PAVEMENT EVALUATION OF KOLLAM ASRAMAM LINK ROAD AND SUBGRADE STABILISATION USING STONE COLUMN: A CASE STUDY

A PROJECT REPORT

submitted by

ASWINI A S

Reg. No: TKM21CETE02

to

the APJ Abdul Kalam Technological University

in partial fulfilment of the requirements for the award of degree

of

Master of Technology

In

Transportation Engineering

Department of Civil Engineering

T K M College of Engineering, Kollam

MAY 2023

DECLARATION

I undersigned hereby declare that the project report "Pavement Evaluation of Kollam Asramam Link Road and Subgrade Stabilisation using Stone Column: A Case Study", submitted for partial fulfillment of the requirements for the award of degree of Master of Technology of the APJ Abdul Kalam Technological University, Kerala is a bonafide work done by me under supervision of Dr. Amal Azad Sahib and Prof. Sai Niveditha M G. This submission represents my idea in my own words and where ideas or words of others have been included, I have adequately and accurately cited and referenced the original sources. I also declare that I have adhered to ethics of academic honesty and integrity and have not misrepresented or fabricated any data or idea or fact or source in my submission. I understand that any violation of the above will be a cause for disciplinary action by the institute and/or the University and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been obtained. This report has not been previously formed the basis for the award of any degree, diploma or similar title of any other University.

Kollam   ASWINI A S

10-05-2023

DEPARTMENT OF CIVIL ENGINEERING

T K M College of Engineering, Kollam

CERTIFICATE

This is to certify that the report entitled **'PAVEMENT EVALUATION OF KOLLAM ASRAMAM LINK ROAD AND SUBGRADE STABILISATION USING STONE COLUMN: A CASE STUDY'** submitted by **ASWINI A S, REG NO.: TKM21CETE02** to the APJ Abdul Kalam Technological University in partial fulfillment of the requirements for the award of the Degree of Master of Technology in Transportation Engineering, Civil Engineering is a bonafide record of the project work carried out by her under our guidance and supervision. This report in any form has not been submitted to any other University or Institute for any purpose.

Guide	Co-Guide
Prof. Sai Niveditha M G Assistant Professor Dept. of Civil Engineering TKMCE, Kollam	Dr. Amal Azad Sahib Associate Professor Dept. of Civil Engineering TKMCE, Kollam

Project Coordinator	Head of Department
Dr. Adarsh S Professor Dept. of Civil Engineering TKMCE, Kollam	Dr. Sajeeb R Professor Dept. of Civil Engineering TKMCE, Kollam

ACKNOWLEDGEMENT

I would like to take this opportunity to express my deep sense of gratitude and sincere thanks to all, without whom this project could never have been completed this well.

First and foremost, I sincerely thank the almighty who is most beneficent and merciful for giving knowledge and courage to complete this project successfully.

I express my heartfelt gratefulness to my guide **Prof. Sai Niveditha M G**, Assistant Professor, Department of Civil Engineering, T K M College of Engineering, Kollam for her valuable guidance, suggestions, intense support and constructive criticism.

I sincerely thank my co-guide **Dr. Amal Azad Sahib**, Associate Professor, Department of Civil engineering, T K M College of Engineering, Kollam for her intense support, guidance, suggestions and constructive criticism.

I am greatly thankful to my project coordinator **Dr. Adarsh S**, Professor, Department of Civil Engineering, T K M College of Engineering, Kollam for his constant supervision as well as for providing necessary information regarding project.

It is my privilege to express my gratitude to **Dr. Sajeeb R**, Professor and Head, Department of Civil Engineering, for encouraging and permitting me to present this project for the partial fulfilment of the requirements leading to the award of M-Tech degree.

Finally, I thank my ever-loving parents for giving unconditional support, encouragement and inexhaustible source of inspiration. Last but not the least I extent my love to all my friends for their priceless love, support and help throughout the project.

ASWINI A S

ABSTRACT

Many roads in India faces accelerated deterioration due to lack of studies conducted prior to the construction. This paper attempts to evaluate a stretch of asphalt pavement on its structural performance. This evaluation gives an idea of the vital distresses happening on the pavement and gives a general review of the pivotal factors influencing the same. Flexible pavements undergo functional deterioration as well as structural deterioration simultaneously due to the combined effects of climate, environment and traffic loads. The functional deterioration is indicated by the changes in surface condition of the pavement in the form of deterioration in the riding quality, which can be measured by simple methods. The rate of structural deterioration of flexible pavements depends on several factors such as, the stability of the existing pavement structure and the component layers, magnitude and repetition of traffic wheel loads, growth rate of traffic loads, effective functioning of pavement drainage system and severity of the climatic and environmental factors. In this study, pavement condition rating is provided on the given pavement section as per IRC:82-2015 based on the quantity of distresses present on the sections. The structural evaluation of flexible pavement is carried out using Benkelman Beam Deflectometer. The characteristic deflection and corresponding overlay thickness needed is determined from the result of BBD. From the results of the SPT done on the soil, the need for Stone column to improve the drainage and settlement is derived. Stone column for the given section is designed as per Priebe's method and stone columns of diameters 0.5, 0.8 and 1m are adopted for study. PLAXIS 2D software is used for determining the effect of stone column on subgrade soil. Both settlement and consolidation effect are studied using PLAXIS 2D.

Keywords: Pavement Condition Rating, Structural Evaluation, Benkelman Beam Deflectometer, PLAXIS 2D

CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

BBD	Benkelman Beam Deflectometer
CBR	California Bearing Ratio
FWD	Falling Weight Deflectometer
IRC	Indian Road Congress
MERLIN	Machine for Evaluating Roughness using Low-cost Instrumentation
PCR	Pavement Condition Rating
PLAXIS	Plane strain and Axial symmetry
SPT	Standard Penetration Test

CHAPTER 1

INTRODUCTION

1.1 GENERAL

In India, the blame of pavement deterioration is mostly put on the adverse climatic conditions and erratic soil characteristic existing at a particular area. Even though the reasons of pavement distresses may be heavy traffic condition, climatic condition, drainage efficiency, temperature and inconsistent subgrade performance round the year, the actual issue of this situation is found to be improper studies carried out before the construction of roadway. It is a fact that pavement's condition begins to deteriorate from the first day following completion, but the rate at which the decay occurs is a direct reflection of insufficient studies conducted and improper planning during execution of work. The lack of timely maintenance works carried out also accelerate the process of decay. Flexible pavement deteriorates more rapidly than rigid pavement and it continues to deteriorate even in idle condition of traffic due to combined action of climatic and environmental factors. The deterioration in roadway can be scrutinized under two heads: functional and structural. Even if one influences the other, both are analyzed separately for ease in study. Road pavement performances are measured by conducting functional and structural evaluations. Functional evaluations aim at finding the utility provided to the user. This includes riding quality, surface roughness and skid resistance. Structural evaluations concentrate on the responses of pavement under the application of vehicle loads. This includes deformations/deflections of the pavement due to traffic load. All these evaluations help in identification of pavement distresses and helps in formulating appropriate remedial measures. Hence it is important to carry out proper pavement evaluation at right time and using right methods.

Evaluation of pavement is a tedious process and a number of features has to be assessed regarding a road section for in depth analysis of the pavement condition, remaining serviceability life, deterioration prediction and possible deterioration in near future. Most of the problems in pavement can be linked to weak subgrade. There are many methods to improve the properties of subgrade such as chemical stabilization, mechanical stabilization etc. One of the most commonly used method for subgrade stabilization is providing stone columns. Stone column not only improve the bearing capacity of soil but also improve the drainage properties and consolidation effects. Recently many Finite element methods are available for the analysis of pavement sections and related subgrade properties. One such FEM analysis method is PLAXIS 2D where subgrade properties are analyzed and is used widely in geotechnical projects.

The present study focuses on the Evaluation of Kollam-Asramam Link Road in Kollam district. The functional evaluation of the pavement section including, Pavement condition rating and Roughness determination has been done. In order to determine the structural capacity of the pavement section and deflection under traffic load the Benkelman Beam Deflectometer test is being carried out. Based on the result of BBD, overlay thickness needed for designed traffic condition is determined. Subgrade stabilization using Stone column is suggested in the present study and the numerical analysis is being carried out using PLAXIS 2D software. From the software, consolidation analysis and load-settlement behaviour with and without stone column is determined.

1.2 PROBLEM STATEMENT

The Link Road connects Chinnakada, Asramam and Taluk Kachery area in Kollam city and stretches for a length of 0.9 km. The development of the road started in 2006 and initially it was a mud road formed by dumping laterite soil. The entire length of the land overlooks the Ashtamudi lake and rests on marshy soil. The black topped asphalt pavement was completed in 2011. Since then, there has been a lot of maintenance activities done on the pavement. But still distresses continue to occur and a huge amount of money is being spend on the maintenance activities. There is noticeable amount of settlement in the pavement stretch and also on the side walk region as in Fig. 1.1. As an initial step functional evaluation of the pavement has been carried out and type and extent of distresses were identified. From the bore hole data obtained from the previous studies conducted on the given stretch it was identified that the SPT value of the subgrade soil is very low. Hence there is a need for stabilization of the subgrade to carry the design traffic load.

Fig. 1.1 Present condition of road section

1.3 OBJECTIVES OF THE STUDY

The objective of the proposed study is

1. To determine the Pavement Condition Rating of road section as per IRC:82-2015.
2. To determine the deflection of the pavement section using Benkelman Beam Deflectometer.
3. To model the pavement section using PLAXIS 2D and determine the Settlement Improvement Factor.
4. To design stone column to improve settlement of subgrade soil.
5. To analyze the effect of consolidation on subgrade using PLAXIS 2D.

1.4 ORGANIZATION OF THE REPORT

The report essentially consists of five chapters. Chapter 1 gives an introductory briefing about the whole topic. It consists of a general overview of the topic, its socioeconomic relevance, scope of the study and a description of how the report is organized. Chapter 2 is the literature review including its critical review and research gap. Chapter 3 consists of the description of the methodology adopted while chapter 4 includes the result obtained and following discussion. Chapter 5 has the conclusion arrived at the end of the project.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Pavement degradation typically results from a combination of traffic, changing weather, drainage, environmental concerns, etc. Due to the above issues, flexible pavements typically degrade more quickly than rigid pavement does. When water is kept in the void areas between the layers of bituminous pavement, the rate of deterioration of the pavement accelerates quickly. In highway sections, there are two kinds of deteriorations: functional deteriorations and structural deteriorations. Functional evaluation methods are used to determine the functional aspects of pavements such as serviceability, roughness, skid resistance etc. Structural evaluation methods are used to determine the structural capacity of pavement such as deflection, layer thickness and material properties.

2.2 FUNCTIONAL EVALUATION

Functional deterioration of pavement occurs due to varying factors such as traffic load, climatic factors, environmental factors, moisture content etc. especially on pavement surface. These pavement deteriorations are marked by undulations such as rutting, cracks, potholes. These affect the riding quality and vehicle operation cost of all automobiles (**Al-Arkawazi et al., 2017**).

Subramanyam et al. (2017) conducted functional and structural valuation on state highway 99. In this study the entire pavement is divided into 8 samples where 5 samples were considered for evaluation. Various distresses like rutting, potholes, raveling etc. were identified using manual distress studies. As per IRC:82-2015 guide line those pavements were classified as poor, good, fair. Each pavement section is rated based on the distress percentage and final rating value of entire pavement is computed by taking average.

Magdi et al. (2015) studied Obeid khatim road in Khartoum to identify various failures in the road section and propose suitable remedial measures for the same. The study consisted of two sections, one to identify the distresses using visual survey and second, to find the actual cause of the distress. The data obtained from field survey were analyzed. Each pavement distress was categorized into different levels such as heavy, moderate, low. Further each pavement distress was classified based on covered area percentage. From the study it was concluded that about 70% of distress in that pavement stretch is caused due to alligator cracking and rutting.

Untung et al. (2017) evaluated the structural and functional properties of south arterial road in Yogyakarta. Based on the study they provided recommendations for maintenance and rehabilitation of the road stretch. In functional evaluation process International Roughness Index and Pavement Condition Indexes were found and pavement was classified as good, damaged and seriously damaged. PCI value is calculated based on the visual studies and IRI value is obtained from the roughness measurement using MERLIN. In a similar kind of study **Tiza et al. (2016)** proposed that the major cause of most of the pavement distresses are due to poor drainage condition of the pavement or inability of the pavement to with stand heavy traffic due to poor design. She also added that in almost every case the pavement distresses occur as combinations and there is a necessity to separate each distress into their respective class and severity level. In a study conducted by **Alaamri et al. (2017)** in Ikzi road different pavement stresses such as pot holes, rutting, alligator cracks, block cracks were identified and measured. The possible causes of these distresses were categorized into 2 sections: during and after implementation process such as improper compaction of pavement layers, asphalt mixture arrival delay, Heat shrinkage after laying, stress and drainage effect.

2.3 STRUCTURAL EVALUATION METHODS

In case of flexible pavement, structural deterioration is marked by increase in the deflection of pavement under the applied traffic load or rutting on the surface of the pavement which indicate permanent deformation of the pavement including subgrade under the wheel load. Structural evaluation of pavement is done to identify the present structural condition of pavement, extent of deterioration, estimation of residual life, design of overlay thickness of the pavement. Structural evaluation can be done in two different ways: destructive method and Non-destructive method. In destructive method, the pavement layer thickness is evaluated using field core data in which a pavement section is removed using a core cutter.

2.3.1 Benkelman Beam Deflectometer

Benkelman beam is a less expensive deflection measuring tool, and the study method is straight and simple to execute. Guidelines for conducting "Benkelman Beam Deflection (BBD) studies for structural evaluation of flexible pavements and for the design of overlay thickness for strengthening the same have been published by the Indian Roads Congress (IRC). As a result, this method of evaluating the pavement and designing the overlay is typically used in this country.

Benkelman beams are used to calculate the amount of a flexible pavement's rebound deflection when a truck's normal wheel load is moved forward. A number of deflection observation locations are marked on the pavement surface of the chosen road segment. At each point, the rebound deflection values caused by the standard wheel load are recorded. These deflection values typically vary from point to point, and a statistical analysis of the deflection data gathered is used to establish the overall "characteristic deflection value" of the pavement length.

2.3.2 Falling Weight and Light Weight Deflectometer

In the Falling Weight Deflectometer (FWD) method, an impulse load is imparted to the pavement surface using a falling weight, and the deflected form of the pavement surface is recorded using velocity transducers or geophones set at various radial distances from the center of the falling weight. Different weights falling from various heights are dropped with varying velocities. The data on surface deflection is gathered and stored using velocity sensors. The elastic moduli of the various pavement layers of the current pavement are calculated using the "back calculation" approach using the values of pavement layer thickness, Poisson's ratio, etc. Utilizing the effective modulus of the present pavement, one may determine the pavement's structural health and estimate overlay thickness.

For quality control and assurance, as well as compaction control, the Light Weight Deflectometer (LWD), also known as Portable Falling Weight Deflectometer (PFWD), has grown in popularity in the earthwork and pavement construction industries. The amounts of the impulse loads are smaller in LWD as compared to FWD. The FWD gadget has been extensively used to calculate the stiffness of the pavement layer. Using the recorded surface deflections, a number of methods are available to determine the resilient modulus of the pavement subgrade.

