the seepage of water from adjacent hills in cut sections, which is highly influenced by monsoon rainfall, which is recharged from precipitation through the overlying weathered zone, and groundwater emerges in springs. Seepage of water can be observed even on the pavement surface at Hualngo and Pukpui areas even during dry seasons. The presence of water saturates and weakens the pavement foundation as the soil tends to lose strength when fully saturated with water.

The resistivity values shown in the failure sections such as Hualngo, Falkawn, MCON and MLT are low, despite the fact that MCON and MLT layer 2 resistivity values are greater than 300 Ωm due to the presence of soft rock. The resistivity values indicate that soil/ rock beneath the pavements is water-bearing materials.

The road which crosses soil and rock strata throughout its alignment and is susceptible to groundwater flow in the form of a spring needs geological assessment. The geological structure along the pavement has played an important role in the performance of the pavement. The dip and dip direction of the rock/soil bedding plane toward the pavement (dip slope), against the pavement (anti-dip slope), parallel with the pavement and fracture affect the movement of groundwater.

The failure sections have a rock bed dip toward the slope/road alignment (dip slope) and parallel with the road alignment. The higher angle of dip and rock bed dip against the slope/road has a better drainage capacity. The good section at Mel 5 has a dip direction against the slope/road with an angle of 41^0. The flow of water through fracture contributes to the failure of the pavement structure when the rock bedding is dip slope.

Pukpui stretch has a bedding plane that dips at an angle of 11^0 parallel to the direction of the road with the same road gradient. A road cut follows the dip direction. The Pukpui area premature failure is caused by the geological structure that allows water to flow along the road gradient and impervious shale rocks that block further water infiltration.

CHAPTER 5

ROCK AGGREGATE SIZE INFLUENCE ON PHYSICAL DEGRADATION OF LAMINATE SEDIMENTARY ROCKS AGGREGATE IN FLEXIBLE PAVEMENT

5.1 Introduction

Aggregate on road construction is extremely important as it affects the overall performance based on its strength, durability, and resistance to sustained load after construction. Laminated sedimentary aggregate is the broken pieces of sedimentary rocks which has a small scale sequence of fine layer. The lamination layer, where clay and silt are present, is the weakest point in the rock structure. Water can penetrate the laminated layer and weaken the rock bonding. Aggregate characteristics such as particle size, shape, and texture influence hot mix asphalt pavement performance and serviceability. Low resistance to deformation will ensue if the aggregate structure is weak [77].

Aggregate degradation means breakdown of particles into smaller pieces through physical or environmental processes. Degradation causes raveling and instability in pavement surfacing, loss of support in road bases, excessive and differential settlements in pavement foundations [78]. The physical degradation is caused by the forces developed during placing, compacting and finally by the traffic while degradation due to environmental actions are caused by temperature change, moisture content variation, freeze-thaw effect and by chemical processes like hydration, hydrolysis, oxidation, leaching, chemical attack of polluted water and dissolved acids [79]. The extent of degradation of road aggregates depends on several factors e.g. aggregate strength, shape, size, grading, compaction and moisture [80]. High strength aggregates suffer less degradation. For this reason, the initial specified gradation should be coarser for a weak aggregate than that for a strong aggregate [81].

The degradation with good quality aggregates are localized to the surface courses, while with lower quality aggregates, it extends up to the deeper layers [82]. Degradation increases with the increase in compactive effort. It increases either by the increase of the magnitude of the load or by the increase of the number of repetitions of the load. However, the effect of the former is more pronounced. No particular engineering test other than the strength property tests are specified to find the degradation susceptibility of road aggregates. Los Angeles abrasion test, Soundness test, Aggregate crushing test and Aggregate impact test with or without modifications have been done to simulate the expected service life conditions [83]. The qualities of asphalt mixtures can be significantly enhanced by using aggregates with good physical and mechanical properties [84, 85]. High-density aggregates and good crush resistance in asphalt pavements can reduce the chance of aggregates breaking under repeated vehicle loads [86, 87]. Degradation of aggregate materials or granular materials mainly occurs in a form of attrition of asperities, corner breakage, splitting of particles into two or more parts [88].

The gradation is closely related to the quality and performance of the pavement. The aggregates retaining on sieve sizes of 2.36 and 4.75 mm provide more than 50% contribution to resist load and rutting, and the aggregates retaining on sieve sizes of 1.18, 0.6 and 0.3 mm provide more than 50% contribution to strength the structure. The aggregate gradation played a considerable role in resisting the permanent deformation of pavement [89-92]. The rutting resistance increased first and then decreased with regard to the changing of the gradation from fine to coarse [93-95].

The size-dependent performance of rock with regard to physical properties is the main focus of the study. As stipulated by IRC 37-2018 and MORT&H, the performance of rock size gradation used in the construction of flexible pavements is not satisfactory in terms of durability, mainly when applied to sedimentary rocks that has a high aggregate degradation and water absorption value. In Mizoram, most rocks cannot withstand repeated wheel loads in the presence of moisture during the long monsoon rainy season. Hydro-geological factors have also contributed to the longevity of the asphalt pavement.

Rock size specification used in the construction of flexible pavements is supposed to have the shear transmission of stress that should be uniformly distributed throughout the flexible pavement. But, uniform shear transmission is not present in the granular layer when weak rock is present. The ability to resist load has a limit due to shear transmission of the load that is not taking place properly in weak rock, which leads to premature failure. Increased stresses, localized stress concentrations, increased rutting, and decreased drainage are the results when rock aggregate fails before the traffic load is transferred to the adjacent support rock. For this reason, modifications to pavement structure design are necessary.

The investigation results suggested, especially for the friable and easily disintegrated rock, is to use larger stone blocks in flexible pavement to withstand the impact of environmental action and wheel load as the design specification is not performing. In a hilly area with highly degraded rock and heavy rainfall, this study will alter the pavement structure design based on materials response to load and environmental action.

5.2 Condition of the site

The rock sample collected area is within Bhuban Formation of Miocene age and comprises of shale, siltstone and sandstone as shown in the figure 1.5. Mizoram terrain is an immature topography and geologically the arenaceous and argillaceous sequences of sedimentary rocks of tertiary age. The rock types exposed include well laminated iron stained splintery green shale, and silt / shale interlaminations with thick zone of pelagic shale and mudstone [96].

5.3 Materials and Methods

5.3.1 Materials

The aggregate used in this study is obtained from river stones i.e Tuipui and crushed aggregates (crushed stone) obtained from different quarries such as Hlimen, Maubuang, Mausen and Sentezel. Tuipui river stone and Hlimen quarry stone has been used for finding the degradation of aggregate physical properties. Maubuang, Mausen and Sentezel rocks are used for Soundness test which evaluate the resistance of rock against weathering action.

Flexible and rigid pavements are constructed with coarse aggregate. The load transfer capability of pavements is heavily influenced by aggregates. Various properties are tested, including strength, toughness, hardness, and ability to absorb water. The aggregate size includes maximum size, size range, and gradation.

5.3.2 Methods

Different coarse aggregates were selected and divided into four ranges of sizes. Each range of coarse aggregates was tested using physical properties tests such as soundness, aggregate impact value, aggregate crushing value, water absorption and slake durability. To determine the physical properties of rock, tests are performed to determine the performance of coarse aggregate in constructing flexible pavement under various loads, including impact, abrasion, and water absorption.

The aggregate (rock) sizes are divided into the normally adopted size of four ranges of aggregate size as per IS sieve standards: R1 (20 mm-16 mm), R2 (16 mm-12.5 mm), R3 (12.5 mm-10 mm), and R4 (10 mm-4.75 mm) for aggregate impact value, aggregate crushing value, water absorption, and soundness value. The grading A to G is adopted for classification of aggregate size in the Abrasion test as per IS code.

The soundness test is adopted to simulate the physical effects of weathering into the aggregates. It is a chemical test to determine the resistance to disintegration of aggregate by soaking aggregate specimen in saturated solutions of sodium sulphate to accelerate the effect of weathering into the aggregates [97]. Slake durability test is used to assess their resistance to weakening and disintegrating. In this test, rock samples are subjected to two cycles of alternate drying and wetting in a slake fluid, usually water [98].

A rating value is assigned to each parameter ranging from 1, 1.1 to 2.0 and 2.1 to 3.0, which correspond to Low, Medium, High, and Very High levels of aggregate degradation index, respectively [99]. Degradation rating and index value are obtained after giving each parameter a rating number; the rating value of each individual parameter is given a suitable weightage (fixed) to calculate the weighted rating value of each individual parameter.

5.4 Results and Discussions

This study examines how aggregate size affects rock physical properties, as well as the possibility of using a bigger stone in flexible pavements to reduce degradation. The size of the aggregate plays a significant role in its durability, permeability and strength. With high moisture content in the pavement structure, weak aggregate (thinly laminated rock) cannot withstand repeated traffic loads [100]. Some laminated sedimentary rocks have a high water absorption capacity but little weathering resistance mainly due to the presence of clay and silt in the lamination layer. Material behaviour depends on the magnitude, time, nature of load to which the material is subjected and moisture levels. The main cause of failure on weak aggregate pavement is a degradation of rock caused by the presence of water, which saturates the rock and causes it to crumble when wheel load is applied as well as the presence of clay and silt in the small scale sequence of fine rock layer (lamination). As a result of this study, larger aggregate size can be introduced into the base layer of the pavement to counteract moisture-induced and traffic load-induced rock degradation as bigger rock size has equivalent depth of water penetration through the rock lamination regardless of the size and more thickness to resist load.

5.4.1 Influence of aggregate size in aggregate impact value (AIV)

AIV is the test that measures toughness. This test simulate the wheel load pounding/dropping action occurs in the undulation or potholes. As shown in figures 5.1, larger sizes of aggregates have given better result over smaller size of aggregate. It means that a larger size of aggregate has a higher capability to withstand repetitive impact load as the larger aggregates are capable of absorbing more impact load due to more thickness. At pavement undulations, impact loads frequently lead to the degradation of soft rock.

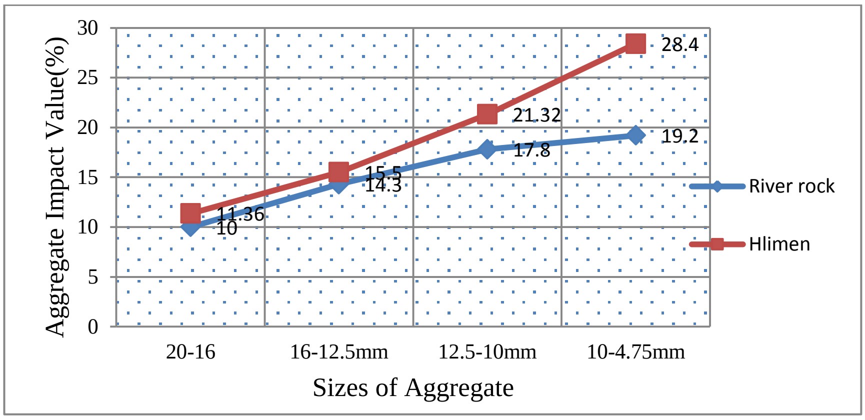

Fig. 5.1 AIV of River rock and Hlimen with different size of rock

5.4.2 Influence of aggregate size in water absorption value (WAV)

High rainfall during the wet seasons causes moisture ingress through the pavement surfaces. The use of dense graded unbound type pavement foundation layers and absence of proper drainage facility lead massive moisture related damage to the pavements and consequent premature failure. The locally available material, stone aggregate, degrades highly in presence of moisture.

