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Structural Evaluation And Overlay Design Of Flexible Pavement Using Falling Weight Deflectometer Test

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Abstract: The structural evaluation and overlay design of flexible pavements are essential for ensuring road performance and prolonging pavement life. The FWD test is a widely adopted, non-destructive method for assessing the structural capacity of flexible pavements by replicating real-world traffic loading conditions. This study outlines a comprehensive approach for evaluating flexible pavements using FWD test and developing effective overlay designs to restore their structural integrity and serviceability. Pavement surface deflections caused by impulse loading are used to back-calculate layer moduli and identify structural weaknesses. The overlay design process is grounded in mechanistic-empirical principles, using the computed moduli and critical strain responses to determine the optimal overlay thickness.

The FWD test was developed to address the limitations of the BBD method. Compared to the BBD, the FWD offers several advantages, including faster data collection, greater accuracy, and the ability to simulate dynamic traffic loading conditions. Unlike the BBD test, which uses static loading and manual measurements, the FWD replicates real-time traffic loads, resulting in more reliable and comprehensive evaluations of pavement performance.

Index Terms – FWD-Falling Wight Deflectometer, BBD-Benkelman Beam Deflectometer, Back-Calculation, Layer Moduli.

I. INTRODUCTION

Flexible pavements play a crucial role in road networks worldwide due to their affordability, adaptability, and ease of construction and maintenance. However, these pavements deteriorate over time due to traffic loads, environmental conditions, and material wear. Consequently, preserving their structural integrity and functionality is a key focus of pavement management. Accurately evaluating pavement conditions is essential for implementing effective maintenance and rehabilitation strategies to extend their lifespan. The Falling Weight Deflectometer test is a widely used non-destructive testing device that assesses the structural capacity of flexible pavements. By simulating dynamic loading similar to vehicle movement, the FWD measures surface deflections, which are then used to estimate the stiffness of individual pavement layers. This information helps engineers determine the structural condition of pavements and identify areas needing rehabilitation. Overlay design is a vital aspect of pavement maintenance, aimed at restoring strength and enhancing ride quality. Traditional methods often rely on empirical approaches that may not fully account for existing pavement conditions. In contrast, FWD-based evaluation offers a mechanistic approach, enabling the design of more effective and durable overlays. This study focuses on using the FWD test for structural assessment and overlay design of flexible pavements. The key objectives include evaluating pavement performance, identifying critical pavement responses, and optimizing overlay thickness for improved durability and functionality. The research presents a comprehensive approach integrating FWD data collection, back-calculation of pavement moduli, and mechanistic-empirical overlay design.

A section of NH-38 in Tamil Nadu has been selected for this study to assess the structural condition and strength of the pavement through the Falling Weight Deflectometer (FWD) test. The chosen stretch spans 30 km and consists of a 4-lane divided carriageway with flexible pavement throughout its entire length. It serves as a vital transport network for the state of Tamil Nadu. And the pavement condition along the considered test stretch is observed to be fair except at few locations, it exhibits some distresses like raveling, narrow cracking and minor rutting.

II. OBJECTIVE OF STUDY

The primary objective of the study is to Evaluate structural capacity of the road pavement to withstand for the present traffic loading conditions, assessment and design of Overlay along the road stretch.

a. **Structural Condition of Existing Pavement:** Evaluate the existing structural capacity of flexible pavement through the FWD test. And to determine the pavement's ability to withstand current and projected traffic loads.
b. **Analyze Pavement Deflections and Layer Moduli:** To Measure pavement surface deflections at various locations to assess the mechanical response of the pavement structure. Back-calculate layer moduli from FWD data to estimate the stiffness and integrity of each pavement layer.
c. **Identifying Structural Deficiencies and Weak Zones:** Detecting weak or deteriorated areas that require rehabilitation or strengthening. Map the variation of pavement strength across the study area for targeted interventions.
d. **Developing an Overlay Design for Pavement Rehabilitation:** By Utilizing collected FWD data and Design traffic loading, calculated the required overlay thickness for improving pavement performance and to enhance structural capacity and extend pavement life.