2.3.3 Summary of Results

Even though FWD test is more costly, the deflection value obtained from FWD is more accurate than BBD test . As a result, the resilient modulus obtained by FWD is more accurate and lesser value than obtained from BBD. The pavement designed with FWD values are lesser and hence cost of construction will be less. Thus, FWD test is said to be more economical than BBD. The value obtained between BBD and FWD test are found to be in good correlation (nearly 0.89). FWD is more deterministic as it involves dynamic loading which stimulate moving traffic load.

The static moduli of subgrade estimated from BBD test is on a lower side as compared with the dynamic moduli of subgrade estimated from LWD. The composite moduli of subgrade values estimated from LWD test are approximately consistent with laboratory estimated moduli of subgrade using repeated triaxial test (**Guzzarlapudi et al., 2016 and Sanjay et al., 2022**).

2.4 FINITE ELEMENT ANALYSIS USING PLAXIS 2D

The finite element programme, PLAXIS 2D was created especially for the study of stability and deformation in geotechnical engineering applications. The increased output tools allow a thorough presentation of computational findings, and the easy graphical input processes enable the speedy development of complicated finite element models. The actual computation is totally automated and is based on reliable numerical techniques.

Maryam et al. (2018) studied both unit cell and plane strain concept for stone column analysis. In this work, a stone column was simulated as a unit cell and as a plane strain model using the finite element programme PLAXIS-2D-V8.2 in order to compare the performances of the two models. The diameter and c/c spacing of the stone columns, the friction angle of the stone column material, and the undrained cohesiveness of the soft soil were the main elements that were examined. Due to their importance in the design of stone columns, the settlement improvement factor and excess pore water pressure were the focus of this parametric study. The primary conclusions of this study were that the settlement improvement factor for the planar strain model ranged between 2.2 to 3.2, which indicates that the settlement was enhanced by a substantial amount, almost twice. The settlement improvement factor in the unit cell concept, however, was limited at 1.53. The settlement improvement's outcomes were contrasted with the usual theoretical explanations for studies into the behaviour of stone columns. In comparison to the plane strain model, the unit cell model displayed a lower peak value of excess pore water pressure.

2.4.1 Effect of geometry of stone column using PLAXIS 2D

Ghorbani et al. (2020) conducted a numerical analysis to examine the effects of stone columns and basal geosynthetic on the stability and deformations of an embankment over a soft deposit. After that, a thorough parametric study was carried out to determine how various crucial elements affected the behaviour of the embankment. The usage of stone columns greatly reduced the overall amount of soil deformations, according to the results. Although its impact diminished when a high stiffness geogrid was positioned underneath the embankment. Additionally, it was discovered that the stone column length had the greatest influence on the overall deformations of the embankment, therefore, increasing the length of the columns from 0.25Hs to 0.75Hs resulted in a reduction of the vertical and horizontal deformations of about 2 to 5 times. Similarly, decreasing the spacing between columns significantly reduces the amount of settlement under the embankment and maximum value of horizontal deformation decreases as depicted in Fig. 2.1

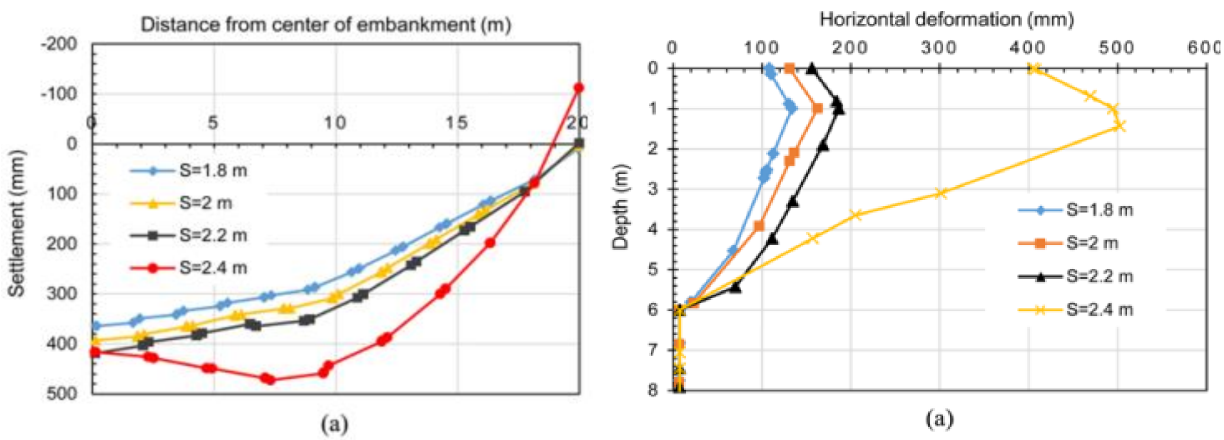


Fig. 2.1 Profile of the soil horizontal deformation and settlement under the embankment (Source: Ghorbani et al., 2020)

Hamzh et al. (2019) studied the bearing capacity of uniform and non-uniform stone columns in soft soil using two-dimensional (2D) Finite Element analysis with PLAXIS 2D. In the numerical studies, a Mohr-Coulomb constitutive soil model was used. By constructing two distinct diameters and lengths, the stone columns were modelled to be non-uniform as in Fig. 2.2. With tested ratios of d_2:d_1 of 1:2, 1:4, and 1:5, the top diameter d_1 was bigger than the lower half diameter d_2. The columns had a diameter length variation of nine various length ratios and a 10 m length (l_1: l_2). From the studies it was concluded that the non-uniform stone column that had the maximum bearing capacity had a d_2:d_1 ratio of 1:5 and most economical section has a ratio of 1:2.

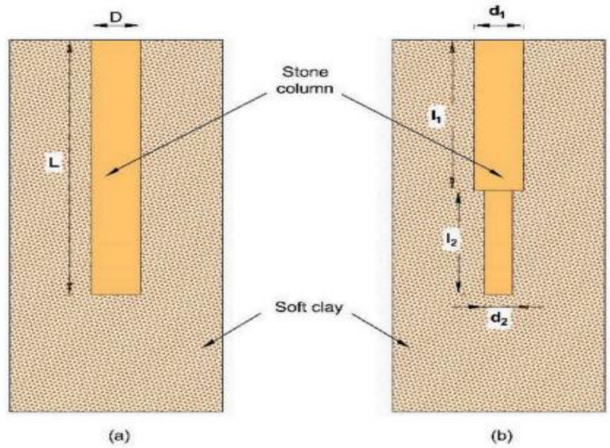


Fig. 2.2 Geometry of stone column (Source: Hamzh et al., 2019)

Thakur (2020), studied the effect of horizontal reinforcement on stone column with the help of PLAXIS 2D. From the study it was known that horizontal deformation prevents bulging of stone column by mobilization of friction. It proves to be better in load bearing capacity of soil when compared with encased columns. In unreinforced stone columns, lateral bulging is quite more in comparison with the reinforced one. From the study it was inferred that the ultimate load carrying capacity is more for horizontally reinforced stone columns than that for vertically reinforced stone columns. The PLAXIS 2D representation is given in Fig. 2.3.

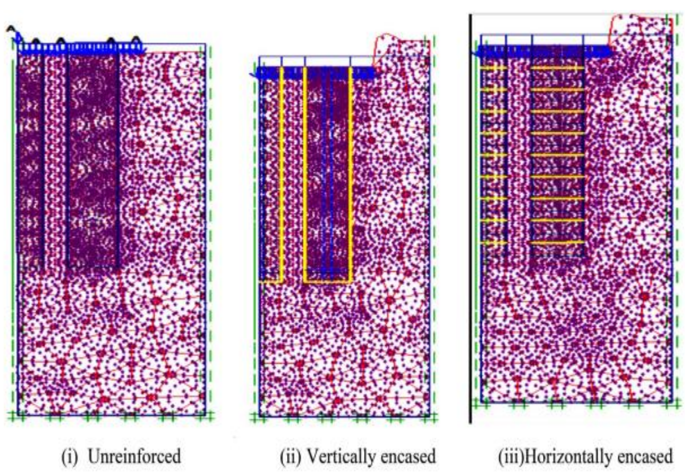


Fig. 2.3 Deformed mesh for group of 3 stone columns(Source: Thakur et al., 2020)

2.4.2 Embankment and consolidation study

Iman et al. (2018) studied the effect of consolidation under embankment using plane strain and axis symmetry method. Two-dimensional (2D) axisymmetric and plane strain study is done ,along with the three-dimensional (3D) numerical analysis of a test embankment on geotextile-encased columns (GECs) . A rectangular strip located beneath the embankment centerline served as the basis for the 3D numerical analysis. The proposed 3D strip model, according to numerical findings, accurately reflected the evolution of the measured pore pressure and deformations during the construction and post-contraction phases of the embankment. Contrary to the unit cell model, the settlement profile along the embankment base and the profile of the horizontal soil deformation beneath the embankment toes could both be accurately determined by plane strain and 3D analysis. The maximum settlement of an embankment can be easily ascertained using AX analysis utilizing the unit cell approach, however it is unable to project the settlement profile at the base of the embankment.

2.5 SUMMARY

Pavement evaluation and maintenance are integral part in the enhancement of serviceable life of the pavement. Functional and structural evaluation of pavement are done to evaluate the pavement distresses and are the most important part of the pavement management system. There are many methods to conduct functional evaluation including computation of roughness, skid resistance value etc. Most commonly adopted method for pavement evaluation is evaluation based on the visual examination survey and analyzing the quantity of each distress. Structural evaluation methods are done to identify pavement deflection characteristics and it helps in identification of exact location of the distress and its origin. Studies have proven that FWD test provides a better result compared to other conventional methods such as BBD. The overlay thickness determined using BBD are higher than that of FWD. Thus, designing using FWD would be economical. PLAXIS 2D is a finite element method for load settlement analysis of pavement along with subgrade layers. Analysis of stone column and their effect on subgrade can be studied using PLAXIS 2D. From the results of finite element analysis, the effect of geometry , spacing and reinforcement type on load settlement behaviour or bearing capacity of subgrade soil can be determined. From the studies on homogeneous soil using PLAXIS, it was found that economical shape for the stone column was achieved at a ratio of d_2:d_1 = 1:2 and a length ratio of l_1:l_2 = 3:7.

CHAPTER 3

METHODOLOGY

3.1 GENERAL

The study location identified in the present work is 900m long Kollam Asramam link road stretch. The width of the carriageway is 10m (2 lanes of 5m) with shoulder width of 2m to 2.5m. Pavement evaluation is carried out to find the adequacy of the pavement section in terms of structural and functional adequacy. The methodology adopted is discussed in Fig. 3.1.

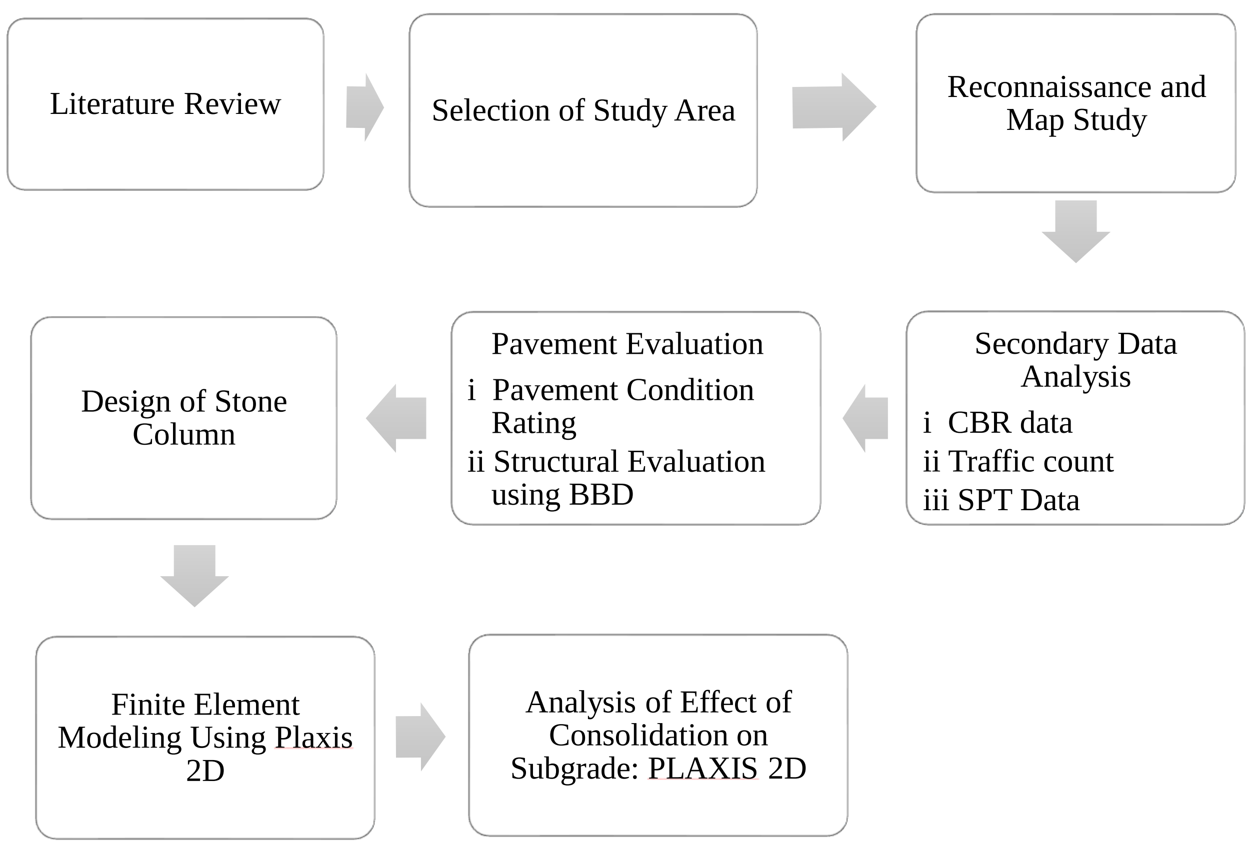


Fig. 3.1 Flow chart of methodology adopted

3.2 PAVEMENT CONDITION RATING

Manual distress data collection was done to identify various distresses present in the pavement section. As per IRC:82-2015 Pavement Condition Rating (PCR) can be provided on each homogeneous section selected. The entire road stretch is divided into 3 sections of 300x14m and pavement condition ratings are provided in the scale of 1 to 3. The various distresses considered are cracking, rutting, shoving, ravelling, potholes, patching and settlement. The pavement condition rating as per IRC:82-2015 is given in Table 3.1.

3.2.1 Field survey and data collection

Manual distress data collection was done to identify various distresses present in the pavement section. As per IRC:82-2015 pavement condition rating can be provided on each homogeneous section selected. The entire road stretch is divided into 3 sections of 300x14m and pavement condition ratings are provided in the scale of 1 to 3 .

Table 3.1 Pavement distress-based rating for highway (IRC:82-2015)

Distress type	Range of distress		
Cracking	>10	5 to 10	<5
Ravelling	>10	1 to 10	<1
Potholes	>1	0.1 to 1	<0.1
Shoving	>1	0.1 to 1	<0.1
Patching	>10	1 to 10	<1
Settlement	>5	1 to 5	<1
Rut depth	>10	5 to 10	<5
Rating	<1	1.1 - 2	2.1 - 3
	Poor	Fair	Good

Each distress represented possess its own weightage value that must be multiplied with the rating provided. The Final rating value is calculated by taking the average of the Weighted rating values of all parameters viz. cracking, ravelling, potholes, shoving, patching, settlement and rut depth. The weightage taken is represented in Table 3.2.