Water absorption values (WAV) measures the ratio between difference of dry weight aggregate and saturated surface dry weight of aggregate to dry weight of aggregate. Larger sizes of aggregates have given lower absorption values compared to smaller sizes as shown in figures 5.2. This may be due to the depth of water penetration to the rock remain the same regardless of the size within a specific period of time. The quantity of water absorbed by smaller and larger size of aggregates remains the same.

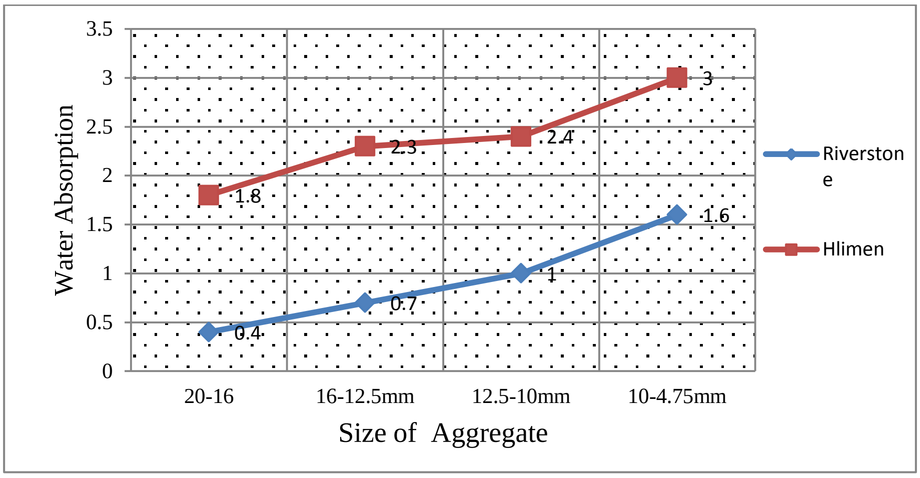

Fig. 5.2 WAV of River stone and Hlimen with different size of rock

5.4.3 Influence of aggregate size on Los Angeles abrasion value (LAAV)

The percentage degradation (wear) of the sample aggregates due to rubbing with steel balls and the falling impact of balls are determined as a percentage in the observation value. LAAV test involves two kinds of action i.e impact (pounding force) and abrasion (frictional force). In flexible pavement, these two action forces reflect the actual conditions of the site at some extent. It has been observed that gradations with a higher sieve size (coarser aggregates) have a lower level of abrasion, as shown in figure 5.3. The time required to wear away a larger size of the rock is more compared to the time required to disintegrate the smaller size of aggregate. LAAV test involves the combination of different gradation sizes of aggregate as shown in table 5.1. At a larger particle size, the desired value of abrasion is obtained.

The lower abrasion value indicates less degradation. The largest particle size mixture, grade E, has less degradation, while the smallest particle size mixture, grade D, has more degradation as shown in the figure 5.3. As the particle size increases, the abrasion value decreases.

Table.5.1 Gradation of aggregates for LAAV

Sieve Size		Weight in gm of test sample for grade						
Passing (mm)	Retained (mm)	A	B	C	D	E	F	G
80	63					2500		
63	50					2500		
50	40					5000	5000	
40	25	1250					5000	5000
25	20	1250						5000
20	12.5	1250	2500					
12.5	10	1250	2500					
10	6.3			2500				
6.3	4.75			2500				
4.75	2.36				5000			
No of Spheres		12	11	8	6	12	12	12
No. of Revolution		500	500	500	500	1000	1000	1000

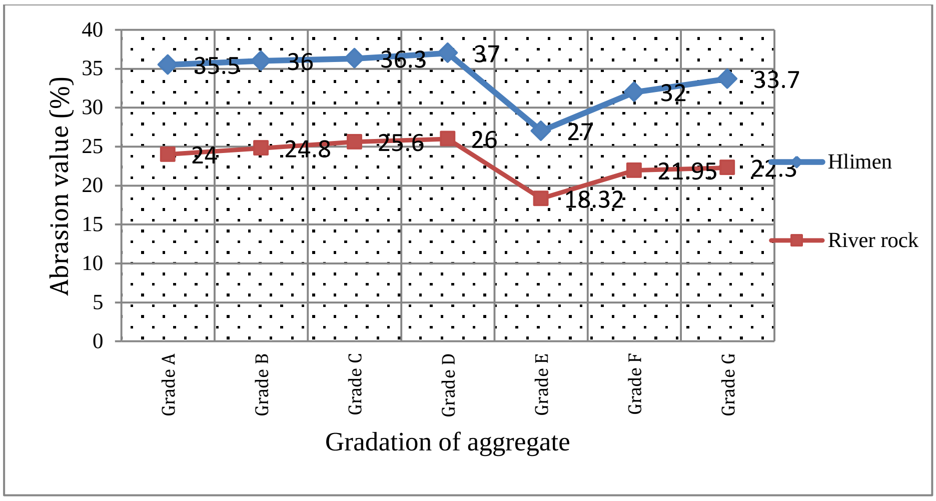

Fig.5.3 LAAV of Riverstone and Hlimen with different gradation of aggregate

5.4.4 Influence of aggregate size on aggregate crushing value (ACV)

Aggregate Crushing Value is used to measure the strength of aggregate. As the size of aggregate gets larger, the strength of aggregate increases. The larger size has the ability to resist more loads (traffic load) due to individual thickness of aggregate that imparts more strength as shown in figure 5.4.

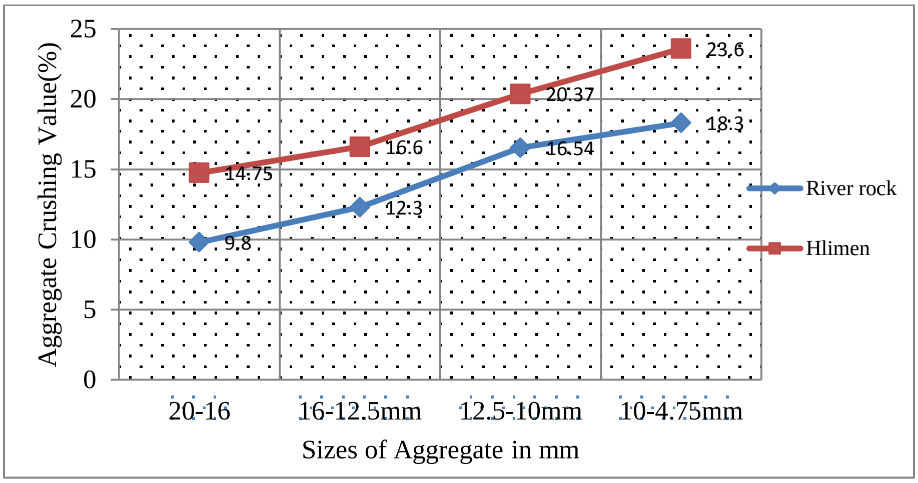

Fig. 5.4 ACV of Riverstone and Hlimen with different size of rock

5.4.5 Influence of aggregate size on degradation due to weathering action

The weathering resistance of the rock is measured using soundness and slake durability test. Soundness tests simulate the physical effects of weathering on aggregates. Aggregates may degrade and disintegrate due to weathering, leading to early pavement distress. Aggregates from the Mausen, Maubuang, and Sentezel rock used in the construction of State Highway were collected for conducting soundness. Sodium sulphate was used to test the soundness of the coarse aggregate taken. The weight loss caused by weathering has been checked after five cycles of repetition. It is found that larger particle size has a lower weight loss value, whereas a smaller aggregate tends to exceed the permitted limit of soundness.

The value for soundness for various aggregate sizes is displayed in Table 5.2

Table 5.2 Soundness value for various aggregate sizes

Sample	Particle Size (mm)	Percentage (%) loss of weight after 5 cycle	Permissible limit as per IS 2386 Part V, & MoRT&H 4th revision
Maubuang	40-20	17.33	<12% with sodium sulphate, <18% with magnesium sulphate after [Sodium sulphate is used in this test.]
	20-10	21.00	
	10-4.75	26.67	
Mausen	40-20	7.8	
	20-10	8.5	
	10-4.75	16.67	
Sentezel	40-20	12.27	
	20-10	14.60	
	10-4.75	19.67	

Slake durability Index (SDI) test measures the influence of weathering on the durability of rock. The Hlimen and Tuipui river rock has been tested as per IS 10050-1981 for slake durability. As shown in table 5.3, the durability value increases with rock size.

Table 5.3 Durability value and weathering grade for various aggregate sizes

Name of the Source	**Rock Size**	**Id_2 (%)**	**Durability classification as per IS 10050-1981**	**Weathering Grade (ISRM)**
Tuipui river sandstone	20-16 mm	96.4	Extremely High	Un-weathered
	16-12.5mm	95.8	Extremely High	Un-weathered
	12.5-10mm	95.4	Extremely High	Un-weathered
	10-4.75mm	93.8	Very High	Partially weathered
Hlimen Sandstone	20-16 mm	95.8	Extremely High	Un-weathered
	16-12.5mm	93.12	Very High	Partially weathered
	12.5-10mm	92.6	Very High	Partially weathered
	10-4.75mm	91.2	Very High	Partially weathered

Soundness and Slake durability values decrease with increasing particle size primarily because of the lesser surface area of the same volume, where weathering action has less of an impact than it would on a smaller-sized rock, which has larger surface area per unit volume.

5.4.6 Aggregate Degradation Rating and Index

Aggregate Degradation Index is the indicator for the magnitude of disintegration. In addition to reducing particle angularity, surface texture, and size, aggregate degradation also reduces aggregate materials grade and shear strength. The disintegration of rock caused by repeated impact loads, and abrasive force, has changed the gradation. Gradation as per design is lost shortly after the application of load on the pavement because of aggregate degradation. In this situation, the larger particle size of the rock will slow down the disintegration process, especially for brittle and soft rock.

A rating value is assigned to each parameter ranging from 1, 1.1 to 2.0 and 2.1 to 3.0, which correspond to Low, Medium, High, and Very High levels of aggregate degradation index, respectively. After giving each parameter a rating number, the rating value of each individual parameter is given a suitable weightage (fixed) to calculate the weighted rating value of each individual parameter.

The weighted rating values of all degradation test parameters, such as AIV, ACV, LAAV and WA are averaged to obtain the final rating value. The following fixed weightages and rating value have been assigned for the computation of the Weighted Rating Value of each individual parameter as stated in table 5.4. The weighted rating value is calculated with the value obtained from aggregate degradation tests as shown in table 5.5 as well as the rating value and aggregate degradation index.

Table 5.4 Aggregate Degradation based Rating for pavement

Parameters	**Ranges (%)**				**Weightage value**
AIV	0-10	10-20	20-30	>30	1
ACV	0-10	10-20	20-30	>30	0.8
LAAV	0-20	20-30	30-40	>40	1.1
WA	0-2	2-3	3-5	>5	1.1
Rating	3-4	2.1-3	1.1-2	0.5	
Degradation Index	Low	Medium	High	Very High	

Table 5.5 Aggregate Degradation Index and Rating value for Tuipui and Hlimen rock

Source of Rock	**Tuipui**				**Hlimen**			
Gradation	20-16 mm	16-12.5mm	12.5-10mm	10-4.75mm	20-16 mm	16-12.5mm	12.5-10mm	10-4.75mm
AIV	10	14.3	17.8	19.2	11.36	15.5	21.32	28.4
ACV	9.8	12.3	16.54	18.3	14.75	16.6	20.37	23.6
LAAV (Grade-B)	14	14.8	15.6	16	35.5	36	36.3	37
WA	0.4	0.7	1	1.6	1.8	2.3	2.4	3
Weighted Rating Value	3.3	2.93	2.69	2.49	2.47	2.21	1.99	1.55
Aggregate Degradation Index	Low	Medium	Medium	Medium	Medium	Medium	High	High

The aggregate degradation rating value is increasing with increase in rock size. This shows that rocks with larger particle sizes have a greater ability to withstand degradations caused by abrasion, repeated impact, crushing, and the presence of moisture.

