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Deterioration Pattern of Flexible Pavement with the Help of Falling Weight Deflectometer

Mohd. Alam, Mohammad Nazim, Sitesh Kumar Singh

Abstract*: Maintenance and repair of the highway network system are major expenses in the state budget. For this reason, various concerned organizations are pointing out the need for developing an intelligent and efficient pavement performance model that can prioritize pavement maintenance and rehabilitation works. Such models can forecast the remaining pavement service life and pavement rehabilitation needs, and can help in the formulation of pavement maintenance and strengthening programmes which will reduce the road agency and road user costs. The flexible pavement performance or deterioration models involve the complex interaction between vehicles, environment, structure and surface of the pavement. Performance models relating to the pavement distress conditions like, cracking, ravelling, potholing, and roughness are analysed and developed by various researchers. Understanding the deterioration pattern of the flexible pavement is very important in order to take the decision for strengthening the pavement. The remaining life of the pavement depends on various factors such as Traffic, Environment and climatic conditions hence keeping in mind these factors. the thesis presents the pattern of the deterioration of remaining life of pavement. The thesis emphasis on determining the remaining life of pavement by conducting the FWD test. The FWD test is conducted on the same pavement for three time at regular interval to verify the remaining life of the flexible pavement.

***Keywords* : flexible pavements, remaining life, traffic, Equivalent Single Axle Load (ESAL), environment, Falling Weight Deflectometer.**

I. INTRODUCTION

Pavements are one of the most significant part of any highway system. Though the pavements of highways are designed for 15-20 years but are falling apart in early stages. In India particularly in northern part due to harsh climatic conditions, roads and highways the pavements are experiencing premature distresses and often deteriorate in different ways to those in the more temperate regions around the world.

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***** Correspondence Author
Mohd. Alam*, M.Tech Scholar, Department of Civil Engineering, Lingaya's Vidyapeeth, Faridabad, India. Email: mdalam.sespl@gmail.com
Mohammad Nazim, M.Tech Scholar, Department of Civil Engineering, Lingaya's Vidyapeeth, Faridabad, India. Email: mohammadnazimm@gmail.com
Dr. Sitesh Kumar Singh, Assistant Professor, Department of Civil Engineering, Lingaya's Vidyapeeth, Faridabad, India. Email: siteshlpu@gmail.com

Moreover, pavements often suffer from accelerated failures caused by inadequate quality control during construction, high axle loads and insufficient funding for maintenance. Understanding the deterioration pattern of the flexible pavement is very important in order to take the decision for strengthening the pavement. The remaining life of the pavement depends on various factors such as Traffic, Environment and climatic conditions hence keeping in mind these factors. the thesis presents the pattern of the deterioration of remaining life of pavement. The thesis emphasis on determining the remaining life of pavement by conducting the FWD test. The FWD test is conducted on the same pavement for three times at regular interval to verify the remaining life of the flexible pavement.

II. LITERATURE REVIEW

This section presents literature review of Falling Weight Deflectometer and Flexible Pavement.

[1] In this study, the FWD information and DCP results on six KDOT asphalt ventures were dissected to build up a connection between the DCP values and FWD-back calculated subgrade moduli. On the basis of this study, several conclusions can be drawn:

1. Existing relationship between the DCP values acquired from the DCP test and CBR and among CBR and subgrade moduli brought about modulus values that fluctuated generally along the project length. These relationships seemed to be temperamental for generally high CBRs or low DCP values.
2. The regression equation associating moduli back calculated from the FWD and the DCP values gives another way to deal with translating DCP results. This methodology is steady and dependable for application in asphalt structure and could upgrade the validity of the DCP as an exact, versatile, but then cheap technique for in situ testing.

[2] This paper presents a technique based on analyzing FWD data among trafficked and non-trafficked paths to decide the degradation and rutting capability of flexible pavement unbound aggregate layers in contrast with the subgrade damage.

The methodology was effectively applied to the FAA's NAPTF first round of adaptable asphalt test areas to show that the analysed HWD test information gave proof of the expanded base damage induced in the NAPTF airport pavement layers during trafficking partly due to the applied gear load wander.

Looking at the adjusted BDI and BCI values from the traffic paths and nontraffic paths by utilizing the exhibited FWD–HWD testing and data analysis methodology can be very important in recognizing the area of harm in the asphalt framework.

The system is particularly applicable to asphalts pavements with thick granular base–subbase layers for surveying the degradation and rutting damage capability of the unbound aggregate layers in contrast with the subgrade.

For thinner black-top and granular layers in road asphalts, the philosophy will in any case be pertinent; in any case, it should consider different FWD sensors, other than those distinguished in this study, that might be connected to base and subgrade damage.

[3] This paper has analyzed the prescient abilities of different flexible pavement performance models proposed by various researchers on the planet.

Yet, a large portion of these models are discovered relevant to a specific arrangement of traffic or environmental conditions, thus featuring the need of model(s) that can work in fluctuated conditions agreeably. The paper introduces a point by point review of different pavement performance models to look the job of factors related to pavement materials, environmental conditions, sort of traffic and volume of traffic, and to recognize the constraints and holes in the present learning on such models.