Table 3.2 Weightage given to each distress as per IRC (Source:IRC:82-2015)

Distress type	Weightage (Fixed) Multiplier factor
Cracking	1
Ravelling	0.75
Potholes	0.50
Shoving	1
Patching	0.75
Settlement	0.75
Rut depth	1

3.3 STRUCTURAL EVALUATION OF PAVEMENT USING BBD

3.3.1 Scope

This test procedure covers the determination of the rebound deflection of a pavement under a standard wheel load and tyre pressure, with or without temperature measurements.

3.3.2 Equipments

Basic equipment shall consist of:

1. Benkelman Beam
    a. Length of probe from pivot to probe position is 244cm

b. Measurement arm with length from pivot to dial gauge -122cm

c. Distance from pivot to front legs 25cm

d. Distance from pivot to rear legs 16cm

e. Lateral spacing of front support legs 33cm

2. A 5-tonne truck having 8170kg rear axle load equally distributed over the two wheels, equipped with dual tyres.
3. Tyre pressure measuring gauge
4. Thermometer (0-100°C) with 1^0 division
5. A mandrel for making 4.5cm deep hole in the pavement for temperature measurement. The diameter of the hole at the surface shall be 1.25cm and at bottom

3.3.3 Procedure

1. The chosen and designated test location on the pavement is marked. If the lane width is less than 3.5 meters, the point should be 60 cm from the pavement edge, and if the lane width is greater, it should be 90 cm from the pavement edge. The measurement locations should be 1.5 meters from the pavement for divided four-lane highways.
2. The dual wheels of the truck are centered above the selected point.
3. The probe of the Benkelman beam is inserted between the duals and placed on the selected point.
4. The locking pin is removed from the beam and the legs are adjusted so that the plunger of the beam is in contact with the stem of the dial gauge. The beam pivot arms are checked for free movement.
5. The dial gauge is set at approximately 1cm. The initial reading is recorded when the rate of deformation of the pavement is equal or less than 0.025mm per minute.
6. The truck is slowly driven a distance of 270cm and slopped.

7. An intermediate reading is recorded when the rate of recovery of the pavement is equal to or less than 0.025mm per minute.
8. The truck is driven forward a further.
9. The final reading is recorded when the rate of recovery of pavement is equal to or less than 0.02mm per minute.
10. Pavement temperature is recorded at least once every hour inserting thermometer in the standard hole and filling up the hole with glycerol (Fig. 3.2).
11. The tyre pressure is checked at two or three-hour intervals during the day and adjusted to the standard, if necessary.

Fig. 3.2 (a) Setting dial gauge to BBD (b) Testing pavement temperature

3.3.4 Calculations

1. Subtract the final dial reading from the initial dial reading. Also subtract the intermediate reading from the initial reading.
2. If the differential readings obtained compared within 0.025mm the actual pavement deflection is twice the final differential reading.
3. If the differential readings obtained do not compare to 0.025mm, twice the final differential dial reading represents apparent pavement deflection.

4. Apparent deflections are corrected by means of the following formula :

   $$X_t=X_a+2.91Y \tag{3.1}$$

   in which,

   X_t = True pavement deflection

   X_a = Apparent pavement deflection

   Y = Vertical movement of the front legs i.e., twice the difference between the final and intermediate dial readings.

5. The rebound deflection (%) shall be the twice of the X_t value.

3.3.5 Correction for Temperature and Moisture Content

The rebound deflection value obtained from BBD tests are affected by pavement temperature. When the study is conducted at cold temperature the pavement become stiffer and hence the rebound deflection value obtained are likely to be lower and vice versa in higher temperature. The standard temperature adopted for test is 35°C and necessary corrections are applied when there is variation in temperature from standard value. Therefore, when deflection studies are carried out at pavement temperatures other than 35°C, temperature correction is applied to the characteristic deflection value "De" at a rate of 0.01mm per °C variation from the standard temperature of 35°C.

The correction is positive or additive if the pavement temperature during the deflection study is lower than 35°C, the correction is negative if the pavement temperature is higher than 35°C. Flexible pavements are at their weakest during the monsoon season when subsurface moisture is at its maximum. Therefore, it is desirable to conduct a structural assessment survey of the BBD towards the end of the monsoon season. Appropriate values of the moisture correction factor can be applied to obtain deflection values corresponding to the highest soil moisture content. Also, the best subsoil moisture depends on the average annual rainfall in the area. Moisture correction factor charts were developed to account for seasonal variations in regions with different annual precipitation and different subsoil types. These charts are given in his IRC:81-1997 guidelines for overlay design based on

BBD research. In a BBD study, subsoil samples are taken to determine field moisture content and soil properties. A moisture correction factor table appropriate for the subgrade type is used to determine the value of the on-site moisture correction factor during the deflection survey.

3.4 DESIGN OF FLEXIBLE PAVEMENT OVERLAY USING BBD DATA

Steps adopted for the design are as follows

1. The data collected from BBD are analyzed and value of mean deflection, standard deviation, characteristic deflections are determined.
2. The modified value of characteristic deflection is determined by applying temperature and moisture corrections.
3. With the known values of initial traffic, Vehicle damage factor, lane distribution factor and design life, the design value of cumulative standard axle(CSA) load is determined in msa.
4. Corresponding to the corrected value of characteristic deflection Dc on X axis of the chart and using the selected design curve, obtain the overlay thickness on the Y axis which is given in terms of Bituminous Macadam (BM) overlay (Fig. 3.3).
5. Making use of equivalency factors of other overlay materials, the BM overlay is converted into desired overlay.

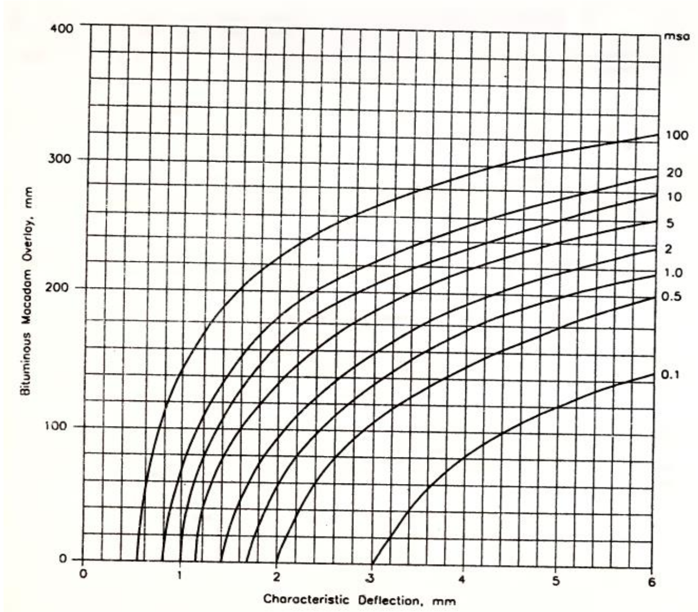


Fig. 3.3 Overlay design curve using BBD data (Source: IRC: 81-1997)

3.5 RESILIENT MODULUS OF SUBGRADE AND SUBSEQUENT LAYERS

Resilient modulus is regarded as the appropriate input for the linear elastic theory chosen for the analysis of flexible pavements because it is measured in a repeated load test while only accounting for the elastic (or resilient) component of the specimen's deformation (or strain). By performing the repeated tri-axial test as described in AASHTO T307-99, the resilient modulus of soils can be measured in a lab setting. Since these equipments are usually expensive, the following relationships may be used to estimate the resilient modulus of subgrade soil (MRS) from its CBR value,

$M_{RS} = 10.0 * CBR$ for $CBR \leq 5$ % (3.2)

$M_{RS} = 17.6 * (CBR)^{0.64}$ for $CBR > 5$ % (3.3)

Where,

M_{RS} = Resilient modulus of subgrade soil (in MPa).

CBR = California bearing ratio of subgrade soil (%)

Poisson’s ratio value or subgrade soil may be taken as 0.35.

The foundational or supporting layer's resilient modulus value and the granular layer's thickness both influence the elastic/resilient modulus value of the layer. Because larger deflections generated by loads result in de-compaction in the lower part of the granular layer, a weaker support does not allow for higher modulus of the upper granular layer. Equation 3.4 can be used to estimate the granular's modulus given its thickness and the supporting layer's modulus value.

$$M_{RGRAN} = 0.2(h) * 0.45 * M_{RSUPPORT} \quad (3.4)$$

Where, h = thickness of granular layer in mm

M_{RGRAN} = Resilient modulus of the granular layer (MPa)

$M_{RSUPPORT}$ = (effective) resilient modulus of the supporting layer (MPa)

For analytical purposes, the granular base and sub-base are treated as a single layer, and the combined layer is given a single modulus value. Therefore, using equation 3.4 and utilizing M_{RGRAN} as the modulus of the combined granular layer and $M_{RSUPPORT}$ as the effective modulus of the subgrade, the modulus of the single (combined) granular layer may be determined when the pavement combines a granular base and granular sub-base. However, if a base layer that has been treated with cement or an emulsion/foam bitumen is employed on top of a granular sub-base, both layers must be taken into account individually in the study and given different modulus values. Equation 3.4 can be used to estimate the modulus of the granular sub-base taking M_{RGRAN} as the modulus of the granular sub-base layer and $M_{RSUPPORT}$ as the effective modulus of the subgrade.

3.6 DESIGN OF STONE COLUMN

The stone column technique is a very effective method of improving soil strength parameters such as bearing capacity and reducing consolidation settlement. It offers a much more economical and sustainable alternative deep foundation solutions. The improvement of the terrain when implemented using the stone column technique helps to provide a much more stable construction solution in weakly cohesive soils. 25 borehole datas are available on the given section for a depth of 10m to 48m. From the borehole data it is clear that the SPT value of the subgrade soil is less than 3 for a depth of about 6 to 8m and there is presence of lateritic fill without adequate compaction and very soft soil below at about 6 to 14m. Compression of this weak soil and excessive consolidation is expected to be the cause of the longitudinal crack and settlement. Since both strength and consolidation effect has to be improved stone column is the best alternative to improve the soil condition.

3.6.1 Design parameters

(i) Diameter (D)

The diameter of the stone column generated increases as the soil becomes softer. Depending on the soil type, its undrained shear strength, the stone size and the construction method, the completed diameter of the hole is always greater than the initial diameter of the casing. The known compacted volume of material needed to fill the hole with a known length and densities may be used to estimate the approximate diameter of the stone column in the field. Generally, the diameter of stone column varies from 0.5m to 1.2m.

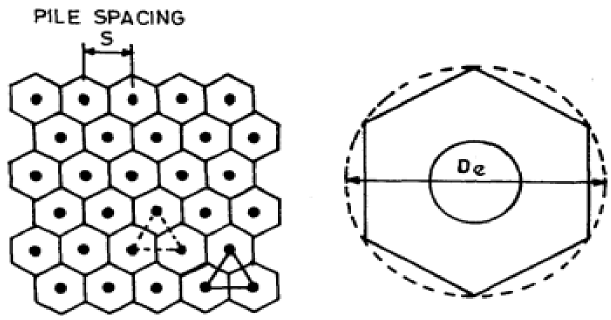
(ii) Pattern and effective diameter

Stone column can be installed both in square as well as triangular pattern. But it is preferable to follow triangular pattern for better or dense packing. For each case the effective diameter changes. Each stone column has a tributary area that forms a regular hexagon around the column. Distribution under different pattern is shown in Fig. 3.4. It can be represented as an equal circular area with the same overall area. The equivalent circle has an effective diameter (De) which is given by following equation:

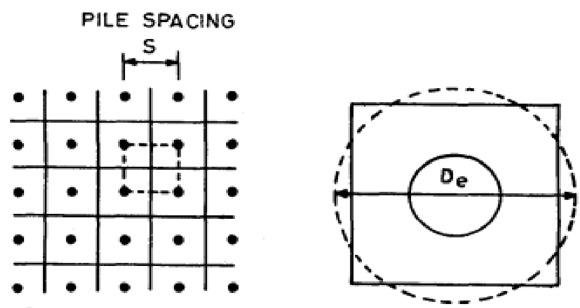
D_e = 1.05 S for an equilateral pattern and

= 1.13 S for a square pattern

Where, S = spacing of the stone columns



1A - TRIANGULAR ARRANGEMENT OF STONE COLUMNS



1B - SQUARE ARRANGEMENT OF STONE COLUMNS

Fig. 3.4 Arrangement of stone columns

(iii) Spacing (S)

Spacing of the stone column can broadly range from 2 times to 3 times the diameter depending on soil type, column material, installation method. A minimum spacing of 2 times diameter is preferrable based on previous studies.

(iv) Replacement Ratio

The composite ground representing an indefinitely large loaded region may be modelled as a unit cell consisting of the stone column and the nearby tributary soil for the purposes of settlement and stability study. The word replacement ratio, A_r is used to specify how much soil the stone replaced.

$$A_r = A_C/A$$

Where, A_C = Area of the stone column

A = Total area within the unit cell

$$A_r = 0.907\ (D/S)^2$$

where the constant 0.907 is a function of the pattern used which, in this case, is the commonly employed equilateral triangular pattern.

(v) Material size

Generally, the size of material used ranges from 6 to 20mm. But the size of the material adopted is largely affected by the property of soil and cost of construction.

3.6.2 Design using Priebe's method

Priebe's method is a semi empirical method for design of stone column and estimation of settlement. In a first step an improvement factor, denoted by n_0, is introduced by which stone columns improve the mechanical characteristics of the initial soil in comparison with its pretreatment properties (without columns). According to this improvement factor, young's modulus of the composite system is increased and, subsequently, the settlement is reduced.

Based on the assumption that the stone column material shears from the start while the surrounding soil behaves elastically, the improvement of a soil by the installation of stone columns is assessed. Additionally, it is assumed that during the installation of the stone column, the soil surrounding the column had already developed to the point where its initial resistance equal that of a liquid, with a coefficient of earth pressure of K=1. Using the fundamental improvement factor n_0, the outcome of such an evaluation is represented as follows

$$n = 1 + \frac{Ac}{A}\left(\frac{\frac{1}{2}+F}{K*F} - 1\right) \tag{3.5}$$

$$F = \frac{(1-\vartheta)\left(1-\frac{Ac}{A}\right)}{1-2\vartheta+\frac{Ac}{A}}\ . \tag{3.6}$$

$$Kac = tan^2(45 - \frac{\theta}{2}) \tag{3.7}$$

Where,

n_0= Basic improvement factor

ν_s= Poisson's ratio of soil

Kac= Earth pressure

$\emptyset_c$ = Angle of friction

The stone column that gives higher settlement improvement factor or lower settlement reduction ratio is selected.

3.7 FINITE ELEMENT MODEL

PLAXIS 2D is a finite element package that has been developed specifically for the analysis of deformation and stability in geotechnical engineering projects. The simple graphical input procedures enable a quick generation of complex finite element models, and the enhanced output facilities provide a detailed presentation of computational results. The calculation itself is fully automated and based on robust numerical procedures.

3.7.1 Numerical modelling of pavement section using PLAXIS 2D

Finite element modelling using PLAXIS is depicted through following steps

(i) Boundary condition and mesh discretization

Two-dimensional finite-element analyses for the flexible pavements are performed using PLAXIS 2D which is an effective FEM-based software, used to perform the deformation and stability analysis of geotechnical engineering activities. For the design of flexible pavements, the thickness of various pavement layers, such as subgrade, sub-base, base, and bitumen layer, is needed. At the boundaries, a standard fixity should be used, which means that only horizontal displacement is restricted in the vertical boundaries, whereas both vertical and horizontal displacements are restricted at the bottom boundary.