Regression equation was developed to predict the Aggregate degradation rating value with respect to aggregate degradation test such as impact, crushing, abrasion and water absorption.

Aggregate Degradation Rating value:

$$AD = 4-0.023*AIV-0.029*ACV-0.011*LAA-0.23*WA \quad (R^2 = 0.99)$$

Where,

AD =*Aggregate degradation rating value*

AIV = *Aggregate Impact Value*

ACV = *Aggregate Crushing Value*

LAA = *Los Angeles Abrasion*

WA = *Water Absorption*

5.5 Summary

Based on the resilient data obtained from the investigation, the following conclusions can be drawn:

The larger size of aggregate has been found to have higher toughness (impact), strength (crushing), abrasion and weathering resistance than the smaller size of aggregate. Larger aggregate size has higher capability of absorbing more repetitive impact load, higher strength due to individual thickness of aggregate and less water absorption due to reduced surface area compared to smaller size over the same volume of rock.

The early degradation of aggregate is mostly caused by environmental factors such as moisture and temperature variations, in addition to wheel load. The rate at which aggregate degradation occurs is influenced by rock size. Abrasive and weathering effect on aggregate is slowed down by larger particles because they degrade away more slowly in the same volume of rock compare to smaller rock.

The aggregate degradation rating value is increasing with increase in rock size. This shows that rocks with larger particle sizes have a greater ability to withstand degradations caused by abrasion, repeated impact, crushing, and the presence of moisture.

The nature of the rocks, being sedimentary and having high water absorption of Hlimen rock, has shown a lower value of physical properties of rock under the application of load compared to Tuipui river rock. The river rock has a better resistance against degradation and weathering action as it is the remaining pieces which endure the extreme environmental condition.

Along with the wheel load, environmental factors including moisture and temperature changes are the primary causes of the early degradation of aggregate. Adoption of larger size aggregate in the pavement base course is suggested to withstand the effect of environmental action and wheel load, especially on the friable and quickly disintegrated rock.

CHAPTER 6
WATER SENSITIVITY ANALYSIS ON LAMINATE SEDIMENTARY ROCKS AGGREGATE COATED WITH PLASTIC WASTE IN BITUMINOUS MIXTURE

6.1 Introduction

Laminated sedimentary aggregate is the broken pieces of sedimentary rock which has a small scale sequence of fine layer. The road constructed in the areas where the majority of the closely laminated sedimentary rock degrades prematurely owing to moisture and load application requires an alternative method. Degradation of aggregate results in ravelling, instability, and settlement of pavement structure. In the construction of roads, aggregate plays a significant role in defining the pavement. It also constitutes the bulk of the material for the various pavement layers in different sizes. They form the basis of the structural member in the pavement as they have to support loads of the overlying wheel loads. The main cause of pavement failure is a degradation of sedimentary rock caused by the presence of water, which saturates the rock and causes it to crumble when wheel load is applied as well as the presence of clay and silt in the small scale sequence of fine rock layer (lamination). To be utilized as pavement material, coarse aggregate must meet certain requirements for strength, hardness, toughness, and disintegration due to weathering [101].

The use of plastic waste in road construction acts as an alternative for its ultimate disposal. It thus significantly helps in the management of plastic waste, as the amount of plastic waste is constantly growing and its ultimate disposal is a primary environmental concern due to its non-biodegradability. When aggregates are used to build roads, plastic waste is added to modify the aggregate characteristics [102-104]. The plastic added to the bituminous mix improves the stability, durability and fatigue life [105-107]. This research concentrates on using plastic trash to coat aggregates to improve their impact, abrasion, specific gravity, and water absorption qualities, which are all relevant to their use in road construction. It is a significant problem to dispose of waste plastic. Most of it is made of low-density polyethylene. The bituminous mix performs better when waste plastic is added [108-110]. The laboratory performance studies of bituminous mixes were conducted to determine their utility in road construction.

Gradation enhances material performance and the bonding capability of the mix. Material gradation must improve the hardness of the surface course [111]. It is critical to understand the Nominal Maximum Aggregate Size (NMAS) since they will affect performance and should be chosen depending on pavement requirements [112]. When compared to smaller sizes over the same volume of rock, larger aggregate sizes can withstand more repetitive impact stress, have stronger strength due to the individual thickness of the aggregate, and have less water absorption due to the reduced surface area. The existence of larger aggregate sizes aids in the attainment of greater density and strength [113].

6.2 Materials and Methods

6.2.1 Materials

Bitumen, Low-density polyethylene (LDPE) plastic waste, coarse aggregate, and fine aggregate are the components of the bituminous mix.

6.2.2 Methods

The following test are conducted to determine the water sensitivity on asphalt concrete and effect of plastic on the physical properties of aggregate and bituminous concrete. The physical properties of rock test such as the Aggregate Impact Value (AIV), Los Angeles Abrasion Value (LAAV), Specific Gravity (SG), Water Absorption (WA), and bitumen and asphalt concrete test such as ductility, penetration, softening point, Marshall stability value, Indirect tensile test were conducted.

The plastic coated aggregate performance on its physical properties has been checked using coarse aggregate physical tests. Six samples were prepared for each addition of plastic content by weight of aggregate. The results of the MORT&H-specified process for determining softening point, penetration value, specific gravity and ductility value are summarized in tables 6.1 and 6.2. Figure 6.1 shows a plot of the Bituminous Grade II gradation.

Marshall bituminous mixes of Bituminous Concrete (BC) grade II samples were prepared using a dry technique. Six samples were prepared for every different mix proportion and each of the different percentage of bitumen or plastic. Bituminous Concrete Grade II was selected in this study. Plastic waste (Low-density polyethylene) was selected as it most used in the study area. Shredded plastic waste was added to the heated aggregate at the temperature of 150-170^{o}C. The mix between the bitumen, aggregate and plastic was heated at the temperature of 150-165^{o}C. The samples were compacted at the temperature of 90^{o}C. Bituminous mix samples were prepared using the Marshall Stability test procedure. By varying the proportion of plastic content, the best plastic content is being produced independently in bituminous mix and aggregate alone.

Water Sensitivity test were used to evaluate water susceptibility and tensile strength and retained tensile strength of the bituminous concrete and the influence of rock gradation on BC. The size effect (different gradation) of coarse aggregate in bituminous concrete in relation to moisture sensitivity is also studied. Preparations of BC grade I & II for Marshall Stability and Tensile strength was done as per IS guidelines.

Table 6.1 Properties of Bitumen

Property	Test Method	Test Results	IS 73:2013 for VG 30
Penetration 25°C , 5s, 0.1mm	IS 1203-1978	51	Min 45
Softening point (R & B),°C	IS 1205-1978	47.6	Min 47
Ductility Test, 25°C, cm	IS 1208-1978	99	Min 75
Specific gravity	IS 1202-1978	1.0	-

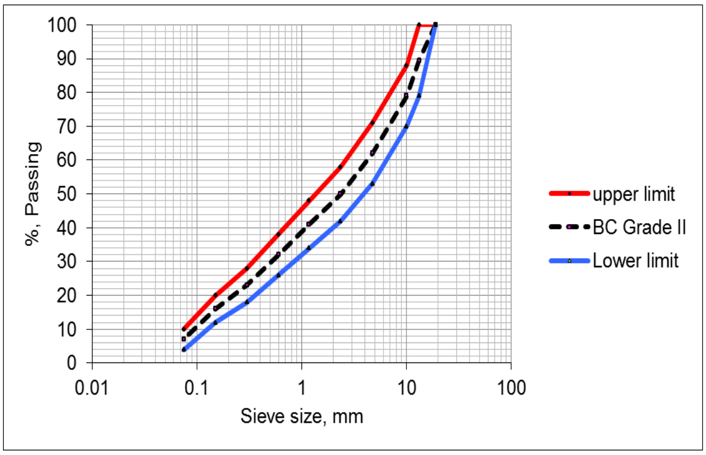

Fig 6.1 Gradations adopted for BC Grade-II

6.3 Influence of plastic coating on the properties of coarse aggregate

Thinly laminated sedimentary rocks are susceptible to weathering. The lamination layer, where clay and silt are present, is the weakest point in the rock structure. Water can penetrate the laminated layer and weaken the rock bonding. The aggregate strength is influenced by the laminated layer depth of rock. The strength of aggregate increases as the depth of the laminated rock layer increases. As demonstrated in table 6.2, a plastic covering on the aggregate surface reduced the values for impact, abrasion, and water absorption. The plastic acts as a binder that firmly bonds the particles together, increasing the coarse aggregate toughness (impact), hardness, abrasion resistance, and water absorption as well as seal the lamination layer of aggregate. The plastic coating stops the aggregate from absorbing water, hence reducing the specific gravity of the material. The aggregate ability to absorb water has reduced as plastic content has increased due to a thin, impermeable plastic layer.

The ideal plastic content for aggregate that has been coated with plastic is 9% of the mass of the aggregate. 9 % is the ideal amount of waste plastic out of total weight to use in plastic coated aggregate. It has been found that adding more plastic to aggregate in excess of 9% by weight does not improve the aggregate characteristics.

Table 6.2 Physical properties of aggregate with and without plastic coating

Properties of Aggregate	**Addition of Plastic content by weight of aggregate**					
	0%	**6%**	**7%**	**8%**	**9%**	**10%**
AIV	17.72%	9.60%	8.09%	6.62%	5.31%	5.98%
LA&AV	42.93%	35.71%	34.18%	32.65%	31.45%	32.13%
WA	4.52%	2.83%	1.41%	0.47%	0.46%	0.94%

6.4 Performance of modified bitumen with plastic in bituminous mix

Some laminated sedimentary rocks have a high water absorption capacity but little weathering resistance mainly due to the presence of clay and silt in the lamination layer. This research examined the use of plastic wastes to improve the performance of weak aggregates used in road construction by modifying bitumen. An attempt is made to increase the performance of weak aggregates by coating them with plastic and to determine the effect of replacing bitumen with plastic in bituminous mixes. Individual values of optimum bitumen contents are obtained considering the average value of maximum density, maximum stability, mid-range of recommended flow value and mid-range of recommended flow voids content as shown in figure 6.2 to 6.7. The optimum bitumen content obtained is 5.5% as shown in table 6.4. Bitumen mixed with plastic provides better results in Marshall Stability value of Bituminous Concrete Grade - II compared to bitumen as a sole binder as shown in the table 6.3 and 6.6.