[4] Flexible pavement fall apart with time because of axle load redundancies henceforth assessment of such asphalts is important to propose amended thicknesses. In this paper NH-218 (Bijapur to Hubballi) is chosen for the investigation. FWD test has been conducted at the chose areas and the test outcomes are broke down utilizing KGP Back programming according to IRC 115-2014 and the design has been checked by IIT-Pave programming. Condition 6.3 and 6.5 given in IRC 37-2012 are utilized to ascertain the reasonable strains which later contrasted and the determined strains from IIT-Pave. Another thickness has been proposed dependent on the KGP back and IIT-Pave results. This paper talks about significance of KGP Back programming while at the same time landing at the overhauled thicknesses for asphalts.

From this study following conclusions are drawn

- For covering structures of adaptable asphalts FWD test results ought to be dissected utilizing KGP back with the goal that outstanding existence of the current layers can be considered for the sensible and efficient plans.
- In view of this examination, it tends to be inferred that FWD can be utilized as BT and Granular layer subgrade quality assessing device for the development and support of the asphalt.

Empirical Design Analysis

Some of the key developments since the past 50 years design/evaluation of flexible of flexible pavement considered for brief literature review include following:

1. Empirical Analyses;
2. Mechanistic- Empirical (M-E) Flexible Pavement Design Methodologies,
3. Non-Destructive Pavement Testing,

The first empirical methods for flexible pavement design date to the mid-1920s when the first soil classifications were developed. One of the first to be published was the Public Roads (PR) soil classification system **[5]**. In 1929, the California Highway Department developed a method using the California Bearing Ratio (CBR) strength test **[6]**. The CBR method related the material's CBR value to the required thickness to provide protection against sub-grade shear failure. The thickness computed was defined for the standard crushed stone used in the definition of the CBR test.

Along with this mechanistic approach, empirical elements are used when defining what value of the calculated stresses, strains and deflections result in pavement failure. The relationship between physical phenomena and pavement failure is described by empirically derived equations that compute the number of loading cycles to failure. The MEPDG is designed to update the 1993 AASHTO Guide for Design of Pavement Structures, which is primarily based on empirical observations from the AASHO Road Test **[7,8]** that began in the 1950s.

The first attempt to consider a structural response as a quantitative measure of the pavement structural capacity was measuring surface vertical deflection. A few methods were developed based on the theory of elasticity for soil mass. These methods estimated layer thickness based on a limit for surface vertical deflection. The first one published was developed by the Kansas State Highway Commission, in 1947, in which Boussinesq's equation was used and the deflection of sub-grade was limited to 2.54 mm. Later in 1953, the U.S. Navy applied Burmister's two-layer elastic theory and limited the surface deflection to 6.35 mm. Other methods were developed over the years, incorporating strength tests. More recently, resilient modulus has been used to establish relationships between the strength and deflection limits for determining thicknesses of new pavement structures and overlays [Bumister et al., 1958]. The deflection methods were most appealing to practitioners because deflection is easy to measure in the field. However, failures in pavements are caused by excessive stress and strain rather than deflection.

III. MATERIALS & METHODS/METHODOLOGY

Falling weight deflectometer survey procedure

In pavement deflection survey following procedure is adopted:

a) The Test point locations were pre-identified, wherein the points were calculated for project section, which were divided into sub-sections based on the visual pavement condition survey data & test pit data.
b) Test was not executed at approaches to existing structures and at locations where the bituminous surface had stripped off and granular layers were visible.
c) Testing procedure was adopted as per IRC: 115.
d) The command is stimulated through the laptop which raises the mass to a predetermined height and provides an impact on the pavement and produces a target load of 40 kN (+/- 4 kN). This impact produces deflection, which is automatically recorded in the system. Following procedure is repeated if suitable data is obtained.

e) For temperature correction factor of bituminous layer, pavement temperature is required therefore after every hour pavement temperature is measured.

Back Calculation of Layer Moduli (KGPBACK)

After completing the field investigating and test, data acquired through Deflection survey is normalized to 40 kN. Using normalized Data, other required parameters as given in **IRC 115** will be used in KGPBACK software, and pavement layer modulus will be obtained. The back calculated moduli of the bituminous & granular layers obtained from software analysis will be applied with following correction factors:

Pavement Temperature Correction Factor

The back-calculated modulus of bituminous layer obtained from deflection survey conducted at a temperature "T_2" °C shall be corrected to estimate the modulus corresponding to a temperature of "T_1" °C using below given equation:

$$ET_1 = \lambda x ET_2$$

Where, λ= Temperature correction factor, is given by,

$$\lambda = (1-0.238\ \ln(T_1)) / (1-0.238\ \ln(T_2))$$

Where,

ET_1= back-calculated modulus (MPa) at temperature T_1°C

ET_2= back-calculated modulus (MPa) at temperature T_2°C

Correction for Seasonal Variation

Following equations are used for seasonal corrections:

$Esub\_mon = 3.351x(Esub\_win)^{0.7688} - 28.9$

$Esub\_mon = 0.8554x(Esub\_sum) - 8.461$

$Egran\_mon=-0.0003x(Egran\_sum)^2+0.9584x(Egran\_sum-32.98$

$Egran\_mon= 10.5523x(Egran\_win)^{0.624} - 113.857$

Where,

Esub_mon = subgrade modulus in monsoon (MPa)
Esub_win= subgrade modulus in winter (MPa)
Esub_sum = subgrade modulus in summer (MPa)
Egran_mon= granular layer modulus in monsoon (MPa)
Egran_win= granular layer modulus in winter (MPa)
Egran_sum = granular layer modulus in summer (MPa)

Remaining Life Calculation

The remaining life is calculated using IITPAVE software.