Mesh discretization should be done using 15-noded triangular elements. Additional mesh refinement should also be carried out at the interfaces of different layers of the pavement. Analysis with 'fine mesh' gives results in higher accuracy but of course with the cost of reasonable computational time. According to IRC: 37-2018, the responses of the pavement is to be analyzed under static loading conditions. The loading is assumed to be circular with a tyre pressure of 565 kPa.

(ii) Material Properties

The axisymmetric linear elastic model is usually considered for unreinforced flexible pavements. Axisymmetric modelling was chosen, because it could simulate circular loading. A model of flexible pavement section contains bituminous layer, granular base layer, granular sub-base layer, and subgrade layer. In the analysis, the different pavement layers should be modeled using the 15-noded structural solid element. For the flexible pavement design, the thickness of various layers and the resilient modulus or elastic modulus of different layers have been calculated from IRC: 37-2018 guidelines as in Table 3.3.

Table 3.3 Material properties for PLAXIS 2D

Material	Asphalt Concrete	Base	Subbase	Subgrade
Model	Linear elastic	Mohr Coulomb	Mohr Coulomb	Mohr Coulomb
Thickness (m)	0.095	0.200	0.210	5
Young’s modulus (KPa)	3000×10^3	$244.4*10^3$	244.4×10^3	20×10^3
Poisson’s Ratio	0.35	0.35	0.35	0.45
Dry density (kN/m^3)		18	17	17
Saturated density (kN/m^3)		18	17	18
Cohesion (kN/m^2)				10
Friction angle(^{o})		33	33	24.3

(iii) Analysis

Various phases are considered to represent the different states of construction in the actual Field scenario. The study is conducted in four steps:

a) Initial phase:

   In this phase, stresses are generated within the soil volume by Ko-procedure which takes into account the stress history of soil.

b) Phase 1: In this phase, subgrade and sub-base layers of pavement are activated from the explorer window

c) Phase 2: In this phase, base and bituminous layers of unreinforced pavement are activated from the explorer window

d) Phase 3: In this phase, the loading condition is activated simulating the deformation of the pavement section. The displacements are applied to the nodes of the pavement subgrade layer using a displacement control function available in PLAXIS 2D.

Three such models are adopted, unreinforced model and 3 model with stone columns with diameter 0.5m, 0.8m and 1m (Fig. 3.5). The spacing adopted is 2 times the diameter for all the cases. In such case the stone column is activated at phase 1 before pavement activation.

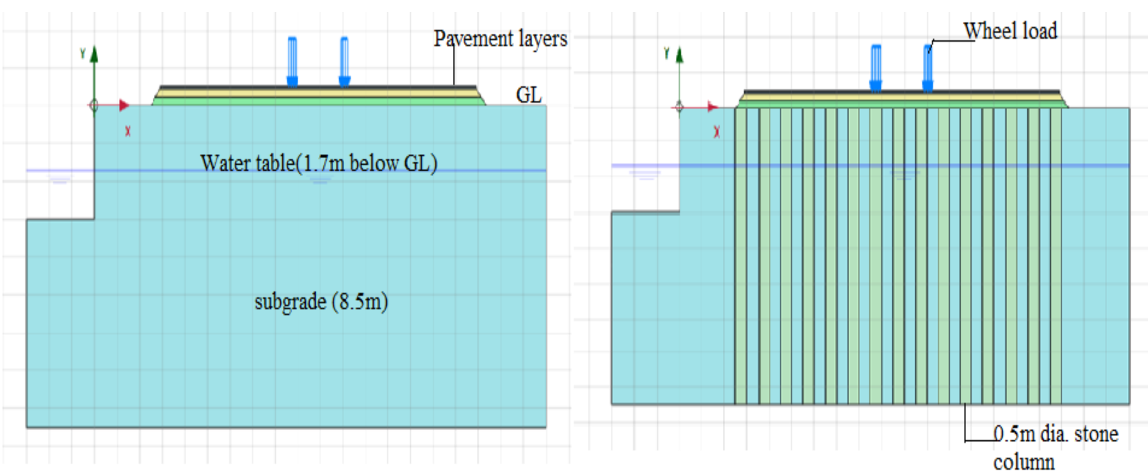


Fig. 3.5 Unreinforced and reinforced model (0.5m dia.) created using PLAXIS 2D

3.7.2 Validation of PLAXIS 2D using IITPAVE

IITPAVE software, developed by IIT Kharagpur, is an elastic multilayer linear analysis tool that is used to design unreinforced pavements . The stresses, strains, and defection for a standard axle load were computed using this software at the critical points for unreinforced pavement, as recommended by IRC: 37-2018. In this simulation, structural analysis of unreinforced pavement is carried out using this IITPAVE software, with required inputs such as the layer thicknesses, moduli, Poisson's ratio values, the standard axle load of 80 kN distributed on four wheels (20 kN on each wheel), and tyre pressure as 0.56 MPa. Since the analysis is performed for the standard axle of 80 kN, an effective single-wheel load of 40,000 N is given as an input parameter. The estimation of resilient subgrade modules for different pavement layers is done based on the empirical relationship.

Once all the required parameters are given as input, the program is executed using the RUN option. After the execution of the program, output has been displayed with strain and displacement values at various locations. The horizontal and vertical strain value from the design model from Plaxis 2D is compared with that obtained from IITPAVE .

3.7.3 Consolidation study using PLAXIS 2D

The construction of an embankment on soft soil with a high groundwater level leads to an increase in pore pressure. As a result of this undrained behaviour, the effective stress remains low and intermediate consolidation periods have to be adopted in order to construct the embankment safely. During consolidation the excess pore pressures dissipate so that the soil can obtain the necessary shear strength to continue the construction process.

(i) General description

The embankment is 20m wide and 3m high. The slopes have an inclination of 1:3. Plane strain model is adopted since water level is present only on one side. The embankment itself is composed of hardened soil. The subsoil consists of 8.5m of soft clay. The phreatic level is located 1.7m below the original ground surface.

(ii) Material properties

The material properties adopted is given in Table 3.4. For the purpose of consolidation hardening soil model is adopted with embankment of 1.5m each is provided one above other to reach a total height of 3m. The main parameter that is considered for stone column is the friction angle. To get desired settlement improvement, friction angle of 44º is considered for all stone columns that are used in the analysis.

Table 3.4 Material properties adopted in PLAXIS 2D

Material	Subgrade	Embankment	Stone column
Model	Mohr Coulomb	Hardening soil	Mohr Coulomb
Thickness (m)	8	3	8.5
Young's modulus (KPa)	20×10^3	$25x10^3$	$20x10^4$
Poisson's Ratio	0.45	0.5	0.3
Dry density (kN/m^3)	17	16	16
Saturated density (kN/m^3)	18	19	20
Cohesion (kN/m^2)	10	1	
Friction angle(º)	24.3	30	12

(iii) Model configuration

A consolidation analysis introduces the dimension of time in the calculations. In order to correctly perform a consolidation analysis a proper time step must be selected. The embankment construction is divided into two phases. After the first construction phase a plastic period of 2 to 3 days is introduced to allow the excess pore pressures to dissipate.

After the second construction phase consolidation period is introduced from which the final settlements may be determined. Hence, a total of four calculation phases has to be defined besides the initial phase.

Initial phase

In initial phase the stress within the soil is generated within the soil volume by Ko-procedure which takes into account the stress history of soil. In the initial situation the embankment is not present. The remaining active geometry is horizontal with horizontal layers, so the K0 procedure can be used to calculate the initial stresses.

Phase 1

In phase 1 the plastic stage is adopted under undrained condition . Here first stage of embankment layer is provided. The time taken for consolidation is 3 to 2 days.

Phase 2

In this phase consolidation analysis is done. Here 80% consolidation is adopted and time taken along with excess pore pressure generated is determined.

The same steps are adopted in subsequent phases until consolidation is completed. When stone column is adopted, the stone column is activated at initial phase and all other phases remain same.

For consolidation analysis, 3 reinforcement models are adopted. Three stone columns of diameters 0.5m, 0.8m and 1m are used and the spacing between each stone columns are taken as twice the diameters. The total depth of stone columns adopted in the design is 8.5m from the ground level. The variation of excess pore pressure and time to complete consolidation is determined for each case. The model created using stone column and unreinforced sections are shown in Fig. 3.6.

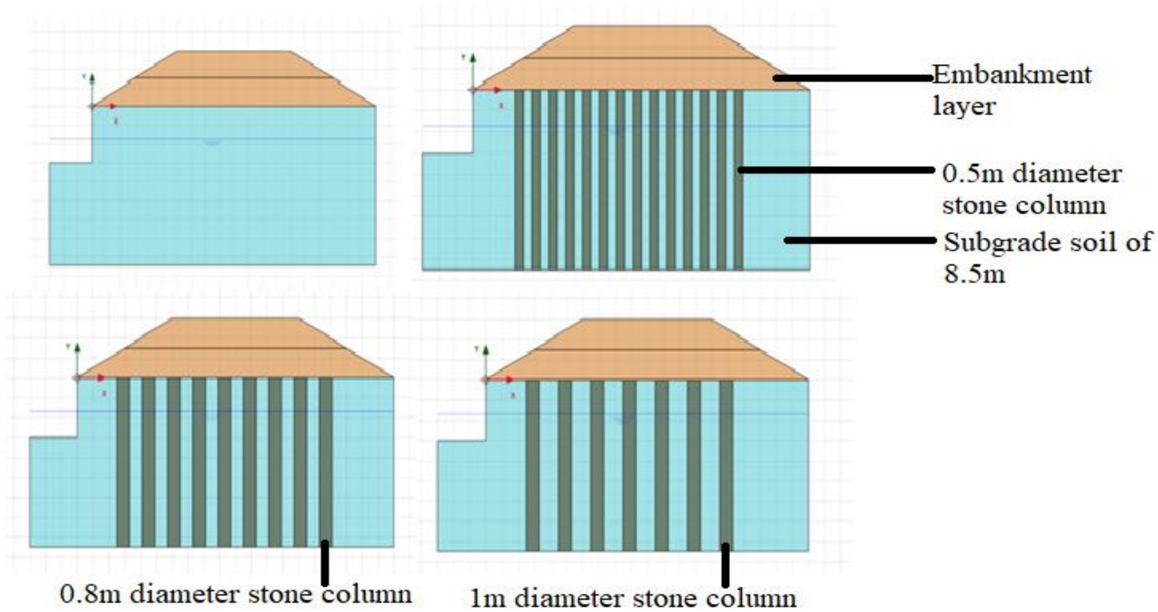


Fig. 3.6 Model created for no treatment ,0.5m diameter, 0.8m diameter and 1m diameter stone column condition

3.8 SUMMARY

As initial step, Functional evaluation of the pavement section has to be done. Pavement condition rating is done by visual examination survey and distress percentage is quantified. Structural evaluation of the pavement section is done using BBD, where static load is applied and rebound deflection is found. The thickness of overlay required is determined using this method. For the purpose of subgrade stabilization stone column is adopted in current project and design of stone column is done using Priebe's method. The suitability of stone column and settlement reduction is determined using PLAXIS 2D software. For that purpose, 3 stone column diameters are adopted such as 0.5m, 0.8m and 1m. Consolidation effect and settlement improvement for each case is separately analyzed using PLAXIS 2D.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 PAVEMENT CONDITION RATING

The entire pavement section is divided into 3 sections as in Fig. 4.1. For each section quantity of each distress is calculated. Various distresses considered in the study as per IRC:82-2015 are rutting, cracking, settlement, potholes, ravelling, patchwork etc. Based on the percentage of distress and weightage of each distress as per IRC:82-2015 . Weighted rating value is estimated for each distress as in the Table 4.1.

Fig. 4.1 Sections considered for evaluation (Source: https://earth.google.com)

From the results of functional evaluation of Kollam Asramam link road, it is found that the road section contains combination of different distresses. Section 2 is the section which is adjacent to lake and this region has more seepage compared to other regions. Moisture significantly affects the flexible pavement by weaken the natural gravel materials especially the subgrade support strength.

Table 4.1 Rating provided for each section

Distress type	Weighted rating value(Section 1)	Weighted rating value(Section 2)	Weighted rating value(Section 3)	
Cracking	2.1	2.1	1.2	Pavement Condition Rating =1.6
Ravelling	1.5	1.5	1.5	
Potholes	1.05	0.6	1.05	
Shoving	3	3	3	
Patching	1.125	0.825	1.57	
Settlement	1.5	1.6	2.25	
Rut depth	2.1	1	2.1	
	1.7	1.5	1.8	

As per IRC:82-2015 the pavement section is under fair condition but requires maintenance. The middle section of the pavement i.e., 300 to 600m stretch possess less pavement condition rating compared to the other sections. This is due to the poor drainage facility in the region and effect of the water body adjacent to the section. Compared to other section this region has a greater number of patches works and maintenance activities. Also rutting in this region is more compared to the other regions representing poor load bearing capacity of the pavement section.

4.2 DEFLECTION OF THE PAVEMENT SECTION

In order to determine the deflection of pavement section Benkelman beam deflection test was conducted. A static load of 80 kN and tyre pressure of 0.56 kg/m^2 was applied for generation of deflection as in Fig. 4.2. The field data obtained from the test are depicted in Table 4.2 and Table 4.3.

Fig. 4.2 Conducting BBD test on pavement surface

Total chainage for the given section is 800m. The temperature of the pavement at an interval of 3 hour is determined to provide necessary temperature corrections. The standard temperature adopted is 35℃.

Table 4.2 Temperature of pavement at different time and chainage

Asramam to link road (ch: 0/000 to 0/800)			
Chainage	Pavement Temperature, °C	Date	Time
0/000	24	11/25/2022	8.30 AM
0/800	31	11/25/2022	12.00 PM

Table 4.3 Dial gauge reading for different test locations

Benkelman beam deflection data					
Asramam to link road (ch: 0/000 to 0/800)					
Location of Test Point	Pavement Temp, °C	Dial Guage Reading			Remarks
		Initial	Inter(mm)	Final(mm)	
0/000	24	100	25	30	LHS
0/050	24	100	26	28	RHS
0/100	24	100	30	32	LHS
0/150	24	100	28	29	RHS
0/200	24	100	27	27	LHS
0/250	24	100	30	31	RHS
0/300	24	100	35	35	LHS
0/350	24	100	33	34	RHS
0/400	24	100	30	31	LHS
0/450	24	100	26	27	RHS
0/500	24	100	30	29	LHS
0/550	24	100	32	31	RHS
0/600	24	100	29	29	LHS
0/650	24	100	30	31	RHS
0/700	24	100	28	29	LHS
0/750	24	100	30	31	RHS(CURVE)
0/800	31	100	29	27	LHS Link Road

Table 4.4 Final deflection data after correction

Chainage	Pavement Temp (°C)	Dial gauge reading			Rebound Deflection (mm)	Correction factors for		Corrected Deflection
		D_0	D_i	D_f		Temp.	Moisture	
0/000	24	100	25	30	1.40	0.11	1.01	1.525
0/050	24	100	26	28	1.44	0.11	1.01	1.566
0/100	24	100	30	32	1.36	0.11	1.01	1.485
0/150	24	100	28	29	1.42	0.11	1.01	1.545
0/200	24	100	27	27	1.46	0.11	1.01	1.586
0/250	24	100	30	31	1.38	0.11	1.01	1.505
0/300	24	100	35	35	1.30	0.11	1.01	1.424
0/350	24	100	33	34	1.32	0.11	1.01	1.444
0/400	24	100	30	31	1.38	0.11	1.01	1.505
0/450	24	100	26	27	1.46	0.11	1.01	1.586
0/500	24	100	30	29	1.42	0.11	1.01	1.545
0/550	24	100	32	31	1.38	0.11	1.01	1.505
0/600	24	100	29	29	1.42	0.11	1.01	1.545
0/650	24	100	30	31	1.38	0.11	1.01	1.505
0/700	24	100	28	29	1.42	0.11	1.01	1.545
0/750	24	100	30	31	1.38	0.11	1.01	1.505
0/800	31	100	29	27	1.46	0.04	1.01	1.515

After applying the correction the mean deflection value obtained(Dm)= 25.836/17=1.52 mm

Standard deviation of the sample set (S)= 0.044

Characteristic deflection =Dm +2S=1.52+(2*.044)=1.607 mm

For the corresponding deflection value and Design traffic load(msa) the overlay thickness required for pavement is determined from the graph in IRC:81-1997. Thus for a traffic load of 19 msa and characteristic deflection value of 1.607mm, the overlay thickness in terms of bituminous macadam is 150mm. Since dense graded bituminous layers are to be used an equivalency factor of 0.7 is multiplied with 150mm. Therefore a DBM of 105mm is required as an overlay.