Table 6.3 Optimum Bitumen Content for BC-II (uncoated aggregate)

BC Grade-II	% bitumen	Gb	Gt	Vv (%)	Vb (%)	VMA (%)	VFB (%)	Stability (kg)	Flow (mm)
S1	4.5	2.31	2.49	7.46	10.18	17.65	57.70	642.9	4
S2	5	2.39	2.48	3.42	11.73	15.14	77.44	696.5	3.3
S3	5.5	2.39	2.46	2.92	12.88	15.80	81.53	857.3	3
S4	6	2.43	2.44	0.57	14.29	14.86	96.19	771.5	2

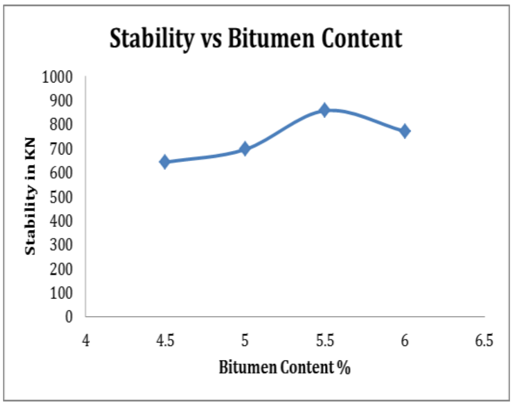

Fig 6.2 Stability versus bitumen content

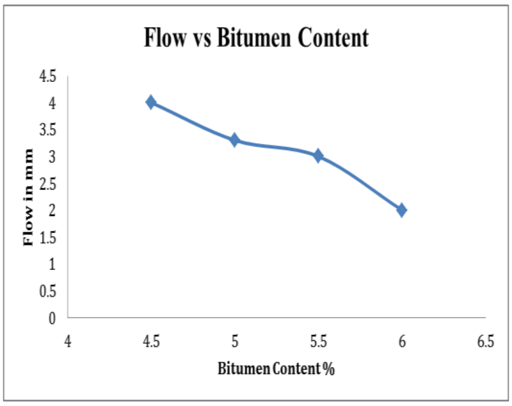

Fig 6.3 Flow versus bitumen content

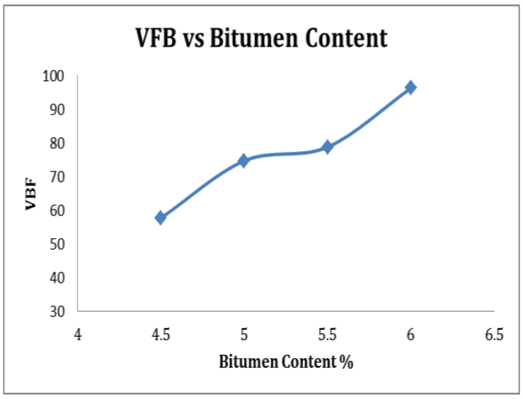

Fig. 6.4 Void filled bitumen vs bitumen content

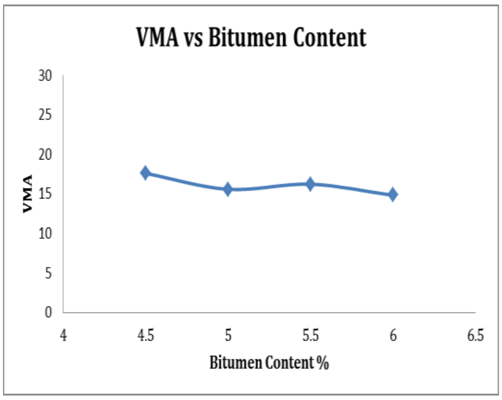

Fig.6.5 Void in mineral aggregate vs bitumen content

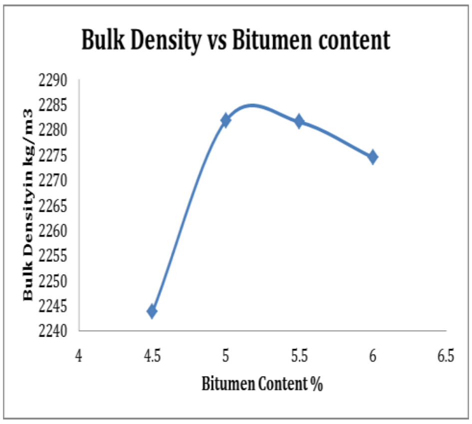

Fig. 6.6 Bulk density versus bitumen content

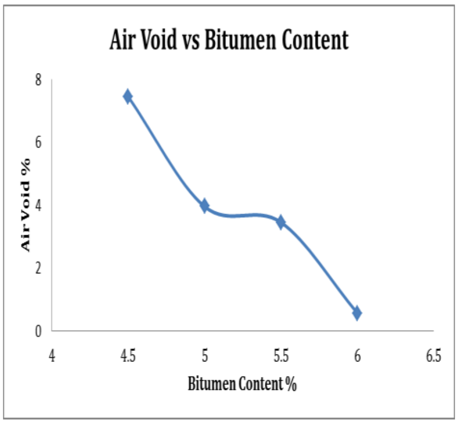

Fig. 6.7 Air Void versus bitumen content

Table 6.4 Optimum Bitumen Content of BC-II

Determination of Optimum Binder content for BC-II by Marshal Stability Method	**Bitumen %**
Maximum Marshal Stability at bitumen content at peak	5.5
Maximum density at Bitumen content at Peak	5.5
% Air Voids(Vv) at 4%	5.5
VFB (Voids Filled with Bitumen) at 70%	5
Flow at 3 mm	5.4
VMA (Voids in Mineral Aggregates) at 12-15%	5.3
Optimum Bitumen Content	**5.5**

It is also possible to completely replace bitumen with plastic as the sole binder. The disadvantage of using plastic as a sole binder is that it is difficult to compact at low temperatures since plastic solidifies quickly. Early solidification of plastic may lead to the compaction problem in the field.The bitumen as a sole binder has also a high VFB and VMA, which are out of limit as per MORT&H as shown in the table 6.5.

Table 6.5 Optimum Plastic Content for BC-II (Full replacement of bitumen)

BC-II (only plastic)	% plastic	Gb	Gt	Vv (%)	Vb (%)	VMA (%)	VFB (%)	Stability (kg)	Flow (mm)
S1	**7**	2.2	2.4	8.1	15.2	23.3	65.3	697.0	4.0
S2	9	2.3	2.4	1.6	20.4	22.0	92.9	866.9	3.0
S3	11	2.3	2.3	1.4	24.4	25.9	94.5	1299.9	3.2
S4	13	2.2	2.2	1.3	28.3	29.6	95.7	1192.0	4.1
S5	15	2.2	2.2	1.2	32.0	33.2	96.4	942.7	5.0
S6	17	2.1	2.2	1.0	35.6	36.7	97.2	845.2	5.3
S7	20	2.1	2.1	0.9	40.8	41.7	97.8	433.4	6.0

The disadvantage of using plastic as a sole binder is that it is difficult to compact at low temperatures since plastic solidifies quickly. A Marshall Stability value of 12.99 KN was obtained with an 11 percent plastic content but failed to achieve the standard requirement and degree of workability when plastic served as the only binder in the mixture.

Table 6.6 Partial Replacement of Bitumen with Plastic in BC-II

BC Grade-II (Bitumen + Plastic)	% of mix	Gb	Gt	Vv (%)	Vb (%)	VMA (%)	VFB (%)	Stability (kg)	Flow (mm)
S1 (85%+15%)	5.47	2.3	2.5	8.1	12.2	20.3	60.2	914	4
S2 (80%+20%)	5.47	2.3	2.5	7.6	12.3	19.8	61.8	931	3.5
S3 (75%+25%)	5.47	2.4	2.5	2	13	15	86.9	1064	3.3
S4 (70%+30%)	5.47	2.3	2.5	6.4	12.4	18.8	66.2	897	2.2
S5 (65%+35%)	5.47	2.4	2.5	3.9	12.8	16.7	76.5	865	2.1

The partial replacement of bitumen with Plastic in BC-II increases the performance of the bituminous mix as shown in the table 6.6. With a replacement of bitumen using plastic by 25%, bituminous concrete (Grade II) has the highest Marshall Stability at 10.64 KN, but only 8.57 KN when bitumen is only used as a binder.

Table 6.7 Addition of Plastic in Bituminous mix of BC-II

BC Grade I	% binder	Vv (%)	Vb (%)	VMA (%)	VFB (%)	Stability (kg)	Flow (mm)
S1(5.5%+0.28% Plastic)	5.78	5.60	13.11	18.71	70.07	1682.75	3.50
S1(5.5%+0.55% Plastic)	6.05	7.87	13.35	21.22	62.90	3803.33	3.00
S1(5.5%+0.83% Plastic)	6.33	8.27	13.85	22.12	62.62	4314.83	2.50
S1(5.5%+1.1% Plastic)	6.6	7.88	14.45	22.34	64.71	4675.19	2.00
S1(5.5%+1.38% Plastic)	6.88	6.97	15.16	22.13	68.51	4400.55	3.00

Addition of Plastic in Bituminous mix of BC-II with addition of 1.1% of plastic by weight of the mix has given the better results as shown in the table 6.7.

6.5 Moisture sensitivity analysis of Bituminous Concrete

Flexible pavements have suffered from an increased rate of damage due to the effects of water. The damage is the result of a lack of cohesion within the mixture caused by a loss of bond strength between asphalt cement and aggregate. This moisture damage mechanism is sometimes referred to as stripping.

The tensile strength ratio of bituminous mixtures indicates their resistance to moisture susceptibility and a measure of water sensitivity. A higher tensile strength ratio (TSR) or retained tensile strength value indicates good resistance to moisture. The higher the TSR value, the lesser will be the strength reduction by the water soaking condition, or the more water-resistant it will be.

Bituminous Concrete grade II of 19 mm and 13 mm maximum nominal size of aggregate has been tested with Indirect Tensile Strength test. It is found out that the material i.e., aggregate used in the bituminous mix is susceptible to water, since the rock aggregate is laminate sedimentary sandstone.

6.5.1 Effect of gradation in water sensitivity of BC

A bituminous concrete with a finer gradation (13 mm NMAS) provides a better tensile strength over coarser gradation (19 mm NMAS) as the bituminous mix can be packed more densely due to the finer aggregate particles as shown in the figure 6.8. Meanwhile, a larger size of gradation, (i.e., 19 mm max. nominal size) has achieved higher retained tensile strength over 13 mm gradation, mainly due to the less degradation that develops in a bituminous structure.

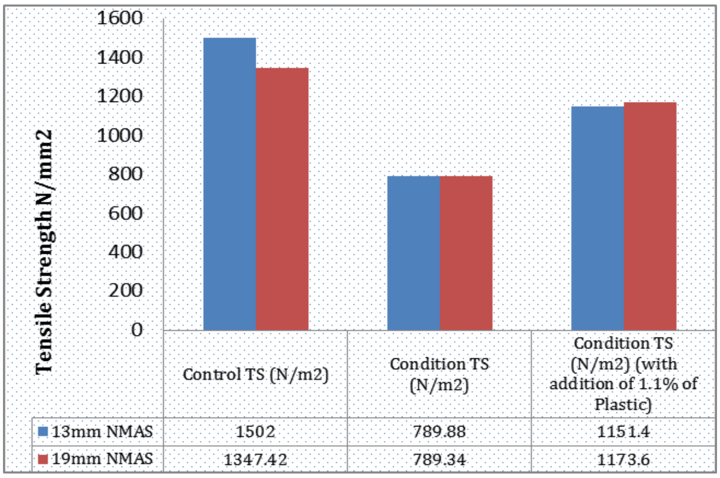

Fig 6.8 Tensile Strength of different gradation of BC

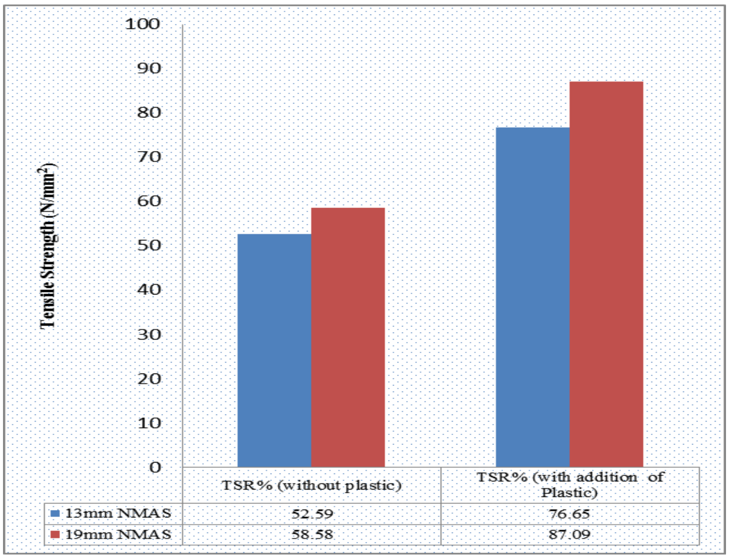

Fig 6.9 Retained tensile Strength of BC

The reduction of tensile strength after asphalt concrete sample undergoes the freeze and thaw cycle (weathering action) both in 19 mm and 13 mm gradation is mainly due to high water absorption of aggregate as well as the presence of small sequence of rock layer.