Performance Models

To analyse pavement performance in terms of rutting and fatigue cracking, following performance models as per **IRC: 115-2014 [9]** will be adopted.

Fatigue in bituminous layer

$$Nf = 0.711\times10^{-4}\times(1/\varepsilon t)^{3.89}\times(1/MR)^{0.854}$$

Where,

Nf = Fatigue life in standard axle load repetitions
εt = Maximum allowable tensile strain at the bottom of bituminous layer
MR = Resilient modulus of bituminous mix, MPa

Rutting in subgrade

$$Nr = 1.41\times10^{-8}\times(1/\varepsilon v)^{4.5337}$$

Where,

Nr = Rutting life in standard axle load repetitions
εv = Maximum allowable vertical strain at the top of Subgrade layer

IV. DATA COLLECTION AND TESTING

The study area selected for project was located in State of Punjab. The study was conducted on SH-12A (S-2) for the section from Km 79.000 to Km 108.800.

Field Investigations

Traffic Study

The traffic study involves the classified traffic volume count and axle load. The Million Standard Axle has been computed from the Classified Traffic Volume Count and Axle Load Survey as shown in **Table 1.**

Table. 1: Summary of Projected MSA for S2 (SH-12A)

Location	Existing Chainage (Km)		Design MSA
	From	To	20 Years
S-2-Maur Mandi	79	108.8	52

Pavement Layer Configuration

In order to be able to model the pavement into a linear multi-layered pavement model, layer thicknesses and material types have to be known. The crust thickness of the road is shown in **Table 2 and Fig. 1.**

Table. 2: Existing Crust Thickness (Bhawanigarh-Sunam-Bathinda – SH-12A)

S. No.	Chainage	Side	BT	WBM	Bricks	TOTAL
1	82+280	RHS	190.00	105.00	80.00	375.00
2	87+200	LHS	105.00	65.00	80.00	250.00
3	92+240	RHS	155.00	80.00	80.00	315.00
4	98+000	LHS	105.00	80.00	80.00	265.00
5	103+300	RHS	190.00	80.00	80.00	350.00

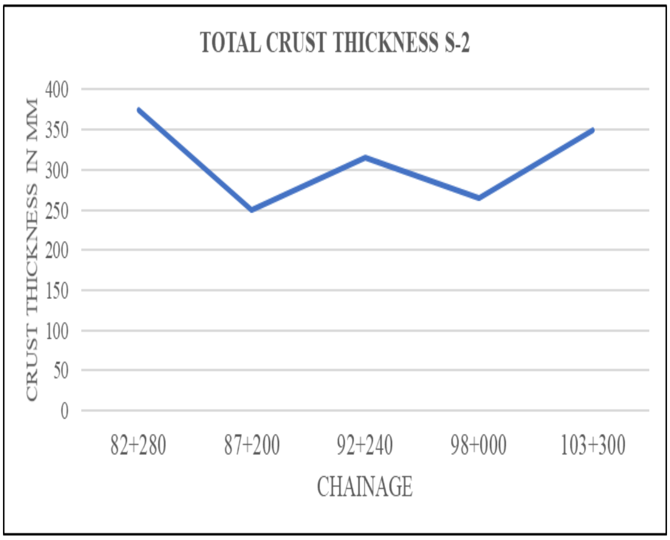

Fig. 1. Existing Pavement Layer of SH-12A

Presentation of FWD Data

Fig. 2. to Fig. 4. shows the graphical presentation of the deflections collected at regular interval.