4.3 STONE COLUMN DESIGN

Stone columns are constructed where in the soft soil is strengthened by replacing a certain percentage of soil with aggregate. The aggregate column will act as a drainage channel to release the excess pore water present in the subsoil.

4.3.1 Parameters considered in design

Priebes method is adopted for the design of stone column. The various parameters adopted for the design is shown in the Table 4.5 as per IS 15284-1.

Table 4.5 Parameters considered for design as per IS 15284-1

Parameters	Value	
Diameter	0.5m to 1.5m	
Pattern	Triangular/Square	
Effective Diameter	Triangular	1.05S
	Square	1.135S
Spacing	2 to 3 dia	
Replacement Ratio	As/A	
Material of Stone	6mm to 40mm	

For better or dense packing, triangular pattern is adopted for the design.

4.3.2 Area ratio for different spacing

Area replacement ratio is used to quantify the amount of soil replaced by stone column. For different diameters of stone column (0.5m to 1m), area replacement ratio is determined for 2 times diameter and 3 times diameter condition.The result obtained is provided in Table 4.6 and 4.7.

Table 4.6 Area ratio for 2 times diameter spacing

Diameter	Spacing(2d)	Effective diameter	Area ratio
0.5	1	1.05	0.226757
0.6	1.2	1.26	0.226757
0.7	1.4	1.47	0.226757
0.8	1.6	1.68	0.226757
0.9	1.8	1.89	0.226757
1	2	2.1	0.226757

Table 4.7 Area ratio for 3 times diameter spacing

Diameter	Spacing(3d)	Effective diameter	Area ratio
0.5	1.5	1.57	0.100781
0.6	1.8	1.89	0.100781
0.7	2.1	2.20	0.100781
0.8	2.4	2.52	0.100781
0.9	2.7	2.83	0.100781
1	3	3.15	0.100781

Stone column with smaller spacing (2d) gives higher area ratio of 0.226 compared to larger spacing.

4.3.3 Selection of Diameter and Spacing

Based on the area ratio, friction angle of stone column material, active earth pressure and poissons value of soil settlement improvement factor(SIF) or settlement reduction ratio can be determined as per priebes method. The table 4.8 shows SIF for various area ratios.

Table 4.8 Settlement Improvement Factor for different area ratios

Area ratio	Friction Angle(°)	K_{ac}	Settlement Improvement Factor	Settlement reduction Ratio
0.1	36	0.259616	1.424278	0.70211
0.2	36	0.259616	1.955552	0.511365
0.3	36	0.259616	2.639677	0.378834
0.4	36	0.259616	3.553077	0.281446
0.1	40	0.217443	1.525963	0.655324
0.2	40	0.217443	2.179673	0.458784
0.3	40	0.217443	3.01588	0.331578
0.4	40	0.217443	4.125831	0.242375
0.1	44	0.180179	1.655422	0.604076
0.2	44	0.180179	2.465012	0.405678
0.3	44	0.180179	3.494842	0.286136
0.4	44	0.180179	4.855031	0.205972

For area ratio from 0.1 to 0.4, settlement reduction ratio is determined and it can be seen that settlement reduction ratio, which is the ratio of settlement of treated soil to untreated soil reduces with increase in area ratio. Since in our study the area ratio is limited to 0.2, the spacing that can be adopted is 2 times the diameter. It can be said that stone column having area ratio 0.2 show better reduction in settlement, hence spacing of 2 times diameter is adopted for modelling using PLAXIS 2D. Similarly the settlement improvement factor is high for friction angle of 44º, hence it is adopted for modelling stone column.

4.4 VALIDATION OF PLAXIS 2D USING IITPAVE

IITPAVE software is used in this study for validation of PLAXIS 2D modeling.

4.4.1 Numerical modelling using PLAXIS 2D

The pavement layer is modelled as per input values from the field condition. A Plane Strain model was utilized in the analysis using 15 noded structural solid elements with medium refinement. The material properties adopted are given in the Table 4.9.

Table 4.9 Material properties for PLAXIS 2D

Material	Asphalt Concrete	Base	Subbase	Subgrade
Model	Linear elastic	Mohr Coulomb	Mohr Coulomb	Mohr Coulomb
Thickness (m)	0.095	0.200	0.210	5
Young's modulus(KPa)	3000×10^3	$244.4*10^3$	244.4×10^3	20×10^3
Poisson's Ratio	0.35	0.35	0.35	0.45
Dry density (kN/m^3)		18	17	17
Saturated density (kN/m^3)		18	17	18
Cohesion (kN/m^2)				10
Friction angle(º)		33	33	24.3
Dilatation angle (º)		3	3	

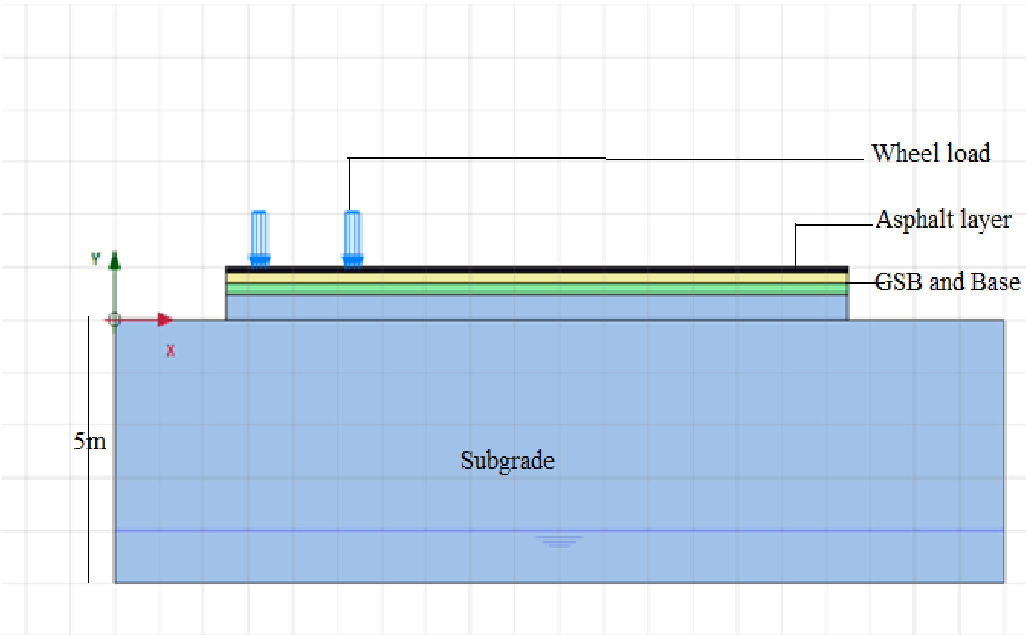


Fig. 4.3 Model developed in PLAXIS 2D

Pavement layers including the subgrade is modelled using PLAXIS 2D as shown in Fig.4.3. Static wheel load with tyre pressure 0.56MPa is provided at a distance of 0.6m from shoulder region. Dual wheel load condition is adopted in this model.

(i) Output stage

Various outputs that can be generated by PLAXIS are load-displacement curves, stress paths, animations, report generators, structural forces and displacements, stress and strain values etc.

Here both horizontal and vertical strain value directly below the wheel load is determined for validation purpose. Strain values at bottom of bituminous layer (Y=0.92) and top of subgrade layer is determined. The maximum strain value from PLAXIS 2D modelling is shown in Table 4.10. Strain values at points directly below the wheel load is determined in this case.

Table 4.10 Strain values

X [m]	Y [m]	Horizontal strain(ε_1)	Vertical strain(ε_3)
3.45	0.92	-2.45E-04	2.98E-04
5.12	0.92	-2.13E-04	3.18E-04
At top of subgrade		-1.07E-03	3.67E-04

4.4.2 Multilayer Linear Analysis of Pavement

IITPAVE has been created to analyze linear elastic layered pavement systems. Using this programme, it is possible to calculate the stresses, strains, and defections brought about at various points in a pavement by a single load that is uniformly distributed and delivered to a circular contact area at the pavement's surface.

In this simulation, structural analysis of unreinforced pavement is carried out using the IITPAVE software with required inputs such as the layer thicknesses, moduli, Poisson's ratio values, the standard axle load of 80kN distributed on four wheels (20kN on each wheel), and tyre pressure as 0.56MPa. Since the analysis is performed for the standard axle of 80 kN, an effective single-wheel load of 40,000N is given as an input parameter. The horizontal and vertical strain value at bottom of bituminous layer and top of subgrade are $0.25*10^{-3}$ and $0.39*10^{-3}$ respectively as shown in Fig.4.4.

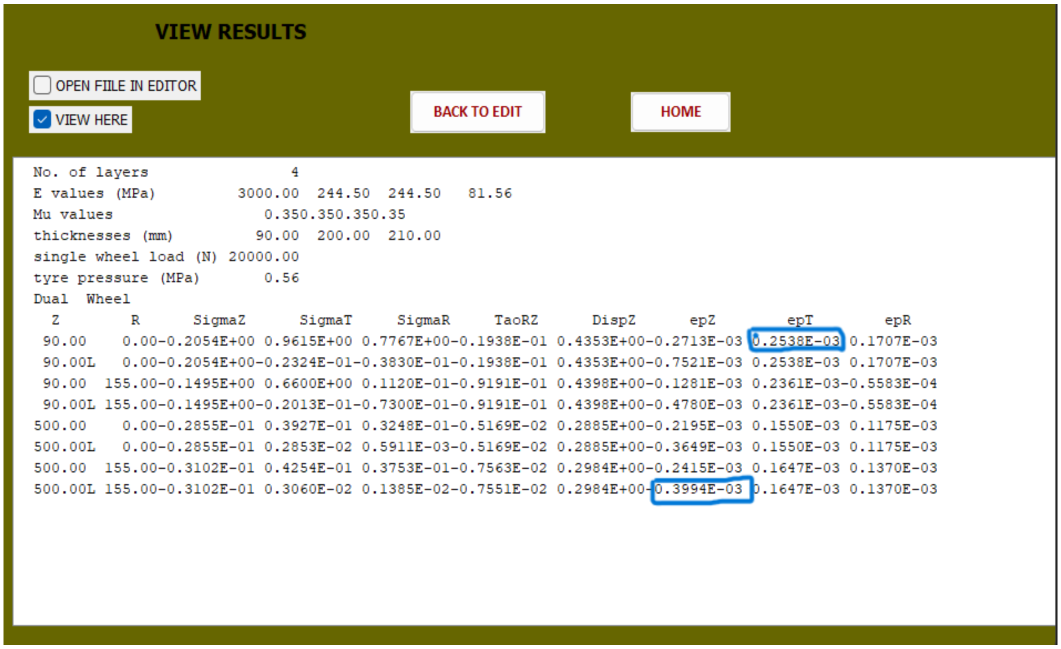


Fig. 4.4 Horizontal and vertical strain from IITPAVE

4.4.3 Validation result

The points X=3.45 and 5.12 represents points where the wheel load is applied . Horizontal and vertical strains obtained from Plaxis 2D and IITPAVE software is compared in Table 4.11.

Table 4.11 Results of strain values from IITPAVE and Plaxis 2D

Strain values	IITPAVE	PLAXIS 2D		
		X	Y	
Horizontal strain	$0.2538*10^{-3}$	3.45	0.95	$-0.245*10^{-3}$
		5.12	0.92	$-0.213*10^{-3}$
Vertical strain	$0.399*10^{-3}$	$0.367*10^{-3}$		

From the results obtained from IITPAVE and PLAXIS 2D it can be inferred that the horizontal and vertical strain values show significant similarity i.e., the variation is less than 5%.

4.5 NUMERICAL MODELLING OF PAVEMENT SECTION USING PLAXIS 2D

Four models are prepared for settlement analysis including pavement section. An unreinforced section and 3 sections with stone column diameter of 0.5m, 0.8m and 1m diameter are adopted. The settlement at surface level and at mid depth of subgrade is analysed in following sections. Here static wheel load is applied at mid point of the two lanes as it represents the featured section.

4.5.1 Analysis without stone column

The effect of static traffic loading on unreinforced pavement section is depicted in Fig. 4.5.

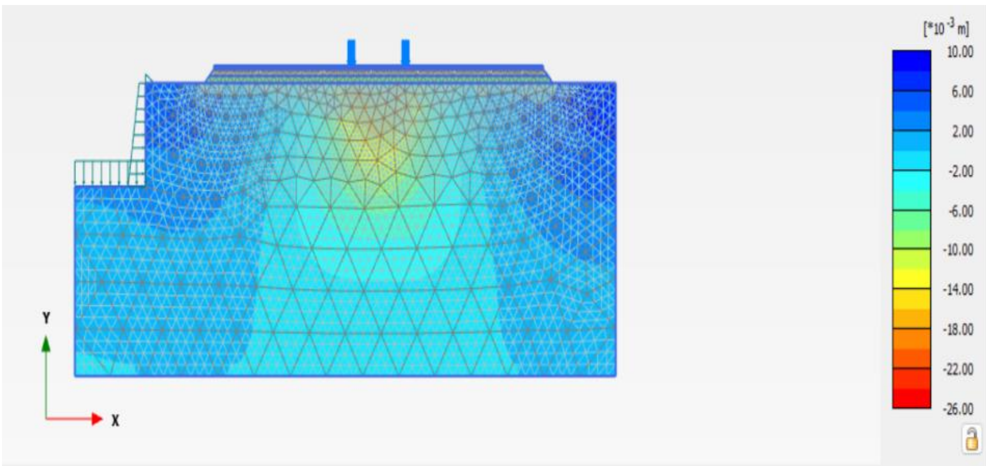


Fig. 4.5 Dispacement model from PLAXIS 2D

The maximum displacement value of 8mm occurs directly below the traffic wheel load. This is graphically represented in Fig. 4.5. The excess pore pressure developed due to effect of nearby water body can be seen in Fig. 4.6 .The maximum pore pressure developed is 26.85 kN/m^2 and prominent pressure is visible near the lake side and under the wheel load.