6.5.2 Influence of addition of plastic in water sensitivity of BC

The minimum specified value of Retained Tensile Strength is 80 % as per MORTH Specification. This criteria is not fulfilled in the present study mainly due to

1. High aggregate degradability under weathering action.
2. High water absorbability
3. The layer (lamina) within the rock aggregate

The rock with high water absorbability and thinly lamination has allowed the water to penetrate through the fine sequence of layer and weaken the bonding of the rock structure.

There is significant reduction in tensile strength for both gradations after BC sample undergoes the freeze and thaw cycle. The reduction of strength is mainly due to the high water absorption of aggregate as well as the presence of small sequence in rock aggregate. The BC is susceptible to water as the retained tensile strength value is less than the prescribed value of 80% due to the presence of highly degradable of rock used.

In order to improve the retained tensile strength, 6.6 % of binder (5.5% bitumen +1.1%plastic) has been added to the bituminous mix. It has been found that the retained tensile strength value in 13 mm and 19 mm NMAS of BC are 52.59 % and 58.58 % respectively without the addition of plastic; meanwhile, 82.81 % and 92.60% respectively with the addition of plastic in a bituminous mix has been obtained as shown in the figure 6.9.

6.6 Summary

The flexible pavement can deteriorates rapidly due to the presence of closely laminated sedimentary aggregates (rock) sensitivity to water. The lamination layer, where clay and silt are present, is the weakest point in the rock structure. The laminated layer can be penetrated by water and weaken the bond between the rock. It has been demonstrated that modified bitumen with plastic increase the asphalt performance. As a result, the following conclusions have been drawn:

The plastic acts as a binder that firmly bonds the particles together and seal the lamination layer of aggregate, increasing the coarse aggregate toughness (impact), hardness, abrasion resistance, and decrease the water absorption.

The optimum amount of plastic waste to be used in plastic-coated aggregate is 9%. Plastic added in excess of 9% by weight in the aggregate and 11% in the bituminous mix do not further improve the performance of the aggregate. Plastic as a sole binder is not suggested as the mix does not meet the desirable requirement and workability. The partial replacement of bitumen with plastic in BC-II also increases the performance of the bituminous mix. A better material performance has been achieved by using plastic wastes as a coating agent on aggregate surfaces and as a binder in bituminous mixes.

A bituminous concrete with a finer gradation (13 mm NMAS) provides a better tensile strength over coarser gradation (19 mm NMAS) as the bituminous mix can be packed more densely due to the finer aggregate particles. Meanwhile, a larger size of gradation, (i.e., 19 mm max. nominal size) has achieved higher retained tensile strength compare to 13 mm gradation, mainly due to the less degradation that develops in a bituminous structure.

The reduction of tensile strength after asphalt concrete sample undergoes weathering action both in 19 mm and 13 mm gradation is mainly due to high water absorption of aggregate as well as the presence of small sequence of rock layer. Laminate sedimentary rock aggregate used in asphalt concrete could not achieve the minimum retained tensile strength value mainly due to the low resistivity of weathering action.

So, the desirable retained tensile strength of BC has achieved by the addition of 1.1 % plastic by weight of the mix into the bituminous mix. The retained tensile strength values in 13 mm and 19 mm NMAS of BC are 52.59 % and 58.58 % respectively without the addition of plastic; meanwhile, 82.81 % and 92.60% respectively with the addition of plastic have been obtained in a bituminous concrete.

The use of larger aggregate sizes and the addition of plastic in the bituminous mix, which increases the retained tensile strength of bituminous concrete, can improve the performance of rock aggregate under moisture and wheel load.

CHAPTER 7
DEVELOPMENT OF SUITABLE PAVEMENT CONSTRUCTION METHOD FOR PREMATURE FAILURE SECTION

7.1 Introduction

Materials fail due to fracture, permanent deformation, degradation due to durability issues. Material response varies with applied load (state of stress), duration of load, temperature, moisture, stress and age dependent properties.

Fundamental material behavior is usually characterized in terms of stress-strain relationship, ability of the material to recover after release of load, time dependency and temperature dependency. Material behaviour can be Linear or Non-linear, Elastic or Plastic, Viscous or Non-viscous or combination of them. It is also depends on the magnitude, time, nature of load, temperatures to which the material is subjected and moisture levels. With increasing stress, a rock deforms elastically, then plastically, before ultimately failing or breaking. A completely brittle rock fails at its elastic limit. For example, if the rocks exhibit an elastic response, the pavement can experience fatigue cracking due to repeated wheel loads. If the rocks exhibit a plastic response, the pavement can experience permanent deformation, which can lead to rutting. If the rocks exhibit a brittle response, the pavement can experience cracking and loss of support. [114].

The most important factors that influence the performance and distress development of the pavement structures are the cross section of the road, the climatic conditions, the traffic (axle) loading of the structure and material properties of the different layer in the pavement structure. Bituminous concrete shows complex non-linear viscoelastic plastic behavior under external loading. Bituminous bounded materials are temperature dependent and the response of unbound material depends on the moisture content [115-121].

7.2 Response models for flexible pavements

Responsive value measures the reaction of the supporting layer against traffic load and can be also defined as resistance to deformation. Deflection is caused by the elastic compression of the subgrade and other pavement layers when the wheel load is applied over it as shown in the figure 7.1. The response value is altered when the pavement layers are compressed and variation of response value also depends on modulus of elasticity, pavement thickness, moisture content and strength of the subgrade.

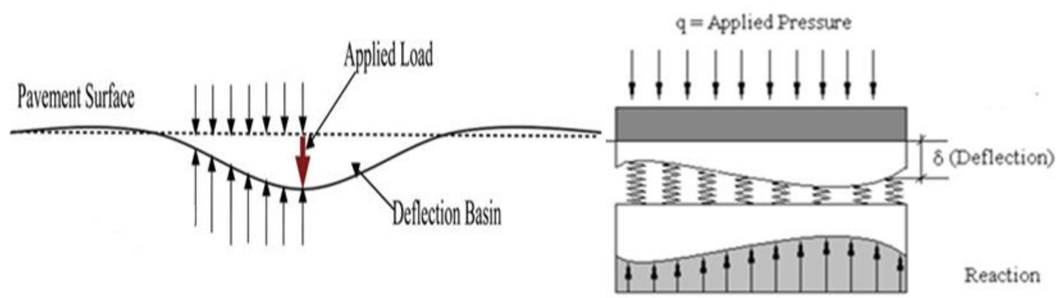

Fig 7.1 Pavement responses to wheel load

(http://publisher.uthm.edu.my/periodicals/index.php/rtcebe)

Pavement design methods can be classified into two broad categories: empirical methods and mechanistic methods. The empirical methods are developed based on the observed performance of actual test roads with known pavement materials and structures subjected to certain traffic and environmental loads. The mechanistic methods are based on fundamental laws of physics and strength of materials. There has been a movement away from the traditional empirical design approach toward mechanistic design procedures because of the serious limitations of empirical methods including deficiencies when accommodating heavier vehicle loads, new material properties, climatic effects, and an inability to handle long-life pavement designs.

Numerous mechanistic pavement response models have been developed over the years, ranging from Boussinesq's one-layer model, to multi-layer elastic theories to finite element models. The response models which are based on multi-layer elastic theory and displacement-based finite element methods are currently the most widely used and both are adopted as the structural response models in the MEPDG [122-124].

Pavement structural analysis includes three main issues: material characterization, theoretical model for structural response, and environmental conditions. Three aspects of the material behavior are typically considered for pavement analysis [11]:

(1) the relationship between the stress and strain (linear or nonlinear);

(2) the time dependency of strain under a constant load (viscous or non-viscous);

(3) the degree to which the material can recover strain after stress removal (elastic or plastic).

Theoretical response models for the pavement are typically based on a continuum mechanics approach. The model can be a closed-formed analytical solution or a numerical approach. Various theoretical response models have been developed with different levels of sophistication from analytical solutions such as Boussinesq's equations based on elasticity to three-dimensional dynamic finite element models. Environmental conditions can have a great impact on pavement performance. Two of the most important environmental factors included in pavement structural analysis are temperature and moisture variation. Frost action, the combination of high moisture content and low temperature can lead to both frost heaves during freezing and then loss of subgrade support during thaw significantly weakening the structural capacity of the pavement leading to structural damage and even premature failures.

7.2.1 Single Layer Model

Boussinesq (1885) was the first to examine the pavement response to a load. A series of equations was proposed by Boussinesq to determine stresses, strains, and deflections in a homogeneous, isotropic, linear elastic half space with modulus E and Poisson's ration ν subjected to a static point load P. This single layer model may be the simplest way to model a pavement structure.

7.2.2 Burmister's Two-layer Elastic Models

Pavement systems typically have a layered structure with stronger/stiffer materials on top instead of a homogeneous mass as assumed in Boussinesq's theory. Therefore, a better theory is needed to analyze the behavior of pavements. Burmister (1943) was the first to develop solutions to calculate stresses, strains and displacement in two-layered flexible pavement systems.

7.2.3 Multi-layer Elastic Model

To attain a closer approximation of an actual pavement system, Burmister extended his solutions to a three-layer system and derived analytical expressions for the stresses and displacements. Many elastic layered computer programs have been developed based on the multi-layered elastic theory since then. Some of the programs that have been widely used in pavement analysis and design are CHEVRON, BISAR, ELSYM5, KENLAYER, and WESLEA. Raad and Figueroa developed a 2-D finite element program called ILLIPAVE to model the flexible pavement behaviors [125]. Nonlinear constitutive relationships were used for pavement materials and the Mohr-Coulomb theory was used as the failure criterion for subgrade soil in ILLIPAVE.
Modeling the behavior of materials under pavements, specifically their response to loads and other prevailing conditions, has been a subject of intensive research.

The pavement design method called as Mechanistic Empirical Pavement Design (MEPD) is developed trying to captures the true behavior of a pavement structure, i.e., its response (induced stresses, strains and deflections) to applied loads and environmental conditions. It is an approach to designing pavement structures that takes into account the physical properties and behavior of pavement materials and the pavement structure as a whole. It is a combination of mechanistic and empirical design methods that allows designers to simulate pavement performance under various conditions and predict how the pavement will behave over time as shown in table 7.1 and figure 7.2. Empirical design methods are based on limited data from a specific region or location. Therefore, they may not be applicable to other regions with different soil, rock and weather conditions as well as often unable to incorporate new materials or technologies that may improve the performance of the pavement [126].