III. SURVEYS AND INVESTIGATIONS

3.1. Road Inventory survey: The road network inventory along the project corridor indicates that the project road is a four-lane divided carriageway, with each carriageway comprising an 8.75 m wide flexible pavement, separated by a 4.5 m wide central median. Flexible pavement is provided throughout the corridor, except at the toll plaza where rigid pavement has been constructed. The existing road condition is observed to be fair, exhibiting few minor distress such as cracking, ravelling, and occasional patching.

3.2. Pavement Composition Survey: The composition of the existing pavement crust was determined from test pit surveys. Test pits measuring 1 m x 0.6 m x 0.6 m were excavated along the roadside at 5 km intervals in a staggered pattern. A total of seven pits were dug along the stretch, and the pavement composition was recorded for each layer. The ***Table 1*** shows the crust thicknesses collected from the Test pit survey.

Table 1: Pavement Composition of Existing Pavement along Project Road

S No	Sample No	Chainage (km)	Side	BT (mm)	WMM (mm)	N.GSB (mm)	Total (mm)
1	TP-1	1+550	LHS	160	230	280	670
2	TP-2	5+600	RHS	150	280	320	750
3	TP-3	10+550	LHS	170	210	290	670
4	TP-4	15+650	RHS	240	220	270	730
5	TP-5	20+070	LHS	200	220	320	740
6	TP-6	25+200	RHS	180	270	270	720
7	TP-7	30+000	LHS	200	200	300	700

3.3. Axle Load Survey: The axle loads were recorded using portable axle pads over a 24-hour period on a random sampling basis at km 21+000 near Toll Plaza. Data collection was carried out simultaneously on both sides to ensure uniformity in sampling. The data collected from the axle load survey was compiled and analyzed using the “fourth power” pavement damage law stated in IRC 37 guidelines to determine the Vehicle Damage Factor (VDF), as presented below.

$$\text{VDF} = \left(\frac{\Sigma Equivalent\ .load\ no.\ axles\ in\ the\ load\ group}{No.\ of\ Vehicles\ carrying\ the\ Axle\ group} \right)$$

The graphical representation of the Vehicle Damage Factor (VDF) distribution for each direction is shown in ***Figure 1*** below

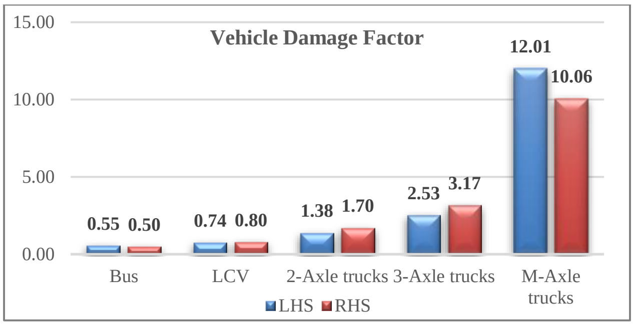


Figure 1: Vehicle Damage Factors

3.4. Average Annual Daily Traffic (AADT): The AADT data along the road stretch has been collected from respective Toll Plaza authorities and the same presented in the ***Table 2*** below.

Table 2: Average Annual Daily Traffic

Vehicle Type	No. of Vehicles		
	BHS	**LHS**	**RHS**
Cars	9072	4536	4536
Bus	1290	645	645
2-Axle	1040	520	520
3-Axle	418	209	209
M-Axle	1082	541	541
LCV	760	380	380
Total	**4590**	**2295**	**2295**

Note: 1. Passenger car count not considered in the analysis
2. 50% of vehicle count considered in each direction

3.5. Growth Rates (GR): For estimating the cumulative traffic expected to use the pavement over the design period, generally 10 years for National Highways, it is necessary to consider the rate at which the commercial traffic will grow over the design period. And hence minimum 5% Annual Growth rates were considered in the analysis. The growth rates for commercial traffic using the project road, collected from the respective authorities, are presented in ***Table 3*** below:

Table 3: Considered Traffic Growth rates

Design Period	Year	Bus	2-Axle trucks	3-Axle trucks	M-Axle trucks	LCV
1	2025	5.0%	5.0%	5.0%	5.0%	5.0%
2	2026	5.0%	5.0%	5.0%	5.0%	5.0%
3	2027	5.0%	5.0%	5.0%	5.0%	5.0%
4	2028	5.0%	5.0%	5.0%	5.0%	5.0%
5	2029	5.0%	5.0%	5.0%	5.0%	5.0%
6	2030	5.0%	5.0%	5.0%	5.0%	5.0%
7	2031	5.0%	5.0%	5.0%	5.0%	5.0%
8	2032	5.0%	5.0%	5.0%	5.0%	5.0%
9	2033	5.0%	5.0%	5.0%	5.0%	5.0%
10	2034	5.0%	5.0%	5.0%	5.0%	5.0%

IV. DESIGN TRAFFIC LOADING

The design traffic, in terms of the cumulative number of standard axles (80 KN single axle with dual wheels) to be carried during the design period of the road has been computed using the equations in clause 4.6.1 of IRC 37-2018 as below:

$$N_{Des} = \frac{365 \times [(1+r)^n - 1]}{r} \times \mathrm{A} \times \mathrm{D} \times \mathrm{F}$$

where,

A – Annual average daily traffic (AADT)

D – Lateral distribution factor

F – Vehicle damage factor (VDF)

n – Design period, in years

r – Annual growth rate

Since our corridor is a 4-lane divided carriageway, a lateral distribution factor of 0.75 has been applied. This indicates that 75% of the total traffic in each direction utilizes both lanes of the road. By using all the above, the Design traffic loading for 10, 15 and 20 years has been calculated and presented as below in ***Table 4***:

Table 4: Design Traffic Loading In MSA

Design Period	UP Direction	DN Direction
10 Years	31	28

V. WORKING PRINCIPLE OF FALLING WEIGHT DEFLECTOMETER

FWD is a dynamic testing apparatus that delivers a sudden load to a pavement and records the resulting surface deformation profile. In a standard FWD setup (see figure below), deflections D_0, D_1, D_2, etc., are captured at progressively increasing radii from the center of the loading plate. The load itself is generated by dropping a known mass onto a stack of springs mounted atop a circular plate. As the load pulses through the plate into the pavement, an array of displacement transducers positioned at set distances from the plate center tracks the transient deflection response of the pavement surface. These transducers, typically geophones or seismometers, convert pavement movement into electrical signals. Both load and deflection readings are recorded by a data acquisition system.

The FWD setup employs a circular loading plate between 300 mm and 450 mm in diameter; in the current survey, a 300 mm plate is used. A mass ranging from 50 kg to 350 kg is released from a adequate drop height to generate load pulses with the desired peak magnitude and duration. For bituminous pavements, the target peak load is 40 KN (± 4 KN), which simulates the force exerted by one dual - wheel set of an 80 KN standard axle load.

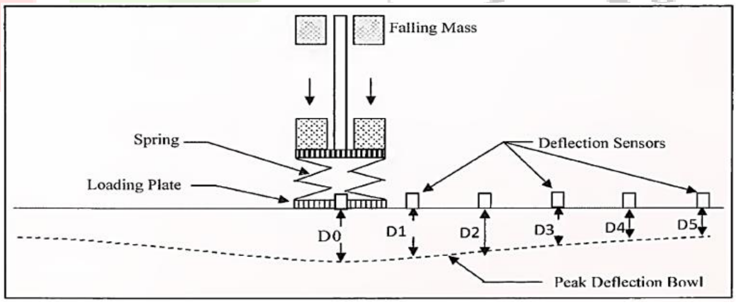


Figure 2: Working Principle of FWD measurement

VI. DEFLECTION MEASUREMENT

As stated above the pavement responses in the form of deflections were collected through the FWD testing machine by ensuring a peak load of 40KN including pavement temperature and other relevant details. Before beginning the survey, load repeatability tests are conducted daily. Before start of the test location has marked in the field. Then the loading plate and deflection sensors are positioned in positions. The impulse load of 40KN has been dropped from a pre-defined fall of height to create the required load intensity. Then the deflection data has been recorded through the geophones placed at a distance of 0, 200, 300, 450, 600, 900 and 1200mm. The measurements were done trice at each location. The first load serves as a seating load and is excluded from analysis.