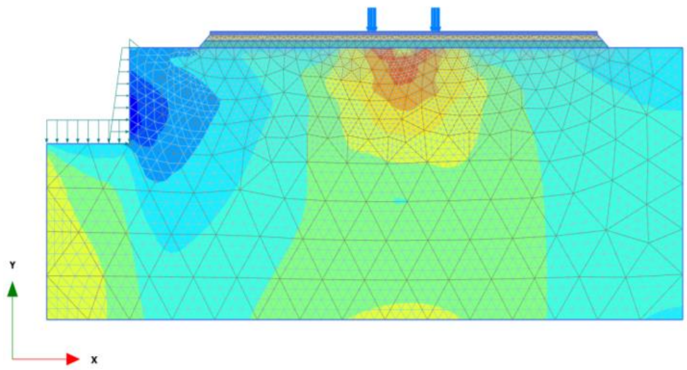


Fig. 4.6 Excess pore pressure diagram

Under non-reinforced condition maximum excess pore pressure occurs at region directly below the load. The magnitude of pressure reduces with depth as in Fig 4.6. Where as the maximum positive pressure occurs at region that is directly in contact with the lake(Blue shade). This positive pressure reduces with the distance from the water front.

4.5.2 Analysis with stone column

The deformed mesh diagram for stone columns of different diameters are depicted in Fig.4.7. Stone columns of diameter 0.5m and 0.8m is shown with dual wheel load applied at center of carriageway. Deformation (Scaled) that occur due to this traffic load is prominent directly below the wheel load. Changes in the mesh due to hydrostatic pressure can be seen from this mesh diagram.

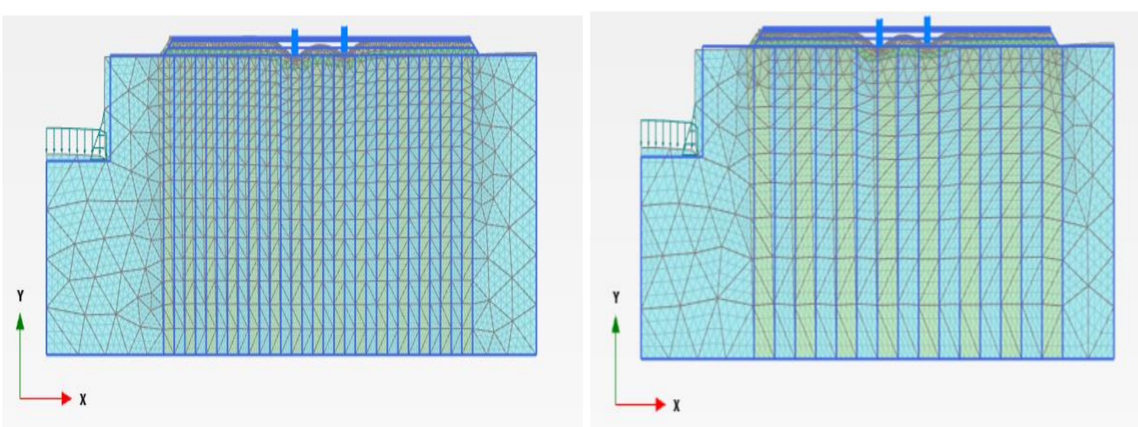


Fig. 4.7 Deformed mesh diagram

4.5.3 Settlement analysis

Settlement at surface level for different stone column diameter and non-reinforced condition is shown in the Fig. 4.8.

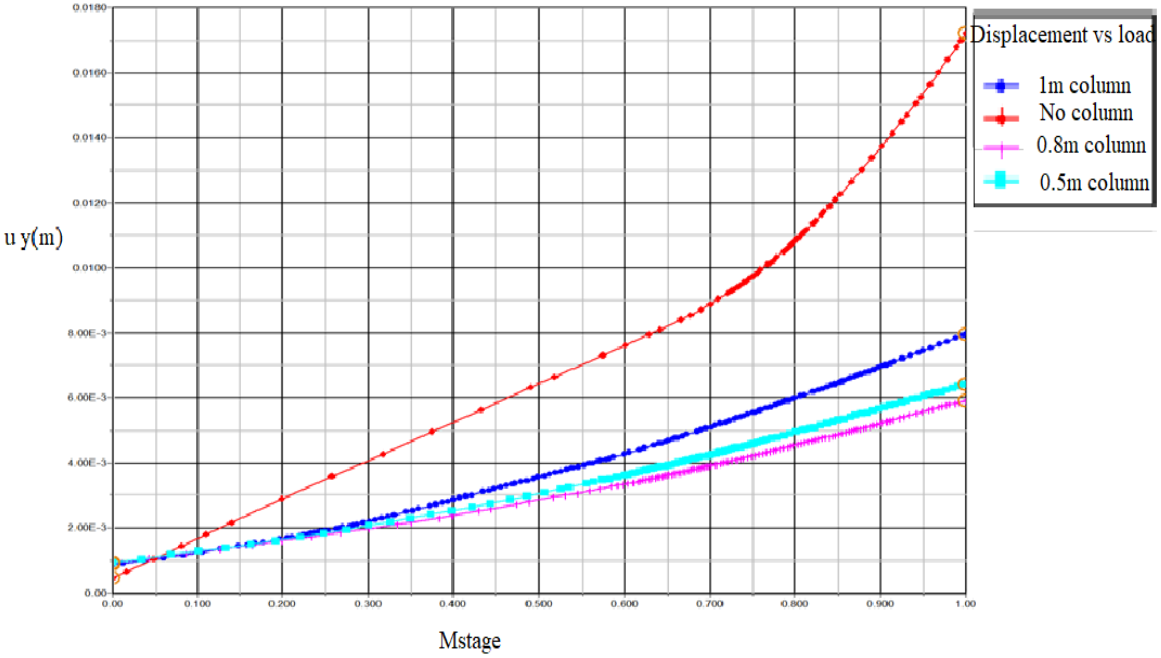


Fig. 4.8 Settlement vs load graph for different stone column diameters

From the graph it can be seen that settlement for unreinforced model is high as compared to the reinforced model with stone columns. For reinforced sections, stone column with 0.5m diameter have lowest settlement and stone column with 1m diameter has highest settlement. This is due to low spacing between the stone columns and densely packed condition. Since the stone columns are tightly packed the soil layer between the stone columns will be in compacted condition.

Maximum volumetric strain in the entire section upto a depth of 8.5m is 0.305 and maximum displacemnt is 24.27mm. Similar to the surface condition 0.5m diameter stone column has maximum reduction in settlement. Addition of stone column reduces the maximum displacement to 55%. The maximum strain and displacement value for each stone column model is shown in Table 4.12.

Table 4.12 Maximum displacement and Maximum volumetric strain value

Condition	Maximum displacement	Maximum volumetric strain
No stone column	24.27mm	0.305
0.5m dia. stone column	10.75mm	0.08
0.8m dia. stone column	12mm	0.25
1m dia. stone column	12.4mm	0.2

4.5.4 Settlement reduction ratio

Settlement reduction ratio is the ratio of settlement after treatment to settlement before treatment. Here all the stone columns show considerable amount of settlement reduction. From Priebe's method the settlement reduction determined for 2 d spacing is 0.4 for an area ratio of 0.2 at a friction angle of 44º. This result is similar to the result obtained from plaxis model.

Table 4.13 Settlement reduction ratio for stone columns from PLAXIS 2D

Stone column diameter	Settlement reduction ratio
0.5m diameter stone column	0.447
0.8m diameter stone column	0.49
1m diameter stone column	0.51

4.6 CONSOLIDATION ANALYSIS

Stone columns can significantly accelerate the rate of consolidation of soft clays due to the following two mechanisms: (i) High column permeability, which causes radial drainage resulting in faster dissipation of excess pore water pressure; and (ii) high column stiffness, which leads to reduced vertical stress on the soil body, thereby reducing the generation of excess pore water pressure

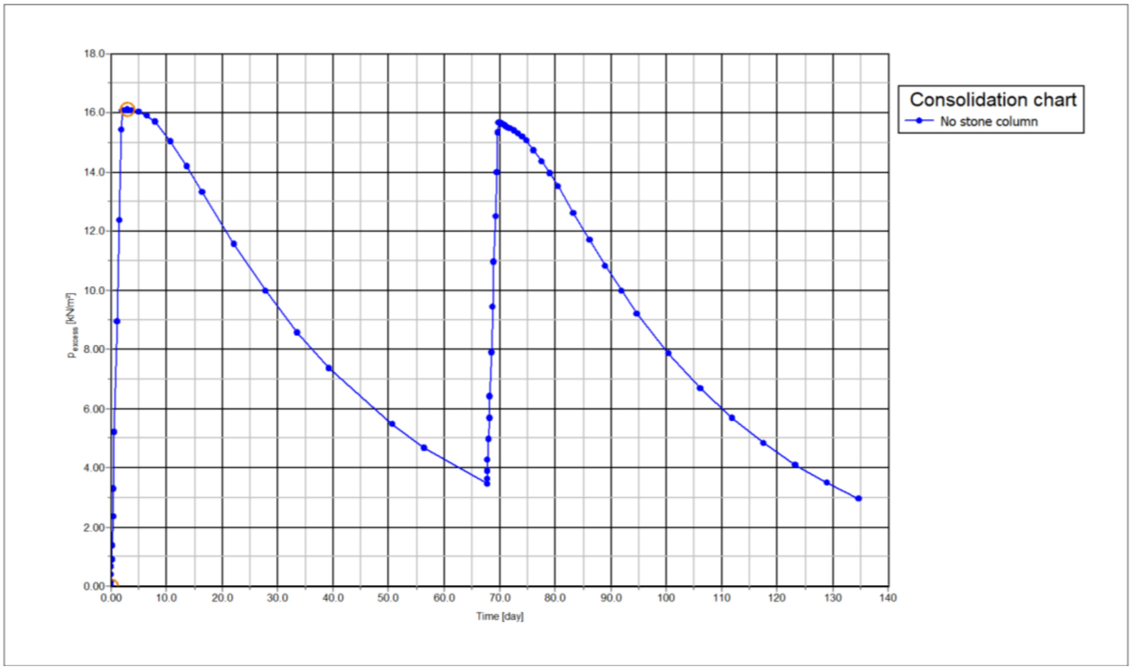
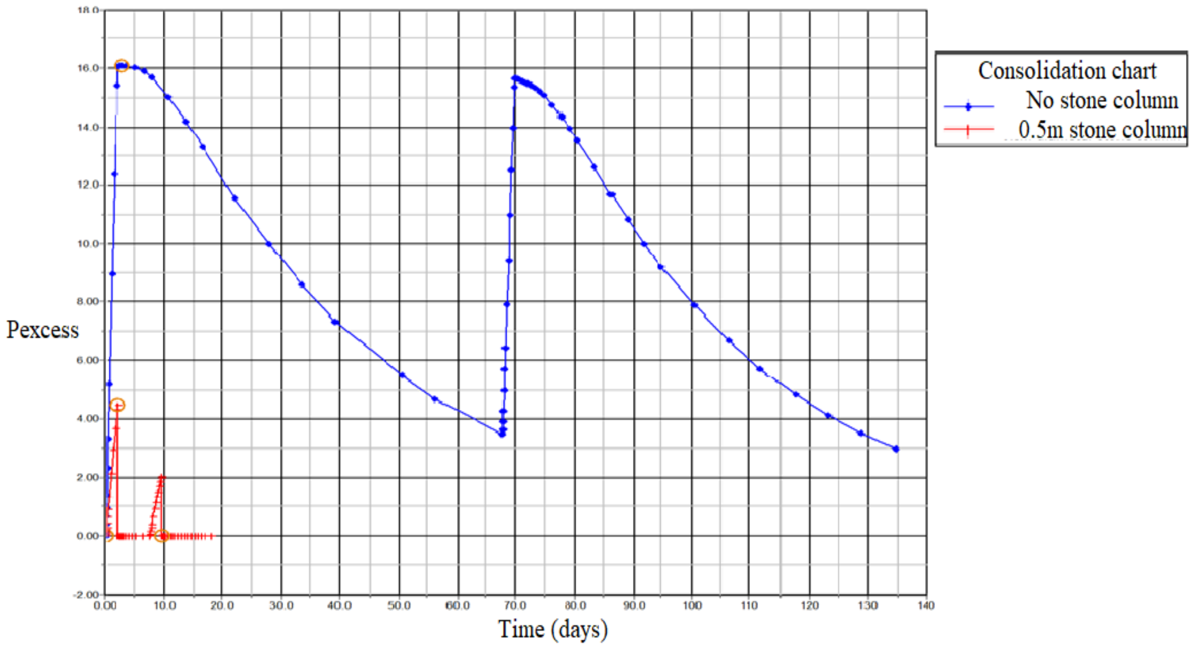


Fig. 4.9 Excess pore pressure Vs Time graph for unreinforced section

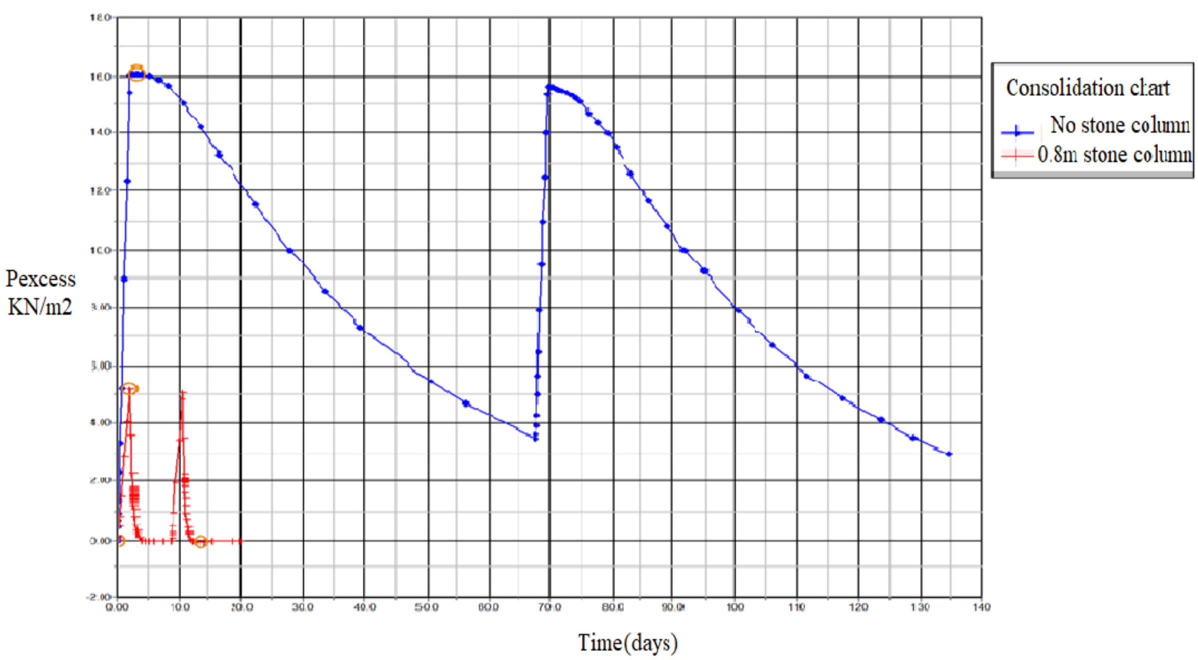
Fig. 4.9 shows the dissipation of excess pore water pressure during both construction and consolidation stages. Initially the pore pressure increases during the plastic consolidation stage, later there is gradual reduction in the pore pressure during the consolidation stages. 80% consolidation is allowed in the present study and it can be seen that it takes about 135 days to complete consolidation. Surcharge is applied to the section in the form of embankment layers to a height of 3m to achieve desired traffic load condition.