Table 7.1 Critical Analysis Locations in a Pavement Structure

Location	Response	Reason for Use
Pavement Surface	Deflection	Used in imposing load restrictions during spring thaw and overlay design (for example)
Bottom of HMA layer	Horizontal Tensile Strain	Used to predict fatigue failure in the HMA
Top of Intermediate Layer (Base or Sub-base)	Vertical Compressive Strain	Used to predict rutting failure in the base or sub-base
Top of Subgrade	Vertical Compressive Strain	Used to predict rutting failure in the subgrade

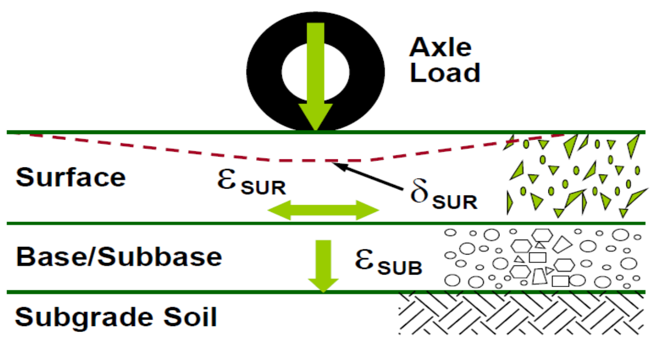

Fig 7.2 Typical cross section of flexible pavement (https://www.researchgate.net/publication/321009054)

7.3 Method

IITPAVE software is an improved version of FPAVE which is developed by research scheme R-56 of MORTH. The mechanistic empirical software called IIT-PAVE is used to analyze the pavement responses and based on the multi-layer theory. IITPAVE can perform structural analysis of flexible pavements using the linear elastic analysis. It can calculate pavement deflections, strains, and stresses under given loading and environmental conditions. This is a multilayer analysis programmer used for design, analysis of the flexible or bitumen pavement using IRC: 37-2018 guidelines [127]. In this software we enter thickness of pavement layers, loads applied over the surface of pavement, tire pressure, spacing between the wheels and Poisson´s ratio as inputs. After running the software actual horizontal tensile strain and vertical compressive strains at critical locations of the pavement are obtained as output.

7.4 Stone block in a granular layer

The flexible pavement is supposed to have a uniform shear transmission of stress; this shear transmission of stress does not exist in granular layer with the presence of weak rock. The ability to resist load has a limit due to shear transmission of load is not taken place properly.

If the rock layer underneath the pavement is weak or highly compressible, it leads to excessive deformation and rutting of the pavement surface. This reduces the structural integrity of the pavement over time, leading to premature failure and may not provide adequate support to the overlying asphalt layers, leading to uneven settling and poor drainage. Weak or soft rock creates stress concentrations that can lead to premature failure of the pavement.

The larger size has the ability to resist more loads (traffic load) due to individual thickness of aggregate that imparts more strength. The disintegration of rock caused by repeated impact loads, and abrasive force, has changed the gradation. Rock gradation as per design is lost shortly after the application of load on the pavement due to load and moisture induced aggregate degradation. In this situation, the larger particle size of the rock will slow down the disintegration process, especially for brittle and soft rock. The early degradation of aggregate is mostly caused by environmental factors such as moisture and temperature variations, in addition to wheel load. The rate at which aggregate degradation occurs is influenced by size. Abrasive and weathering effect on aggregate is slowed down by larger particles because they degrade away more slowly in the same volume of rock compare to smaller rock as discussed in Chapter 5.

It is obvious that the condition of elastic layer model adopted in flexible pavement i.e, shear transmission of stress at the adjacent supporting element does not properly occur at the granular layer mainly due to brittle response of weak laminated sedimentary rock under the influence of combined action of moisture and load. The pavement prematurely fails due to loss of support at the granular layer. To counteract this situation, the stone block (30 cm x 20 cm x 15 cm) may be introduced at the granular layer to withstand the rock degradation as well as influence of water as shown in the figure 7.3. The loss of support due to the disintegration of rock creates less permeability that blocks drainage and develops rutting, crack within a short time in a pavement structure.

7.5 Effect of material properties on pavement structure response

The stress analysis IITPAVE software is used to compute the stresses and strains at critical location of the pavement. Horizontal tensile strain at the bottom of the bitumen layer and vertical compressive strain on the top of the subgrade are considered as the critical parameters to limit cracking and rutting in bituminous and non-bituminous layers.

The response of flexible pavement and stone block pavement structure in term of deflection, stress and strains using IITPAVE software are studied with different input parameters. The input for flexible pavement are CBR 5%, traffic load 5 msa, poisson ratio 0.35, tyre pressure 0.56 MPa, dual wheel load 20 kN, thickness of pavement layer 495 mm and elastic modulus of 2000 MPa, 148.22 MPa, 50 MPa respectively. Again, the input parameters for stone block pavement are CBR 5%, traffic load 5 msa, poisson ratio 0.35, tyre pressure 0.56 MPa, dual wheel load 20 kN, elastic modulus of 2000 MPa, 1500 MPa, 148.22 MPa, and 50 MPa for bituminous layer, stone block, granular layer and subgrade respectively [127]. The results indicate that stone block pavement response better in term of deflection, stress and strain.

Vertical compressive strain at the top of the subgrade and horizontal tensile strain at the bottom of the bituminous layer are the two critical locations considered in the elastic layer method of analysis of pavement structure adopted by the Indian standard code of practice. The allowable vertical compressive strain at the top of the subgrade due to the present traffic calculated as per IRC: 37-2018 is 0.000784. Again, the allowable horizontal tensile strain at the bottom of the bituminous layer is 0.000501. In both the cases, the actual strains are less than the allowable strains as shown in the table 7.2.

The analysis result therefore indicates that the pavement layer is still sufficient to take the present traffic load without significant structural failure. Moreover, the pavement fails at many locations. It is obvious that the pavement failure is not only due to the wheel load, the material behavior under the influence of water and wheel load combination is the main reason of failure in this study area as discussed in Chapter 4 and 5.

Table 7.2 Comparison between Stone Block and Conventional Flexible Pavement

Type of Pavement	No of Layers	Thickness (in mm)	E values (N/mm^2)	Deflection (mm)	Vertical Compressive Strain at the top of subgrade	Horizontal Tensile Strain at the bottom of wearing course
Stone Block Flexible Pavement	Bituminous Layer	30	2000	0.418	3.21E-04	6.05E-05
	Stone Block	220	1500	0.416		
	Granular Layer	150	148.22	0.381		
	Subgrade	Infinite	50	0.351		
Conventional Flexible Pavement	Bituminous Layer	95	2000	0.705	6.45E-04	3.89E-04
	Granular Layer	400	148.22	0.693		
	Subgrade	Infinite	50	0.486		

Furthermore, the effect of elastic layer of pavement structure (including bituminous layer, granular layer, subgrade) response on different elastic modulus of granular layer has been examined by putting input parameters such as CBR 7%, traffic load 5 msa, poisson ratio 0.35, tyre pressure 0.56 MPa, dual wheel load 20kN are made constant while changing the elastic modulus of base layer from 150 to 300 MPa in IITPAVE.

The analysis result shows that the vertical compressive strain at the top of the granular layer is almost the same with top pavement layer. Increasing the elastic modulus at granular layer reduces the deflection, horizontal tensile strain, and vertical compressive strain, as shown in Table 7.3.

This indicates that the material property at the top of the granular must be strong enough to take up the stress induced by the traffic load under the influence of moisture and wheel load. Even though the elastic modulus of one particular rock remains constant regardless of its size, the larger rock can carry more loads because of its larger surface area. In the absence of a stronger material (higher elastic modulus), the use of a larger rock (stone block) will reduce deflection, compressive strain, and tensile strain.

Table 7.3 Pavement structure response with respect to different elastic modulus using IITPAVE

Sl No	No of Layers	Thickness (mm)	E values (N/mm^2)	Deflection (mm)	Vertical Compressive Strain	Horizontal Tensile Strain
1	Bituminous Layer	80	2000	0.679	2.14E-04	3.91E-04
	Granular Layer	400	150	0.672	-1.33E-03	2.58E-04
	Subgrade	∞	62	0.432	-6.22E-04	
2	Bituminous Layer	80	2000	0.627	2.11E-04	3.79E-04
	Granular Layer	400	183	0.618	-1.17E-03	2.44E-04
	Subgrade	∞	62	0.415	-5.79E-04	
3	Bituminous Layer	80	2000	0.606	1.97E-04	3.57E-04
	Granular Layer	400	200	0.596	-1.11E-03	2.35E-04
	Subgrade	∞	62	0.407	-5.65E-04	
4	Bituminous Layer	80	2000	0.555	1.62E-04	3.06E-04
	Granular Layer	400	250	0.546	-9.56E-04	2.16E-04
	Subgrade	∞	62	0.388	-5.19E-04	
5	Bituminous Layer	80	2000	0.5176	1.35E-04	2.67E-04
	Granular Layer	400	300	0.5082	-8.43E-04	2.00E-04
	Subgrade	∞	62	0.3723	-4.81E-04	

The crushed laminated sedimentary rocks at the granular layers in Mizoram fail under the action of stress and weathering due to the high susceptibility and degradability of rock under the wheel load with the influence of water.

In order to counter the combination of wheel load and water induced damage on the pavement, the stone block and addition of plastic in bituminous mix is introduced as discussed in Chapter 6.

The pavement deflection reduces from top to bottom as well as horizontal tensile strain. Moreover, the vertical compressive strain is more at the top of granular layer compare to the top layer of bituminous and subgrade as shown in the table 7.3.

7.6 Proposed Pavement Construction Method

The premature failure of granular layer as well as bituminous layer is mostly caused by high water table, environmental factors such as moisture and temperature variations, in addition to wheel load. The pavement models which have better drainage, pavement layer ability to resist the load, less degradability of rock under the influence of water are presented here.

The suggested pavement component layers have similarities with the conventional pavement component layer except the vertical aggregate drain and adoption of stone block below the wearing course. The bedding sand is also introduced below the stone block layer to distribute the wheel load uniformly. In case of high water table, geo-membrane layer is provided above the subgrade layer.

Table 7.4 Stone Pavement Component Layer

Low Traffic	**Heavy Traffic**
Thin Wearing Course (Surface Dressing/PMC/MSS)	Thick Wearing Course (BC/SDBC/SMA)
Crack Resisting layer	Crack Resisting layer
Stone Block (30x20x20) cm	Stone Block (30x20x20) cm
Bedding Sand (20 mm)	Bedding Sand (20 mm)
Granular Sub-Base	Base Course (WMM)
Geo-membrane	Granular Sub-Base
Subgrade	Geo-membrane
	Subgrade

7.6.1 Pavement construction method for weak rock, high water table and seepage from the hillside

If the rock layer underneath the pavement is weak or highly compressible, it leads to excessive deformation and rutting of the pavement surface. This reduces the structural integrity of the pavement over time, leading to premature failure and may not provide adequate support to the overlying asphalt layers, leading to uneven settling and poor drainage. Weak or soft rock creates stress concentrations that can lead to premature failure of the pavement.

In high moisture content of the pavement structure, weak aggregate (thinly laminated rock) cannot withstand repeated traffic loads. Most of the laminated sedimentary rocks have a high water absorption capacity and low weathering resistance mainly due to the presence of clay and silt in the lamination layer. The main cause of failure on weak aggregate pavement is a degradation of rock caused by the presence of water, which saturates the rock and causes it to crumble when wheel load is applied as well as the presence of clay and silt in the small scale sequence of fine rock layer (lamination). Larger aggregate size (stone block) can be introduced into the granular layer of the pavement to counteract traffic load and moisture -induced rock degradation as bigger rock size has more thickness to resist load and equivalent depth of water penetration regardless of the size.

The larger size of aggregate has been found to have higher toughness (impact), strength (crushing), abrasion and weathering resistance than the smaller size of aggregate. Larger aggregate size has higher capability of absorbing more repetitive impact load, higher strength due to individual thickness of aggregate and less water absorption due to less surface area compared to smaller size over the same volume of rock as discussed in Chapter 4 and 5.