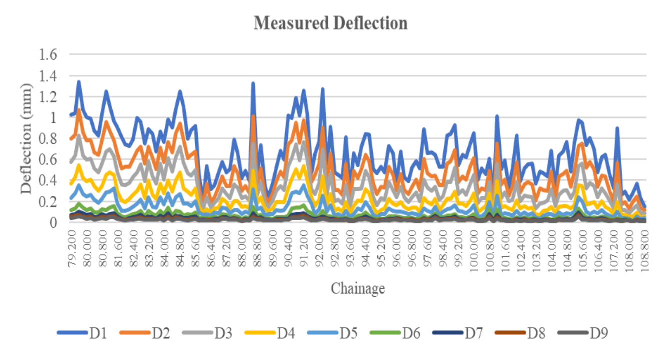

Fig. 2. Graphical Presentation of Deflection Collected for the first Time

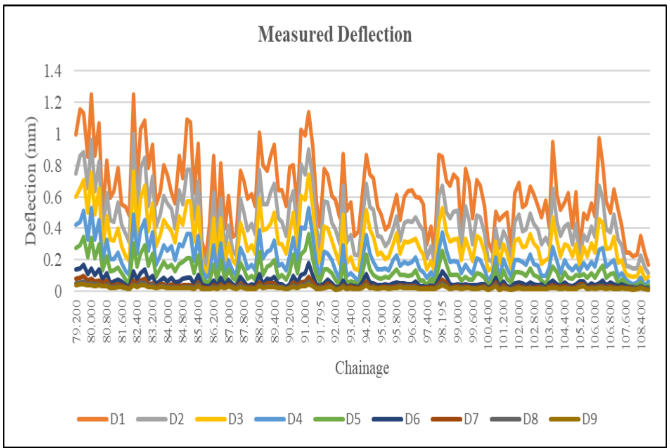

Fig. 3. Graphical Presentation of Deflection Collected for the Second Time

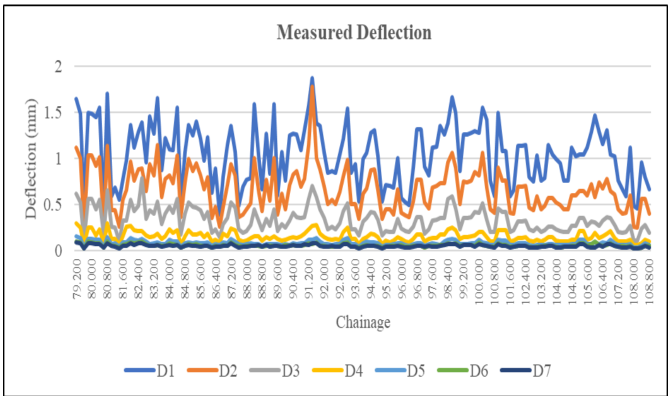

Fig. 4. Graphical Presentation of Deflection Collected for the Third Time

The collected deflection is averaged out to make 7 sections for the analysis purpose. The averaged deflection is shown in **Table. 3 to Table. 5.**

Table. 3: Summary of Average Deflection (First Collection)

Chainage (Km)		Distance from Load Centre (mm)								
		0	200	300	450	600	900	1200	1500	1800
From	To	D1	D2	D3	D4	D5	D6	D7	D8	D9
79.2	80	1.10	0.87	0.66	0.43	0.28	0.14	0.08	0.06	0.05
80	85	0.92	0.70	0.52	0.33	0.20	0.09	0.06	0.04	0.03
85	90	0.56	0.40	0.30	0.19	0.12	0.06	0.04	0.03	0.02
90	95	0.75	0.55	0.40	0.25	0.15	0.07	0.04	0.03	0.03
95	100	0.63	0.44	0.31	0.18	0.11	0.05	0.03	0.03	0.02
100	105	0.54	0.37	0.25	0.15	0.09	0.04	0.03	0.02	0.02
105	108.8	0.53	0.37	0.25	0.14	0.08	0.04	0.03	0.02	0.02

Table. 4: Summary of Average Deflection (Second Collection)

Chainage (Km)		Distance from Load Centre (mm)								
		0	200	300	450	600	900	1200	1500	1800
From	To	D1	D2	D3	D4	D5	D6	D7	D8	D9
79.2	80	1.08	0.82	0.64	0.45	0.29	0.14	0.08	0.05	0.04
80	85	0.77	0.57	0.43	0.28	0.17	0.08	0.05	0.03	0.03
85	90	0.64	0.46	0.33	0.21	0.13	0.06	0.04	0.03	0.02
90	95	0.68	0.50	0.35	0.22	0.13	0.06	0.04	0.03	0.02
95	100	0.59	0.42	0.29	0.18	0.10	0.05	0.03	0.02	0.02
100	105	0.53	0.37	0.26	0.16	0.10	0.04	0.03	0.02	0.02
105	108.8	0.44	0.31	0.22	0.13	0.08	0.04	0.03	0.02	0.02

Table. 5: Summary of Average Deflection (Third Collection)

Chainage (Km)		Distance from Load Centre (mm)						
		0	200	500	900	1400	1900	2400
From	To	D1	D2	D3	D4	D5	D6	D7
79.2	80	1.33	0.89	0.47	0.22	0.12	0.08	0.07
80	85	1.12	0.75	0.41	0.18	0.09	0.06	0.06
85	90	0.98	0.63	0.32	0.15	0.07	0.05	0.05
90	95	1.13	0.75	0.36	0.16	0.08	0.06	0.05
95	100	1.07	0.66	0.32	0.15	0.08	0.06	0.05
100	105	0.98	0.58	0.28	0.13	0.07	0.05	0.04
105	108.8	0.96	0.55	0.26	0.13	0.07	0.06	0.05

Back Calculation of Layer Moduli (KGPBACK)

Using normalized Data, other required parameters as given in **Table. 6** are used in KGPBACK software, and pavement layer modulus is obtained.