Similar to unreinforced section, soil with stone column also shows a similar trend in dissipation of excess pore pressure during consolidation stage. Initially the pore pressure increases during construction stage and reduces during consolidation stage. Stone column with 0.5m diameter shows better consolidation results with reduction in no. of days to complete consolidation from 135 days to 16 days. For all reinforced sections with stone column the days to complete consolidation is less than 20 days shown in Fig. 4.10.



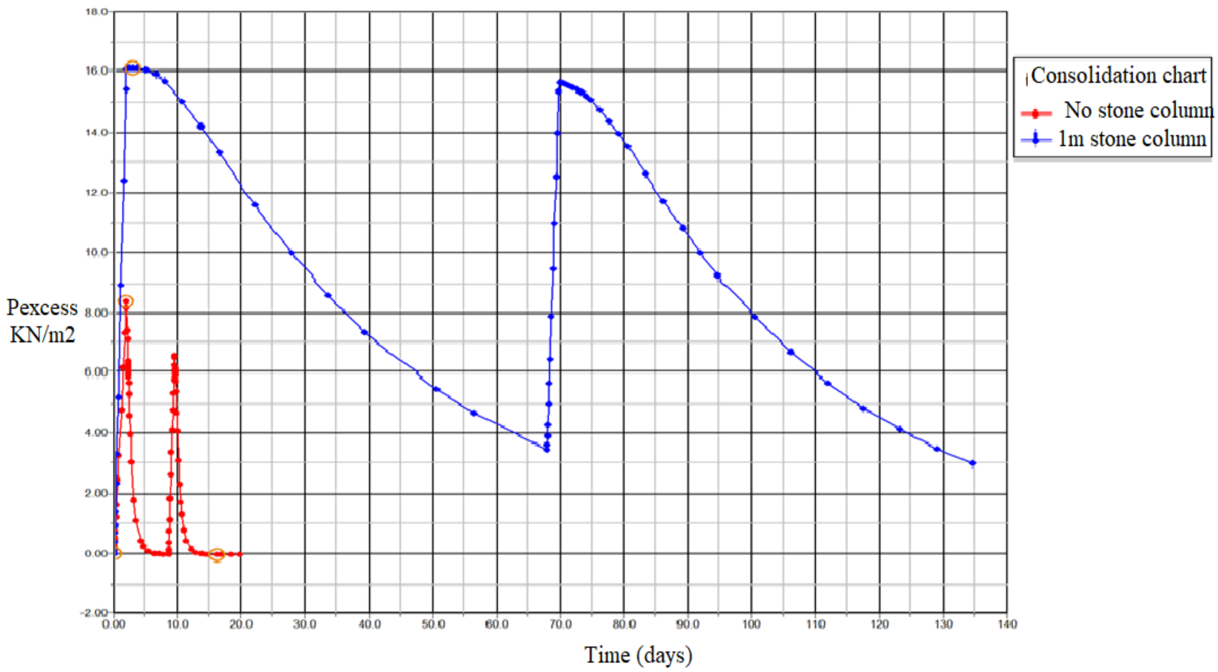
Consolidation chart
No stone column
0.5m stone column
Pexcess
Time (days)

(a)



Consolidation chart
No stone column
0.8m stone column
Pexcess
KN/m2
Time(days)

(b)



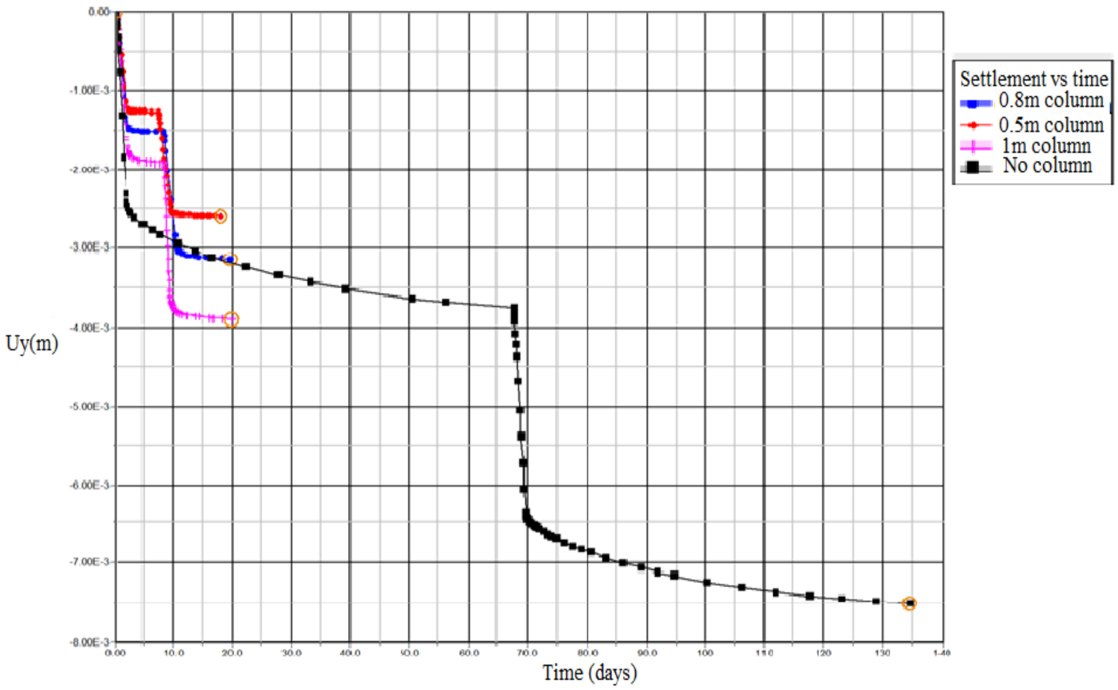
(c)

Fig. 4.10 Excess pore pressure Vs Time graph for (a) 0.5m diameter stone column (b) 0.8m diameter stone column (c) 1m diameter stone column

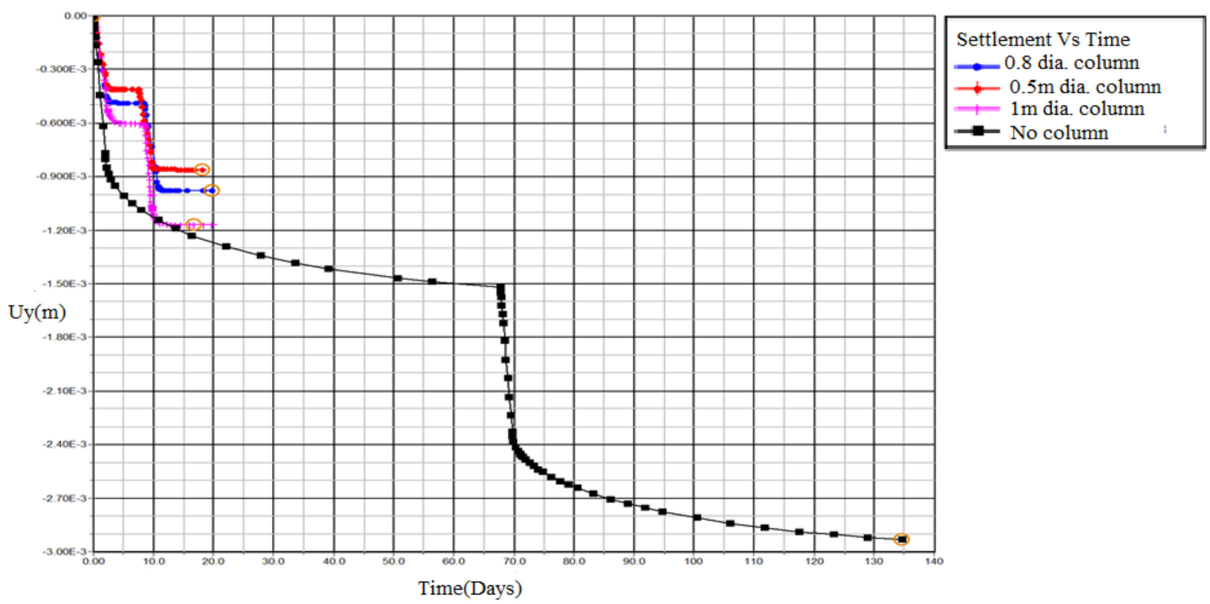
Compared to 0.8 and 1m diameter stone columns 0.5m diameter stone column shows better results in dessipating excess pressure quickly due to the lesser spacing between the columns. This provide a better drainage condition. Hence spacing of stone column is an important parameter to complete consolidation. There is clearly a large difference in maximum excess pore pressure developed in each case.

4.6.1 Settlement at surface level and subsurface level

The two main points considered for settlement analysis is one at surface level and other at subsurface level at depth of about 5m below the surface. The points considered are at the mid section of the embankment when horizontal geometry is considered. The variation in settlement values for various stone column geometries at different levels are shown in the Fig. 4.11.



(a)



(b)

Fig. 4.11 Settlement at end of 80% consolidation for different stone columns (a) at surface and (b) at subsurface levels

The graph represents the maximum settlement values for both reinforced and un reinforced conditions at end of consolidation period. At surface level the settlement values are usually higher compared to sub surface section values. The maximum settlement without stone column at surface level is about 7.5mm at end of 80% consolidation where as it is 2.8mm at mid section for same consolidation rate.

For both surface point and subsurface point the trend in settlement reduction with stone column condition is similar. Stone column with maximum displacement value is 1m diameter stone column, but the difference in settlement values obtained from other stone columns is negligible.

4.6.2 Settlement with depth

About 8m depth of soil is considered for settlement analysis with and without stone column condition. The variation in settlemnt with depth for each case is shown in Fig. 4.12 and Fig. 4.13.

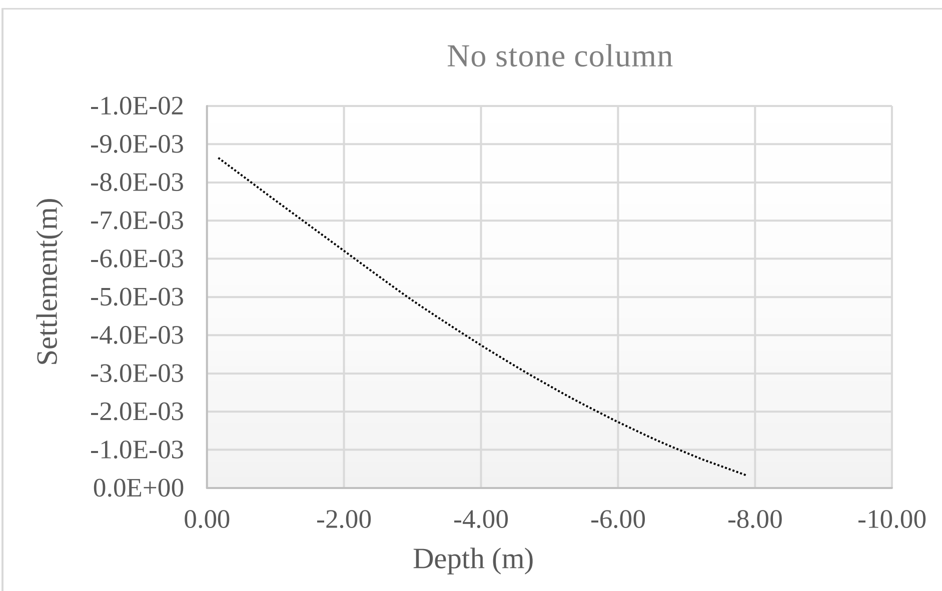
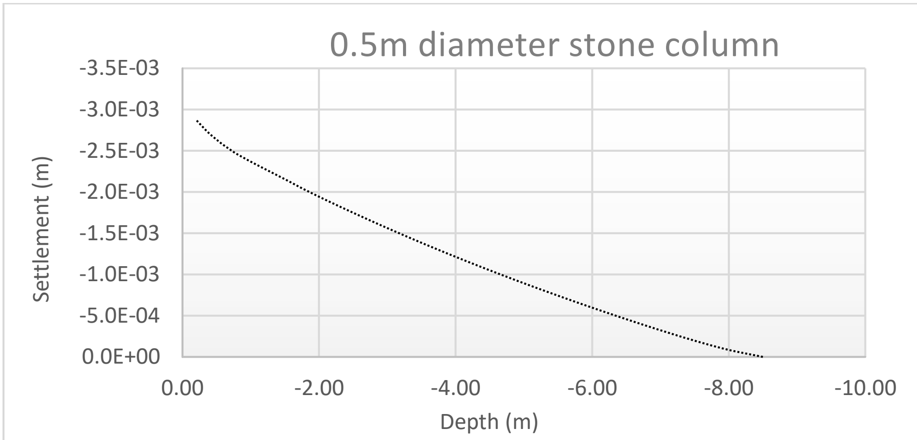
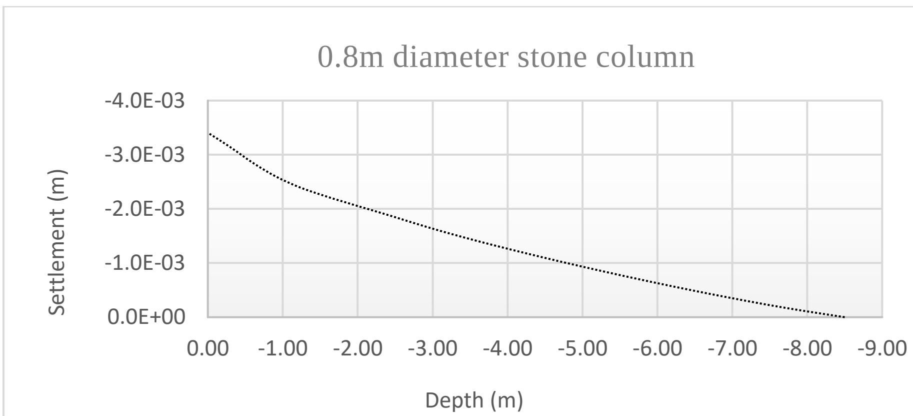


Fig. 4.12 Settlement Vs Depth for no stone column condition

Settlement of subgrade soil decreases with depth. It can be seen that upto a depth of 8m only the effect of load is prevailing, below that the settlement is negligible. So only upto

8m there is a need for stone column. Settlement of soil points at end of 80% consolidation is depicted in the following graphs.





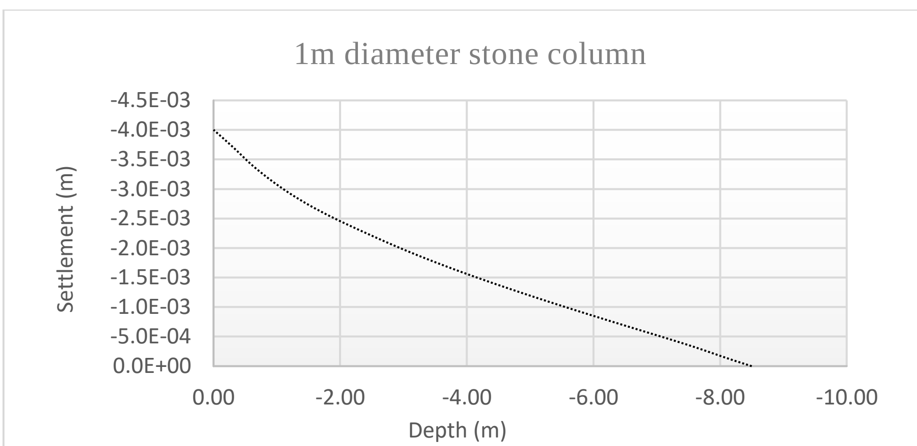


Fig. 4.13 Settlement Vs Depth for stone column models

By providing stone column it can be seen that maximum settlement reduces to nearly 50%. Maximum reduction in settlement values are seen in stone column with minimum diameter and minimum spacing. With increase in diameter the reduction in settlement value reduces. With reduction in spacing the soil remains in tightly compacted condition and therefore compression due to load can be reduced. The maximum value for settlement under different stone column conditions are shown in Table 4.14.