The rate at which aggregate degradation occurs is influenced by size. Abrasive and weathering effect on aggregate is slowed down by larger particles because they degrade away more slowly in the same volume of rock compare to smaller rock.

The pavement model has been developed for highly degradable rock with the presence of combined action of water seepage and traffic load. The pavement construction techniques for weak rock, water seepage, and high rainfall area are shown in figure 7.3 and 7.4.

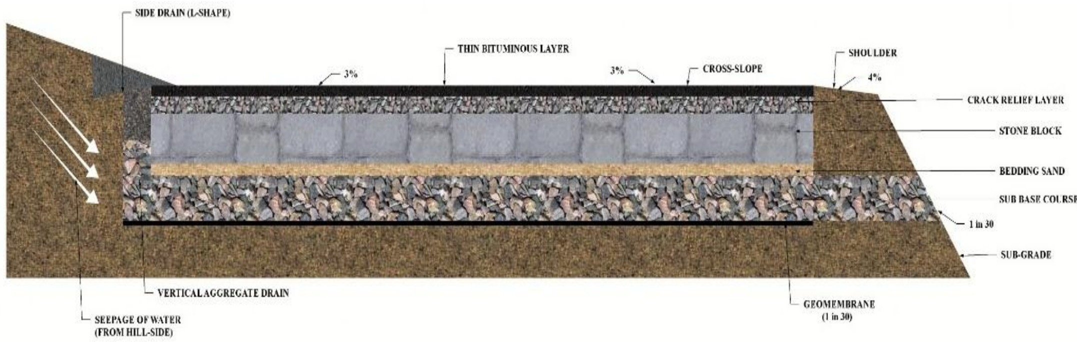

Fig 7.3 Typical cross section of low traffic volume road for weak rock, high water table and seepage from the hillside

Fig 7.4 Typical cross section of heavy traffic volume road for weak rock, high water table and seepage from the hillside

7.6.2 Pavement construction method for moisture induced failure at the subgrade soil

Road cutting at a differential level on sloping terrain results in non-uniform soil strength across the transverse section. The hillside of the road has a higher chance of subgrade degradation despite its higher soil density owing to the higher possibility of dampness and water intrusion from the hillside as discussed in chapter 3.

The hillside section of the pavement becomes damage quickly due to water ingress which saturated and softens the subgrade soil. One of the major causes of premature failures is due to the emanation of groundwater in the form of spring along the hilly side of the road that causes debonding the soil strength as discussed in Chapter 4.

It is possible that the build-up of moisture in the subgrade would ultimately lead to its degradation under vehicular load and this was evident from the localized appearance of rutting and depressions at several locations. In a few cases, substantial accumulation of water had led to the puncturing through of the sub-base layer and complete failure of the subgrade.

Replacement of the subgrade soil with granular material was found to be ineffective as the new material would get saturated in a short period of time at the subgrade layer where water saturation is high in monsoon season. The technique of applying a layer of stone soling was determined to be the most effective in terms of both cost and time as shown in the figure 7.5 and 7.6.

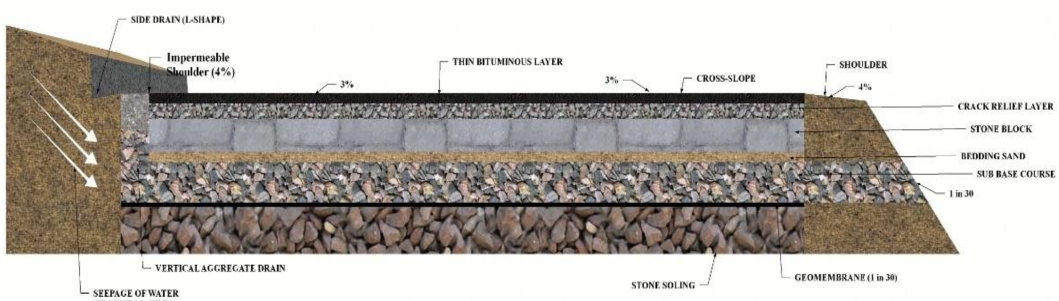

Fig 7.5 Typical cross section of Low Traffic road for moisture induced failure at the subgrade soil

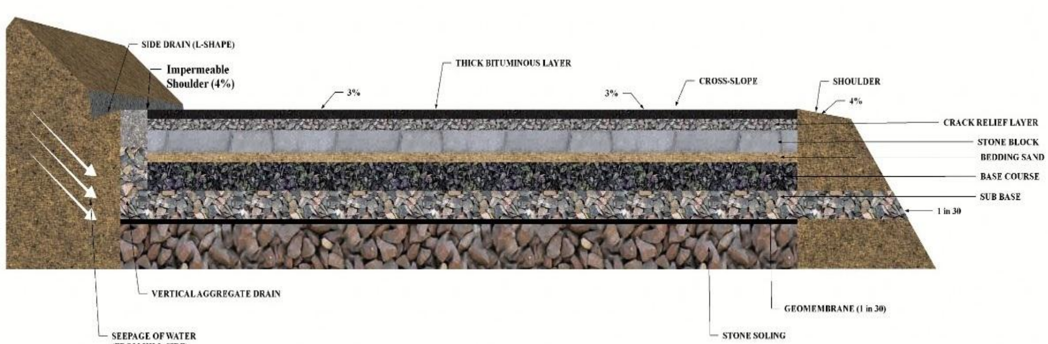

Fig 7.6 Typical cross section of Heavy Traffic road for moisture induced failure at the subgrade soil

7.7 Summary

When a vehicle travels over a flexible pavement, the wheels exert a load on the surface, causing the pavement to deform. The deformation is distributed throughout the layers of the pavement, including the base, sub-base, and subgrade layers. The rocks in the base and sub-base layers provide stability and load-bearing capacity to the pavement. The types of response exhibited by the rocks under wheel loads affect distribution of stresses and strains throughout the pavement layers and durability.

Rock gradation as per code specification is lost shortly after the application of load on the pavement due to load and moisture induced aggregate degradation. Rock aggregate at the granular layer breaks before traffic load is transferred to the adjacent support rock that result in higher stresses, localized stress concentrations, increased rutting, and reduced drainage.

Strength and stiffness of the rock layer can be improved by providing the stone block as alternative to crushed stone. The materials used especially in granular layer rock are not good enough in Mizoram that need to be addressed by changing the size of rock used. The stone block is having less degradation due to impact, abrasion, crushing and weathering mainly due to the size and its massiveness (stiffness) is the suggested solution.

To mitigate the undesirable effects of brittle rock response due to wheel load and water induced damage in a pavement, the vertical aggregate drain and drainage layer (GSB) is introduced as well as the stone block with the bedding sand under the wearing course. The geo-membrane is provided above the subgrade layer to prevent the water entry from the subgrade. In case of weak subgrade soil, stone soling at the foundation level is suggested.

CHAPTER 8

CONCLUSIONS

8.1 Conclusions

A detailed study on the premature failure of flexible pavement along different stretches in state highway-I have been carried out and presented in this work. Seven different stretches in state highway-I have been selected for this purpose, associated with the problem of premature pavement failure. The various causes of premature failures of flexible pavement manifesting in different forms such as cracks, potholes, quick disintegration of rock, subgrade failure and rutting were investigated to ascertain the exact reason for such failures. A comprehensive approach involving both field and laboratory based investigation methods were employed to understand and substantiate the observations made in different stretches along the highway. Representative soil samples were collected from different stretches and tested in laboratory to find out various engineering parameters of soil related to pavement subgrade. The amount and direction of groundwater flow are determined by geological conditions. In order to study to the geological features of the affected areas, a geological structures assessment survey was conducted such as measurement of dip amount, dip directions/orientations, strike, folds and bedding of rock. To gain further insight into the premature failure problems, electrical resistivity survey was also employed to identify possible failure reasons pertaining to groundwater and its movement; and in general soil structure. In addition to this field and laboratory based investigation a method, subsequent numerical validation of the work was also conducted using a pavement analysis program. In the light of the various information gathered from field test, laboratory test and analysis results, most likely reasons for premature failure were found to be the presence of water in a pavement structure resulting in softening of the subgrade soil, rock degradation and detachment of the bonding between aggregate and bitumen. Further, most feasible mitigation measures to overcome such shortcomings are proposed and are presented methodically.

Following important conclusions were derived from the present study:

1. It is found that pavement failure occurs in multiple stretches mainly due to oversaturation of soil caused by poor drainage. For instance, pavement stretches in Hualngo and Pukpui have a layer of shale rock that prevents water from penetrating further downward resulting in oversaturation of subgrade soil and consequent softening of subgrade soil. This softening of soil may be responsible to the disintegration of overlying pavement structure. Based on the engineering properties of the soil, it may be observed that subgrade soil by itself is not the primary reason for pavement failure; rather, failure happens when the soil becomes significantly saturated with water over an extended period of time.

2. Type of rock and geological features encountered in the alignment of the hill road section hugely influence the pavement performance. Wherein road is constructed on the excavated slope of a hilly terrain, road-cut sections expose different rock features with springs at various locations. In relation to the geological and hydrogeological features of the exposed rock as a result of excavation cut, following important observations were made:

    a) Road failure is associated with hydrogeological features such as the seepage of water from adjacent hills in cut sections especially during monsoon rainfall, which is recharged from precipitation, and groundwater emerges in springs. Seepage of water can be observed even on the pavement surface at Hualngo and Pukpui areas even during dry seasons. The presence of water weakens the pavement foundation as the subgrade soil tends to lose strength when fully saturated with water. Notably, above observation was complimented by resistivity survey results which indicate that soil/ rock beneath the pavements are water bearing strata.
    b) The road sections which are cut through soil and rock strata along its alignment and is susceptible to seepage flow in the form of a spring needs

geological assessment such as dip angle, dip direction and bedding of exposed rock. The dip direction of the rock/soil bedding plane toward the pavement (dip slope), against the pavement (anti-dip slope), parallel with the pavement significantly affects the movement of groundwater. Whereas roads cut through anti-dip slope are most favorable; in contrast, the flow of water through fracture and rock bedding contributes to the failure of the pavement structure when the bedding is dip slope as road cut follows the dip direction. Nonetheless, higher the dip angle; better is the drainage capacity for roads cut through dip slope. For instance, the good section at Mel 5 has a dip direction against the slope/road with an angle of 41^0. On the other hand, Pukpui stretch has a bedding plane that dips at an angle of 11^0 parallel to the direction of the road with the same road gradient. The Pukpui area premature failure is caused by the geological structure that allows water to flow along the road gradient and impervious shale rocks that block further water infiltration.

3. The larger size of aggregate has been found to have higher toughness (impact), strength (crushing), abrasion and weathering resistance than the smaller size of aggregate. Larger aggregate size has a higher capability of absorbing more repetitive impact load, higher strength due to individual thickness of aggregate and less water absorption due to reduced surface area compared to smaller size over the same volume of rock. Abrasive and weathering effect on aggregate is slowed down by larger particles because they degrade away more slowly in the same volume of rock compare to smaller rock. This indicates that rocks with larger particle sizes have a greater ability to withstand degradations caused by abrasion, repeated impact, crushing, and the presence of moisture.

4. Flexible pavement can deteriorate rapidly due to the presence of thinly laminate sedimentary aggregates (rock) which are susceptible to water. The lamination

layer, where clay and silt are present, is the weakest point in the rock structure. The laminate layer can be penetrated by water and weaken the bond between the rock.