Table. 6: Input Parameters for KGPBACK Software

Parameters	Values	
Single Wheel Load (N)	40000	
Contact Pressure (Mpa)	0.56 (As per IRC: 115 and IRC 37)	
Number of deflections measuring	7 or 9	
Radial distance between each geophones (mm)	0 200 500 900 1400 1900 2400 0 200 300 450 600 900 1200 1500 1800	
Measured Deflections (mm)	Normalized defections obtained after normalization of field data	
Pavement Layer Thickness	As per Table 5.5	
Poisson's ratio values	0.5 0.4 0.4 (bituminous layer, granular layers & subgrade as per IRC: 115-2014)	
Moduli range (as per IRC:115 2014 guidelines)	BT Layer	750 to 3000
	Granular layers	100 to 500
	Subgrade	As per

The sample input and output of the KGPBACK is shown in **Fig. 5. and Fig. 6.**

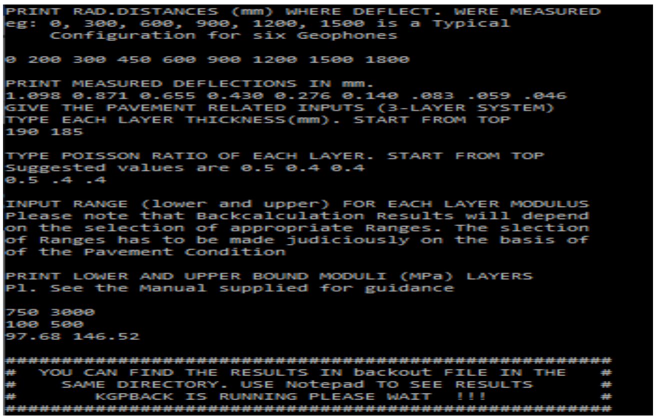

Fig. 5. Input window of the KGPBACK

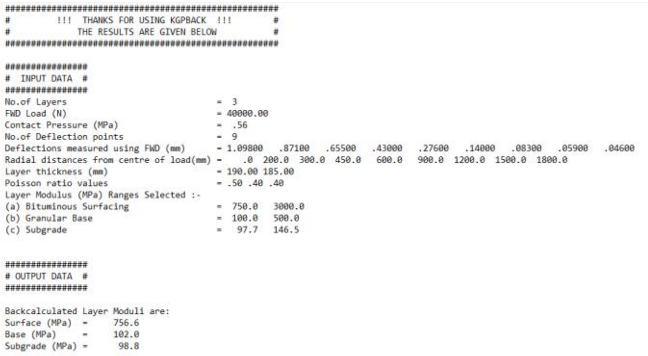

Fig. 6. Output window of the KGPBACK

Using the inputs given in **Table. 6** the back calculated Moduli of each layer is calculated and presented in **Table. 7 to Table. 9** and in **Fig. 7. to Fig. 9.**

Table. 7: Back Calculated Moduli for the First Collection

Chainage	Back Calculated Moduli (MPa)		
	Bituminous Layer	Granular Base	Subgrade
79.200-80.000	757	102	99
80.000-85.000	761	100	146
85.000-90.000	1740	100	217
90.000-95.000	763	100	190
95.000-100.000	1258	100	23
100.000-105.000	763	102	280
105.000-108.800	781	100	247

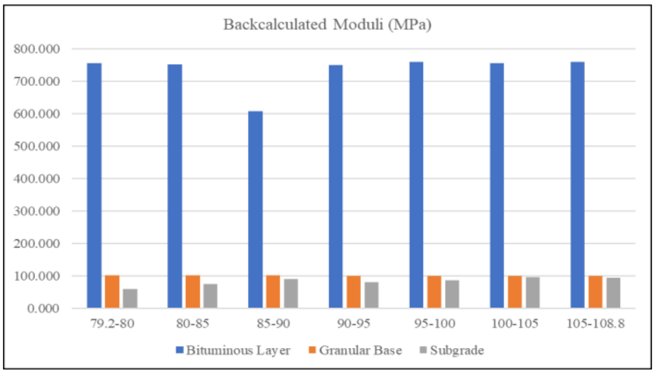

Fig.7. Back Calculated Moduli of the First Collection

Table. 8: Back Calculated Moduli for the Second Collection

Chainage	Back Calculated Moduli (MPa)		
	Bituminous	Granular	Subgrade
79.200-80.000	2989	496	156
80.000-85.000	750	100	175
85.000-90.000	1203	100	229
90.000-95.000	761	101	212
95.000-100.000	1177	100	252
100.000-105.000	763	102	294
105.000-108.800	750	101	287