Table 4.14 Settlement values at surface and bottom points

Condition	Surface settlement (m)	Settlement at bottom point(m)
No stone column	-8.66E-03	-3.38E-04
0.5m diameter stone column	-2.88E-03	-3.25E-05
0.8m diameter stone column	-3.41E-03	-6.45E-05
1m diameter stone column	-4.00E-03	-1.04E-04

At surface level maximum reduction in settlement is possible when stone column of 0.5m diameter is used. A 67% reduction in settlement is possible using stone column of 0.5m diameter whereas a settlement of 53% occurs while using stone column of 1m diameter.

4.7 SUMMARY

From functional evaluation done on pavement section it was found that the condition rating is 1.6 and pavement section requires maintenance. Structural evaluation done necessitates the need for overlay due to prominent deflection determined using BBD test. Stone column provided in the subgrade at a depth of 8.5m helps in reduction of settlement and improves the bearing capacity of the soil. Using PLAXIS 2D models are created for different stone column diameters and consolidation study was done. Inclusion of stone column reduces settlement as well as reduces the consolidation time from 135 days to less than 20 days. Similarly there is about 55% reduction in settlement due to addition of stone column.

CHAPTER 5

CONCLUSIONS

5.1 GENERAL

This study focusses on the pavement evaluation of Kollam Asramam link road . Prior to the structural evaluation study, the functional evaluation of the pavement was conducted by visual examination survey and pavement condition rating was provided based on IRC:82-2015. The Structural evaluations are done to determine the structural capacity of the pavement in terms of deflection, pavement layer thickness or material properties. Here the structural evaluation of the pavement was conducted using Benkelman Beam Deflectometer(BBD). BBD determines the rebound deflection of the pavement stretch based on static load condition. From the results of BBD test the characteristic deflection of pavement is determined and necessary corrections such as moisture correction and temperature corrections are provided. This corrected value is used to determine the overlay thickness of existing pavement based on the design traffic load.

From previous studies conducted, it can be seen that excessive and prolonged consolidation of soil is one of the major issue of subgrade settlement over years. To overcome the issue of drainage and settlement, stone columns can be adopted as the main remedial measure. Priebe's method of analysis is used for the design of stone column, considering area replacement ratio and settlement improvement factor. An area replacement ratio of 0.2 is adopted in present study as it provides the best settlement improvement factor. Three stone column designs are adopted with varying diameters of 0.5, 0.8 and 1m each having a spacing of twice the diameter.

In this study, Plaxis 2D software is used for the analysis of stone columns. PLAXIS 2D is a Finite element programme used to model and deploy geotechnical problems. Here, it is used to analyze the subgrade soil and determine the suitability of stone columns. Settlement of the pavement including the pavement section and consolidation study using surcharge load is mainly focussed in this study. Validation of PLAXIS 2D software is done using the

IITPAVE software. The strain values obtained from PLAXIS 2D software are compared with strain values obtained from IITPAVE software for validation purpose.

5.2 SPECIFIC CONCLUSIONS

- From the visual examination survey it was concluded that the pavement has a PCR value of 1.6, which means it is in fair condition but close to failure condition.
- The roughness and serviceability of the pavement is poor and demands necessary maintenance procedure.
- From the Structural evaluation study done using BBD, it was concluded that the pavement shows considerable deflection (1.607mm). Hence overlay has to be provided. For a traffic load of 19 msa and characteristic deflection value of 1.607mm, the overlay thickness in terms of Dense bituminous macadam of 150mm has to be provided.
- From the results of model created using PLAXIS 2D it can be inferred that the settlement of the soil both at surface level and at subgrade level is reduced while using stone columns.
- After 80% completion of consolidation, there is a 55% reduction in settlement when using 0.5m diameter stone column. Maximum reduction in settlement is observed when using 0.5m diameter stone column.
- A considerable reduction in number of days to complete consolidation is also noticed. The time taken to complete the consolidation reduced from 135 days to less than 20 days.
- Stone column with minimum diameter and spacing i.e., 0.5m provides the best results both in case of settlement reduction and consolidation. This can be due to compacted arrangement of stone column with minimum spacing and proper drainage of excess water through stone column.

- The horizontal and vertical strain obtained from plaxis software are 0.245×10^{-3} and 0.36×10^{-3} where as the strain values obtained from IITPAVE software are 0.2538×10^{-3} and 0.399×10^{-3}. From the results it can be inferred that the strain values obtained from both the softwares are varying within a limit of 5% .

5.3 FUTURE SCOPE

Structural evaluation using BBD is carried out in the present study which is a conservative method. Better results in deflection can be obtained from other methods such as light weight falling deflectometer or FWD tests compared to BBD. Similarly, in the present study only static wheel load condition is analysed but dynamic wheel load condition can be used to provide better representation of traffic conditions while using PLAXIS 2D. Also various parametric studies can also be conducted with varying the length or spacing of stone column and variation in settlement can be analysed.

REFERENCES

1. Ahirwar, S. K., & Mandal, J. N. (2017). Finite element analysis of flexible pavement with geogrids. *Procedia engineering*, *189*, 411-416.

2. Al-Arkawazi, S. A. F. (2017). Flexible Pavement Evaluation: A Case Study. Kurdistan *Journal of Applied Research*, 2(3), 292-301.

3. Ambily, A. P., & Gandhi, S. R. (2007). Behavior of stone columns based on experimental and FEM analysis. *Journal of geotechnical and geoenvironmental engineering*, *133*(4), 405-415.

4. Avinash, N. R., Vinay, H. N., Prasad, D., Dinesh, S. V., & Dattatreya, J. K. (2014). Performance evaluation of low volume flexible pavements-a case study. In T&DI Congress 2014: Planes, Trains, and Automobiles (pp. 69-78)

5. Banerjee, S., Srivastava, M. V. K., Manna, B., & Shahu, J. T. (2022). A Novel Approach to the Design of Geogrid-Reinforced Flexible Pavements. *International Journal of Geosynthetics and Ground Engineering*, *8*(2), 1-15.

6. Banerjee, S., Srivastava, M. V. K., Manna, B., & Shahu, J. T. (2022). A Novel Approach to the Design of Geogrid-Reinforced Flexible Pavements. *International Journal of Geosynthetics and Ground Engineering*, *8*(2), 1-15.

7. Elsawy, M. B. D. (2013). Behaviour of soft ground improved by conventional and geogrid-encased stone columns, based on FEM study. *Geosynthetics International*, *20*(4), 276-285.

8. Emersleben, A., & Meyer, N. (2012). The use of vertical columns in combination with geocell stabilized load transfer platforms for the construction of roadways over soft soils. In *GeoCongress 2012: State of the Art and Practice in Geotechnical Engineering* (pp. 1302-1309).

9. Faheem, H., & Hassan, A. M. (2014). 2D PLAXIS finite element modeling of asphalt-concrete pavement reinforced with geogrid. *JES. Journal of Engineering Sciences*, *42*(6), 1336-1348.

10. Fonseca, E. C., Palmeira, E. M., & Barrantes, M. V. (2018). Load and deformation mechanisms in geosynthetic-reinforced piled embankments. *International Journal of Geosynthetics and Ground Engineering*, *4*(4), 1-12.

11. Gaber, M., Kasa, A., Abdul-Rahman, N., & Alsharef, J. (2018). Simulation of stone column ground improvement (comparison between axisymmetric and plane strain). *Am J Eng Appl Sci*, *11*(1), 129-137.

12. Guzzarlapudi, S. D., Adigopula, V. K., & Kumar, R. (2016). Comparative studies of lightweight deflectometer and Benkelman beam deflectometer in low volume roads. *Journal of Traffic and Transportation Engineering (English Edition)*, *3*(5), 438-447

13. Hamzh, A., Mohamad, H., & Bin Yusof, M. F. (2022). The effect of stone column geometry on soft soil bearing capacity. *International Journal of Geotechnical Engineering*, *16*(2), 200-210

14. Hosseinpour, I., Soriano, C., & Almeida, M. S. (2019). A comparative study for the performance of encased granular columns. *Journal of Rock Mechanics and Geotechnical Engineering*, *11*(2), 379-388.

15. Ibrahim, S. F., Ahmed, N. G., & Mohammed, D. E. (2016). Developing a Performance Criteria for Stone Columns to Improve Surface Pavement for Weak Subgrade Conditions. *Procedia engineering*, *143*, 1309-1316

16. IRC 37 (2018). Guidelines for the design of flexible pavements second revision. Indian Roads Congress, New Delhi.

17. IRC 81 (1997). Guidelines for strengthening of flexible road pavements using Benkelman Beam Deflection Technique. Indian Road Congress, New Delhi.

18. IRC 82 (2015). Code of practice for Maintenance of Bituminous Road Surfaces. Indian Road Congress, New Delhi.

19. IS 15284-1 (2003). Design and construction for ground improvement - Guidelines, Part 1: Stone columns [CED 43: Soil and Foundation Engineering].

20. Kessler, K. (2009). Use of DCP (dynamic cone penetrometer) and LWD (light weight deflectometer) for QC/QA on subgrade and aggregate base. In *Material Design, Construction, Maintenance, and Testing of Pavements: Selected Papers from the 2009 GeoHunan International Conference* (pp. 62-67)

21. Mani, K., & Nigee, K. (2013). A study on ground improvement using stone column technique. *International Journal of Innovative Research in Science, Engineering and Technology*, *2*(11), 6451-6456.

22. Masoumeh Mokhtari, Behzad Kalantari,(2012).Soft Soil Stabilization using Stone Columns-A Review", *Electronic Journal of Geotechnical Engineering*, Vol. 17, pp. 1459-1466.

23. Mohanty, P., & Samanta, M. (2015). Experimental and numerical studies on response of the stone column in layered soil. *International Journal of Geosynthetics and Ground Engineering*, *1*(3), 1-14.

24. Report on 'The DPR for the WOTK MLA 2020-21 improvements by providing DGBM and BC to Ashramam link road.

25. Sabouri, M., Khabiri, S., Asgharzadeh, S. M., & Abdollahi, S. F. (2022). Investigating the performance of geogrid reinforced unbound layer using light weight deflectometer (LWD). *International Journal of Pavement Research and Technology*, *15*(1), 173-183.

26. Sanjay, R., Tejeshwini, S., Mamatha, K. H., & Dinesh, S. V. (2022). Comparative study on structural evaluation of flexible pavement using BBD and FWD. *Materials Today: Proceedings*, *60*, 608-615.

27. Siswoyo, D. P., & Setyawan, A. (2017). The evaluation of functional performance of national roadway using three types of pavement assessments methods. *Procedia engineering*, *171*, 1435-1442.

28. Subramanyam, B., Aravind, S., & Prasanna Kumar, R. (2017). Functional and structural evaluation of a road pavement. *Int. J. Civ. Eng. Technology*, 8, 1299-305.

29. Thakur, A., Rawat, S., & Gupta, A. K. (2021). Experimental and numerical modelling of group of geosynthetic-encased stone columns. *Innovative Infrastructure Solutions*, *6*(1), 1-17.

APPENDIX A

A.1 FUNCTIONAL EVALUATION

Functional evaluation done on the pavement section includes determining the pavement condition rating and roughness of pavement section. For determining PCR value IRC:82-2015 is taken as reference and the ratings for each section is described in Table A1, A2 and A3. Roughness of pavement section is determined using MERLIN and Roughness Index value obtained is shown below.

Table A1. Final rating value of section 1

Distress type	Input (%)	Rating	Weightage	Weighted rating value
Cracking	0.2	2.1	1.0	2.1
Ravelling	0.4	2.1	0.75	1.5
Potholes	0.02	2.1	.5	1.05
Shoving	0	3	1	3
Patching	1.6	1.5	0.75	1.125
Settlement	0.2	2.1	0.75	1.5
Rutdepth	3	2.1	1	2.1
			Rating	1.7

Table A2. Rating of section 2

Distress type	Input (%)	Rating	Weightage	Weighted rating value
Cracking	0.3	2.1	1	2.1
Ravelling	0.2	2.1	0.75	1.5
Potholes	0.11	1.2	0.5	0.6
Shoving	0	3	1	3
Patching	2.7	1.2	0.75	0.825
Settlement	1	2.1	0.75	1.6
Rutdepth	15	1	1	1
			Rating	1.5

Table A3. Rating of the section 3

Distress type	Input (%)	Rating	Weightage	Weighted rating value
Cracking	2.5	1.2	1	1.2
Ravelling	0.1	2.1	0.75	1.5
Potholes	0.03	2.1	0.5	1.05
Shoving	0	3	1	3
Patching	0.6	2.1	0.75	1.57
Settlement	0	3	0.75	2.25
Rutdepth	4	2.1	1	2.1
			Rating	1.8

The roughness of the given stretch of road is measured using MERLIN. 200 observations were conducted at regular intervals to gauge the roughness of a section of road. The equipment is placed on the ground for each observation with the wheel in its normal position and the back foot, probe, and stabilizer in contact with the pavement. The location of the pointer on the chart is then noted with a cross in the relevant column, and a cross is also noted in the tally box on the chart to keep track of the total number of observations made.

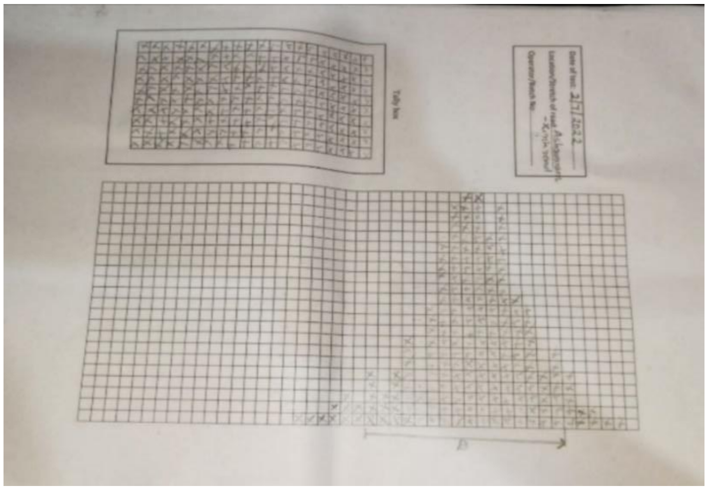


Fig. A1 Graph plotted using MERLIN

The position mid-way between tenth and eleventh crosses counted from each end of distribution is marked below the chart as D. It is measured in millimeter scale and this is the roughness on merlin scale (Fig. A1).

D=16 *5=80mm

IRI=0.593+0.0471*D

=4.361m/km

LIST OF PUBLICATIONS

1. Aswini A S, Amal Azad Sahib and Jijin A (2022). Functional and Structural Evaluation of Flexible pavement : A Review. “International Conference on Modeling and Simulation in Civil Engineering” (ICMSC), December, TKM College of Engineering, Kollam, Kerala.