5. It has been demonstrated that modified bitumen with plastic increase the asphalt performance and thus can be used as an effective measure to prevent premature pavement failure. The plastic acts as a binder that firmly bonds the particles together and seal the lamination layer of aggregate, increasing the coarse aggregate toughness (impact), hardness, abrasion resistance and water absorption.
6. Plastic added in excess of 9% by weight in the aggregate and 11% in the bituminous mix didn't result in any significant improvement in aggregate performance. Plastic as a sole binder is not suggested as the mix does not meet the desirable requirement and workability criterion. The partial replacement of bitumen with plastic in BC-II also increases the performance of the bituminous concrete. A better material performance has been achieved by using plastic wastes as a coating agent on aggregate surfaces and as a binder in bituminous concrete.
7. Desirable retained tensile strength of BC was achieved by the addition of 1.1 % plastic by weight of the sample into the bituminous mix. When plastic was added, the retained tensile strength values in 13 mm and 19 mm NMAS of BC increased to 82.81% and 92.60%, respectively, from 52.59% and 58.58% in the case of without plastic. The reduction of tensile strength after bituminous concrete sample undergoes weathering action is mainly due to high water absorption of aggregate as well as the presence of small sequence of rock layer.
8. The use of larger aggregate sizes and the addition of plastic in the bituminous concrete, which increases the retained tensile strength of bituminous concrete, can improve the performance of rock aggregate under weathering action and wheel load.
9. In order to prolong the life of the pavement, larger size of boulder (stone block) below the wearing course for the reduction of physical rock degradation and

coating of aggregate with plastic or addition of plastic in bituminous mix has been introduced. Besides, vertical drainage and transverse drainage in the pavement structure as suggested in pavement model may be adopted to drain out the water quickly from the hillside water seepage. Strength and stiffness of the rock layer can be improved by providing the stone block as alternative to crushed stone.

10. The amount of water ingress to the pavement structure can be reduced by providing good surface drainage such as precise cross slope, shoulder. Shoulders should not confine surface water. The high grass should be cut regularly, while elevated shoulders should be cleared.

11. The subsurface water or groundwater is contributed by the emanation of spring from the hill slope, capillary rise, high water table and ingress of water from top pavement surface and shoulder. Adoption of drainage layer at the GSB (granular sub-base), introducing the sand layer, and installing a French drain is suggested to prevent subsurface water-induced premature failure.

8.2 Scope for Further Studies

Further improvement on the strengthening of the laminate sedimentary soft rock may be explored so that the pavement structure could sustain the load during the design life. An experimental performance assessment of the stone block pavement maybe carried out in relation to its actual load-bearing capacity, drainage capabilities, and rate of rock disintegration in the presence of water.

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LIST OF PUBLICATIONS

Journal Publications

1. **Laldintluanga, H**., Ramhmachhuani, R., Mozumder, R.A. F Lalbiakmawia. Hydrogeological Effects on Premature Failure of Flexible Pavement in Hilly Area Along State Highway-I in Mizoram, India. Indian Geotech J 53, 29–41 (2023). https://doi.org/10.1007/s40098-022-00651-x. [Scopus, WoS]

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1. **H Laldintluanga**, Rebecca Ramhmachhuani, " The Effect of Coarse Aggregate Sizes on Their Physical Properties," National Conference on Emerging Trends in Engineering Science & Technology, Hooghly, India. *Transactions on Engineering Science and Technology,* March, 2022, pp.7-10. **(Paper presented)**

2. **H Laldintluanga**, Rebecca Ramhmachhuani, "A Study on the Performance of Plastic Coating on Weak Aggregate in a Bituminous Mix," International Conference on Advances in Energy, Environment for Sustainable Development (AEESD-2022) National Institute of Technology Meghalaya, India. January, 2022. (**Paper presented**)

Patent

1. A System for Assessing and Mitigating Pavement Failure Risks along Highways (Ein System zur Bewertung und Minderung des Risikos von Fahrbahnausfällen entlang von Autobahnen Gebrauchsmusters) Nr. 202023106711 Patented by Hmar, Laldintluanga, Aizawl, Mizoram

PARTICULARS OF THE CANDIDATE

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ABSTRACT

EVALUATION OF PREMATURE FAILURE OF FLEXIBLE PAVEMENT IN HILLY AREA OF MIZORAM

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EVALUATION OF PREMATURE FAILURE OF FLEXIBLE PAVEMENT IN HILLY AREA OF MIZORAM

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IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY IN CIVIL ENGINEERING DEPARTMENT,

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A detailed study on the premature failure of flexible pavement along different stretches in state highway-I which connects the lifeline between North and South of Mizoram, India have been carried out and presented in this work. The state highway has experienced premature failure at multiple locations. Seven most critically affected areas having failure span length ranging from 85 m to 200 m were chosen for the study. Underlying causes of premature failures of flexible pavement manifesting in different forms such as cracks, potholes, quick disintegration of rock, subgrade failure and rutting were investigated to ascertain the exact reason for such failures. A comprehensive approach involving both field and laboratory based investigation methods were employed to understand and substantiate the observations made in different stretches along the highway. Representative soil samples were collected from different stretches and tested in laboratory to find out various engineering parameters of soil related to pavement subgrade. The amount and direction of groundwater flow are determined by geological conditions. In order to study to the geological features of the affected areas, a geological structures assessment survey was conducted such as measurement of dip amount, dip directions/orientations, strike, folds and bedding of rock. To gain further insight into the premature failure problems, electrical resistivity survey was also employed to identify possible failure reasons pertaining to groundwater and its movement; and in general soil structure. In addition to this field and laboratory based investigation method, subsequent numerical validation of the work was also conducted using a pavement analysis program. In the light of the various information gathered from field test, laboratory test and analysis results, most likely reasons for premature failure were found to be the oversaturation of pavement structure resulting in softening of the subgrade soil, rock degradation and detachment of the bonding between aggregate and bitumen.

Premature failure of flexible pavement refers to the deterioration or breakdown of the pavement surface or structural layers earlier than expected. The loss of strength and stiffness of the subgrade soil, which is normally supposed to facilitate the load transfer mechanism from the overlying layers of flexible pavement, is one of the primary causes for the premature failure of flexible pavement. Weakening of the subgrade soil in any form during the service life of pavement structure may seriously undermine the stability and performance of flexible pavement. Problems related to stability and performance of flexible pavements may manifest in the form of crack, undulation and rutting etc. Similar observations were made at the locations suffering from premature failures; therefore, strength and stiffness assessment of the existing subgrade soil was conducted as a first line of investigation. Consequently, soil samples were obtained from subgrade level at affected locations and laboratory tests were conducted to find out properties pertaining to strength, stiffness, permeability, compaction and plasticity behaviour of soil.

During field inspection it was observed that pavement sections near the hilly side at some of the affected stretches were damaged by the emanation of seepage water in the form of spring from the hill slope of the road. Groundwater flow plays a significant role in pavement performance. Weakening of subgrade, sub-base and base by groundwater movement and consequent saturation of subgrade soil is one of the important factors leading to pavement failure. Seepage of water can be observed even on the pavement surface at some stretches even during dry seasons. The presence of water weakens the pavement foundation as the subgrade soil tends to lose strength and stiffness when fully saturated with water. In relation to this, impact of hydrogeological factors on pavement performance has been also studied in the present work. In order to facilitate the hydrogeological study, electrical resistivity method using Vertical Electrical Sounding (VES) technique was employed to study groundwater condition and lithology at selected points. A careful evaluation of geophysical and geotechnical test results revealed that geological features are also found to be one of the most crucial factors affecting the pavement performance. The road sections cut through soil and rock strata along its alignment are susceptible to seepage flow in the form of a spring and were evaluated for their geological structure. The dip direction of the rock/soil bedding plane toward the pavement (dip slope), against the pavement (anti-dip slope), parallel with the pavement significantly affects the movement of groundwater. It is observed that anti-dip slope rocks are most favorable from pavement performance criterion as it tends to take away the seepage water from pavement in contrast to the rock having dip slope. Moreover, for roads cut through dip slope, rocks higher the dip angle; better is the drainage capacity. For instance, the good section at Mel 5 has a dip direction against the slope/road with an angle of 41^0. On the other hand, Pukpui stretch has a bedding plane that dips at an angle of 11^0 parallel to the direction of the road with the same road gradient. The Pukpui area premature failure is caused by the geological structure that allows water to flow along the road gradient and impervious shale rocks that block further water infiltration.

A careful examination of aggregate samples collected from different stretches showed significant sign of early deterioration suggesting it to be a probable factor leading to premature failure of pavement. Degradation of aggregate is the disintegration of aggregate due to the wheel load and moisture saturation that cause loss of support in unbound granular layer and reduction of drainage in pavement materials. Sedimentary rocks from different quarries were evaluated to determine the influence of aggregate size on the degradation of materials. The size-dependent performance of rock with regard to physical properties in flexible pavement is studied. In order to determine the effect of rock aggregate size on their rock degradation properties, different sieve ranges were selected. The sieve sizes are graded into four categories according to Indian Standard sieve: R1 (20 mm-16 mm), R2 (16 mm-12.5 mm), R3 (12.5 mm-10 mm), and R4 (10 mm-4.75 mm) for degradation test such as, aggregate impact value (AIV), aggregate crushing value (ACV), water absorption (WA) and weathering test such as soundness and slake durability. It is observed that the larger size of aggregate has exhibited better resistance against impact load, abrasion, weathering action and water absorption compare to smaller size over the same volume of rock. The rate at which aggregate degradation occurs is influenced by rock size. Along with the wheel load, environmental factors including moisture and temperature changes are the primary causes of the early degradation of aggregate. The river rock has a better resistance against degradation and weathering action as it has lesser lamination (a small scale sequence of fine rock layer). Adoption of larger size aggregate in the pavement granular layer is suggested to withstand the effect of environmental action and wheel load, especially on the friable and quickly disintegrated rock. A rating index and value have been developed for aggregate degradation.

As a counter measure to prevent early disintegration of aggregate, an attempt has been made in the present study to evaluate the efficacy of plastic wastes (low-density polyethylene) in order to strengthen the properties of laminated sedimentary aggregates (rock) and bituminous concrete. Effect of addition of plastic on different properties of laminated sedimentary aggregate (rock) and the bituminous concrete in pavement has been examined and subsequently, a comparative study was undertaken. Optimum plastic content is obtained for both aggregate alone and bituminous concrete by varying the percentage of plastic wastes. It is observed that plastic as a sole binder is not effective as the mix does not meet the desirable requirement and workability criterion. Rather plastic acts as a binder that firmly bonds the particles together and seal the lamination layer of aggregate, increasing the coarse aggregate toughness (impact), hardness, abrasion resistance, and water absorption. Notably, addition of plastic in conjunction with larger size aggregate resulted in significant improvement of retained tensile strength of bituminous concrete. Laminate sedimentary rock aggregate used in asphalt concrete could not achieve the minimum retained tensile strength value mainly due to the low resistivity of weathering action. A larger size of rock gradation has achieved higher retained tensile strength compare to smaller size of rock gradation, mainly due to the less degradation that develops in a bituminous structure. Therefore, use of larger aggregate sizes along with addition of plastic in the bituminous mix is found to increases the retained tensile strength of bituminous concrete to meet the minimum specified criterion and thus can be used to improve the performance of rock aggregate under moisture and wheel load.

The deflection caused by the applied load in the flexible pavement is responded back by the underlying supporting layers. The response-value is the degree to which the layer will re-act the vertical stresses. In real pavement structure, the displacement distribution is not continuous, neither it is fully discontinuous. The response of the materials in terms of strain, stress and deflection induced by wheel load has been calculated in the present study using a pavement analysis. Accordingly, a pavement construction method was proposed to make use of the highly degradable rock in conditions characterized by heavy rainfall and water seepage. This study presents a pavement construction model with improved drainage, better pavement layer stress resistance, and less rock degradation in the presence of water and vehicle load.