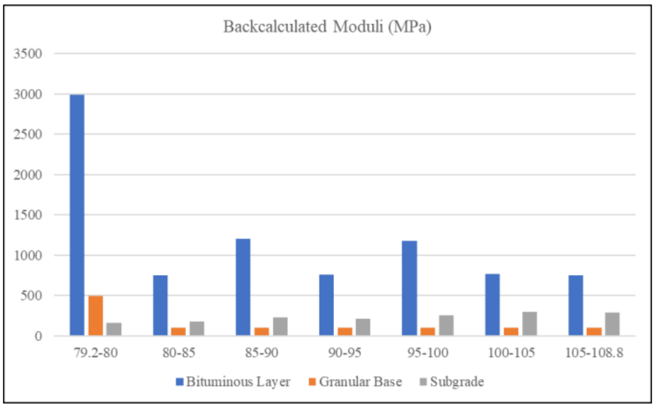

Fig. 8. Back Calculated Moduli of the Second Collection

Table. 9: Back Calculated Moduli for the Third Collection

Chainage	Back Calculated Moduli (MPa)		
	Bituminous Layer	Granular Base	Subgrade
79.200-80.000	757	102	60
80.000-85.000	752	102	74
85.000-90.000	608	102	91
90.000-95.000	750	100	81
95.000-100.000	761	100	86
100.000-105.000	757	100	97
105.000-108.800	761	100	95

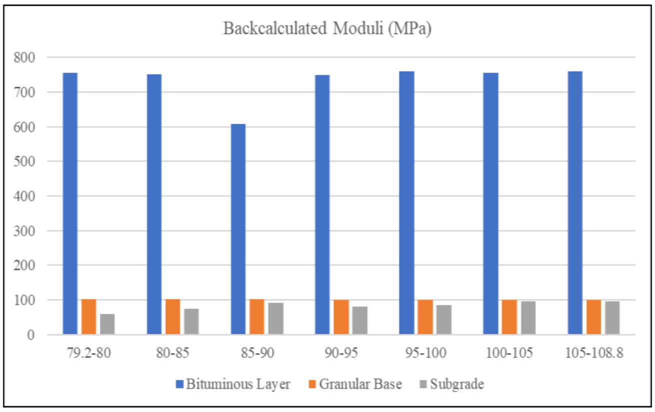

Fig.9. Back Calculated Moduli of the Third Collection

The back calculated moduli of the bituminous & granular layers obtained from software analysis were applied with following correction factors.

- Pavement Temperature Correction Factor
- Correction for Seasonal Variation

The corrected moduli are given in **Table. 10 to Table. 12** and **Fig.10. to Fig. 12.**

Table. 10: Corrected Back Calculated Moduli for the First Collection

Chainage	Corrected Back Calculated Moduli (MPa)		
	Bituminous Layer	Granular Base	Subgrade
79.200-80.000	757	75	86
80.000-85.000	761	73	126
85.000-90.000	1740	73	181
90.000-95.000	763	73	161
95.000-100.000	1258	73	8
100.000-105.000	763	75	226
105.000-108.800	781	73	202

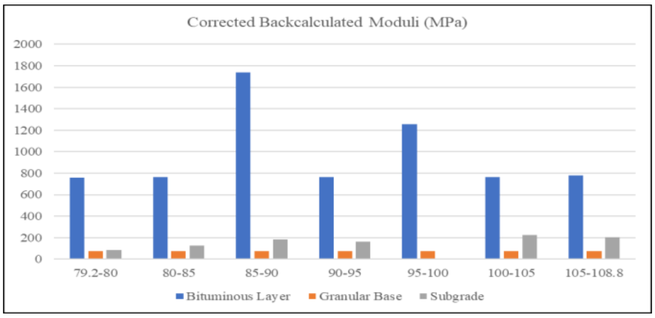

Fig.10. Corrected Back Calculated Moduli of the First Collection

Table. 11: Corrected Back Calculated Moduli for the Second Collection

Chainage	Corrected Back Calculated Moduli (MPa)		
	Bituminous Layer	Granular Base	Subgrade
79.200-80.000	2795	394	134
80.000-85.000	701	73	149
85.000-90.000	1125	73	189
90.000-95.000	712	74	177
95.000-100.000	1100	73	206
100.000-105.000	714	75	236
105.000-108.800	701	74	231

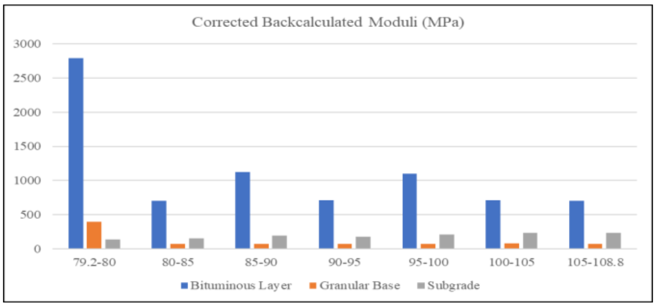

Fig. 11. Corrected Back Calculated Moduli of the Second Collection

Table. 12: Corrected Back Calculated Moduli for the Third Collection

Chainage	Corrected Backcalculated Moduli (MPa)		
	Bituminous Layer	Granular Base	Subgrade
79.200-80.000	732	75	49
80.000-85.000	728	75	63
85.000-90.000	588	75	79
90.000-95.000	726	73	69
95.000-100.000	736	73	74
100.000-105.000	732	73	84
105.000-108.800	736	73	82

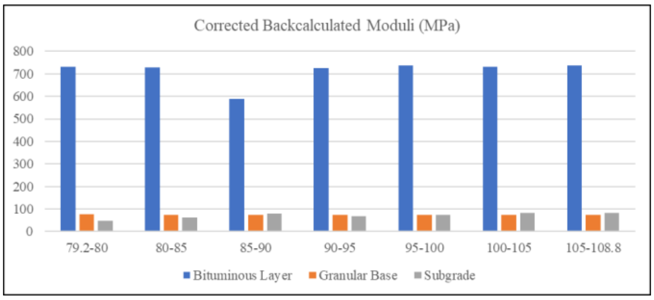

Fig. 12. Corrected Back Calculated Moduli of the Second Collection

As per IRC 115, 15th percentile modulus (15% of the values will be less than this value) of each of the three layers should be considered for analysis of the remaining life. The 15th percentile modulus of each of the three layer is given in **Table. 13** and **Fig. 13.** shows the variation of the moduli in Bituminous Layer.

Table. 13: 15th Percentile Modulus

	First Time Collection	Second Time Collection	Third Time Collection
15th Percentile Moduli of	757	701	615
15th Percentile Moduli of	73	73	73
15th Percentile Moduli of	23	137	52

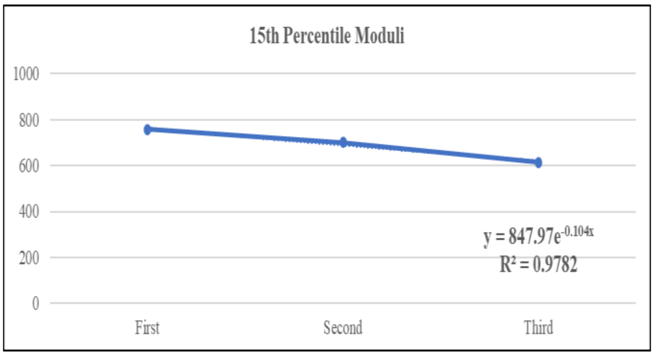

Fig.13. 15th Percentile Moduli

Remaining Life Calculation

The structural analysis of the FWD deflection data is used to determine the design life of the pavement structures to withstand the predicted traffic load. It should however be kept in mind that these residual life estimates are essentially governed by the mechanistic characteristics of the pavement materials and the predicted traffic load. It is now a days becoming standard practice to use mechanistic-empirical methods for design and evaluation of pavements. In such an analytical approach the critical locations in a pavement structure are:

- ➢ The horizontal strains at the bottom of a bituminous layer
- ➢ The vertical strain (deformation) at the top of unbound base/subbase layers and the subgrade.

For calculating the remaining life, the horizontal strains and vertical strains is calculated by the IITPAVE. The remaining life is calculated using IITPAVE software using following input parameters as given in **Table. 14.**

Table. 14: Input Parameters for IITPAVE

Parameters	Typical Values Adopted
Number of layers (n)	3
Elastic Modules (E), in MPa	As per FWD Back-Calculation
Poisson's Ratio (µ)	0.5, 0.4, 0.4
Thickness of Layers (h), mm	161mm and 165mm (Average of the
Dual wheel load (N), Tyre	20000, 0.56

The remaining life of pavement in terms of fatigue life and rutting life using the above performance models are tabulated in **Table. 15. Fig.14.** shows the variation in remaining life of the pavement over the period of time.

Table 4.15 Calculated Remaining Life of Pavement

	Remaining Life (msa)	
	Fatigue	**Rutting**
First Collection	1.09	1.12
Second Collection	0.98	0.99
Third Collection	0.82	0.81

The minimum of the Fatigue life and Rut life is remaining life of the pavement.

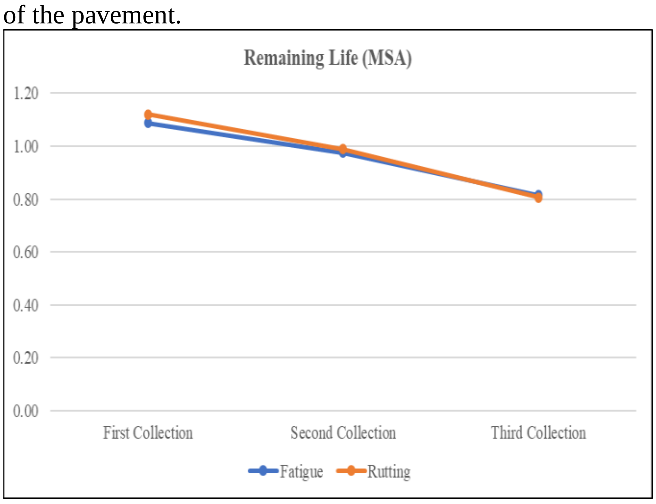

Fig.14. Variation in Remaining Life

V. RESULTS AND DISCUSSIONS

General

From the previous chapter, analysis and discussions, it is seen that the objectives of this study are achieved. The pattern is very clear in terms of the Moduli and Remaining Life

Result of Data Analysis

The main objectives of this research study were to review the deterioration pattern of the flexible pavement. The same objective is achieved and can be seen in **Fig. 15. and Fig. 16. Fig.15.** shows the decline in the Moduli value of the flexible pavement with time. However, **Fig. 16.** shows the same pattern of declination in Remaining life of the pavement.

It is also clear from the analysis that the strength of the pavement is directly associated with the Moduli value. The pattern in the deterioration is also shown in terms of exponential equation with R^2 value of 0.9782. The deterioration equation is as follows.

$$y = 847.97e^{-0.104x}$$

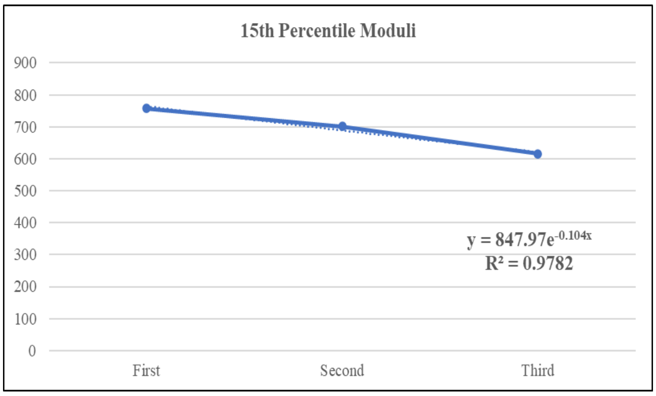

Fig. 15. Declination of the Moduli Value

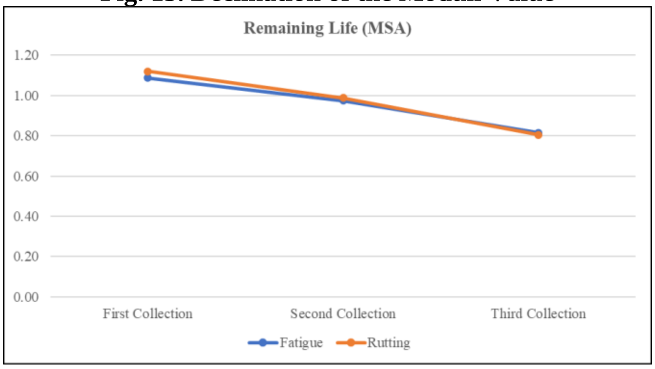

Fig. 16. Declination of Remaining Life

VI. RECOMMENDATIONS, LIMITATIONS AND FUTURE SCOPE

Recommendations

- ➢ The maintenance work on the pavement should be done properly with time.
- ➢ The deflection test should be carried out at regular interval in order to design the proper overlay.
- ➢ The data base of the pavement performance with time should be prepared which will be used in developing the more realistic performance model.

- The Moduli of the pavement is deteriorated with time. The deterioration of the Moduli gives the declination in the remaining Life.
- The deterioration in the remaining life depends on various factor such as traffic, material quality, climate of the area etc.

Limitations of Research

- There are limited data in the country on the field performance of such type of construction to understand and model their performance in the field.
- For developing the better performance model, a bunch of data of different roads with different condition is required.

Future Scope

- The present study is made on one road and the pattern of deterioration is determined on basis of the analysis. However, the more realistic pattern of deterioration can be developed by studying the different road.
- The study can also be elaborated by varying the various factors such as temperature and CBR value of the subgrade.

VII. SUMMARY AND CONCLUSIONS

The main objectives of this research study were to review the deterioration pattern of the flexible pavement. The same objective is achieved. The decline in the Moduli value of the flexible pavement with time shows the deterioration of the pavement. As per IRC 37 the pavement is designed for the numbers of standard axle. So, with the passage of the number of standard axles the life of the pavement decreases. In order to keep the pavement strong enough to take the traffic load it is essential to carried out the deflection test regularly on the pavement and accordingly the overlay should be laid over the pavement. The study can be concluded with the following points

- The data base of the pavement performance with time should be prepared which will be used in developing the more realistic performance model.
- The Moduli of the pavement is deteriorated with time. The deterioration of the Moduli gives the declination in the remaining Life.
- The deterioration in the remaining life depends on various factor such as traffic, material quality, climate of the area etc.
- For developing the better performance model, a bunch of data of different roads with different condition is required.
- It is also clear from the analysis that the strength of the pavement is directly associated with the Moduli value

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AUTHORS PROFILE

Mohd. Alam, Civil Engineer, M.Tech Scholar, Department Of Civil Engineering Lingaya's Vidyapeeth Faridabad – 121002, Haryana (India)

Mohammad Nazim, Civil Engineer, M.Tech Scholar, Department Of Civil Engineering Lingaya's Vidyapeeth Faridabad – 121002, Haryana (India)

Dr. Sitesh Kumar Singh, C.Eng.(I) is a Consultant, Chartered Engineer, an Author, Assistant Professor. He is also the member of Professional Societies. He has authored 02 books, has presented 10 papers in various national & international conferences, published 27 papers in various national & international journals of repute and has 02 patents registered.