Air and pavement temperatures were recorded at hourly intervals. Pavement temperature was periodically measured by drilling 40mm deep holes into the surface layer. Deflection measurements were conducted only when the pavement temperature is within the range of 20 to 45°C. Data has been collected at six points per lane for every kilometer, following a zig-zag pattern to ensure uniformity, i.e., 0,300,600 in Inner Lane and 150,450,850 in outer lane.

Figure 3: Deflection and Temperature measurements

VII. DEFLECTION DATA ANALYSIS

7.1. Normalization of Deflection data: The deflection responses collected under varying load conditions were normalized to the standard load of 40 KN.

$$R_c = \frac{R_a \times P_{std}}{P_a}$$

where,

R_c – Corrected deflection for 40KN std load

R_a – Actual collected deflection at P_a load

P_{std} – Standard Load of 40KN

P_a – Actual applied load in KN

7.2. Back-Calculation of Layer Moduli: By considering the existing condition of the pavement as Fair, the upper and lower limit of resilient modulus are considered as given in ***Table 5:***

Table 5: Considered moduli ranges

Layer	Bituminous Layers	Granular Layer	Subgrade
Modulus Value (MPa)	750-3000	100-500	50-100

Back-calculation is a process used to adjust assumed modulus of resilience (MR) values based on the measured deflections along the road. This process follows an iterative approach, continuously refining the assumed moduli until the calculated deflection basin closely aligns with the measured deflection basin within an acceptable tolerance. For this analysis, the KGPBACK tool, a Genetic Algorithm-based software was utilized.

7.3. Correction for Temperature: The stiffness of the bituminous layer is highly sensitive to temperature, which leads to variations in pavement surface deflections depending on the temperature of the bituminous material. When the bituminous surface layer is thicker than 40 mm, a correction factor must be applied. Since the deflection measurements were recorded at temperatures at or below 35°C, adjustments have been made using the temperature correction factor specified in IRC 115, as detailed below.

$$ET1 = \lambda ET2$$

Where,

ET_1= back-calculated modulus of bituminous layer at temperature T_1

ET_2 =back-calculated modulus of bituminous layer at temperature T_2

λ = temperature correction factor, is given as

$$\lambda = (1-0.238 \ Ln \ T1)/(1-0.238 \ Ln \ T2)$$

7.4. ***Correction for Seasonal Variation:*** Moisture content influences the strength of subgrade and granular subbase/base layers. The impact on strength depends on factors such as the type of subgrade soil, the gradation of the granular layers, and the nature of fines present. According to IRC:115-2014, the weakest condition of the granular layers and subgrade occurs during the post-monsoon season. Since the survey was conducted in winter season, winter seasonal correction factor has been adopted as given below.

- Seasonal Correction factor for Granular layers, $\mathbf{E_{mon}= 10.5523 \times (E_{win})0.624 – 113.857}$
- Seasonal Correction factor for Subgrade layer, $\mathbf{E_{mon}= 3.351 \times (E_{win})0.7688 – 28.9}$

7.5. ***Delineation of Homogeneous sections:*** The selected road stretch has been sub-divided into smaller units, known as “Homogeneous sections”, ensuring that each sub-divided section of the road exhibits uniform characteristics throughout.

A commonly used statistical method for identifying homogeneous sections is the "Cumulative Difference" approach, which has been widely applied in numerous highway projects across India. In this method, the actual cumulative sums of a measurement series are compared to the sums that would have resulted from adding the average values. The difference between these two sums is referred to as the cumulative difference. The Surface Curvature Index (SCI), calculated as the difference between D0 and D300, serves as a bowl shape parameter that reflects the contribution of the upper layers. This parameter is used to calculate the cumulative difference.

Whenever there is a change in the slope of the cumulative difference vs. distance (or number of test locations) plot, it indicates a potential delineator for identifying homogeneous sections. The homogeneous sections identified for the project stretch are shown in the ***Table 6*** below along with the corresponding graph.

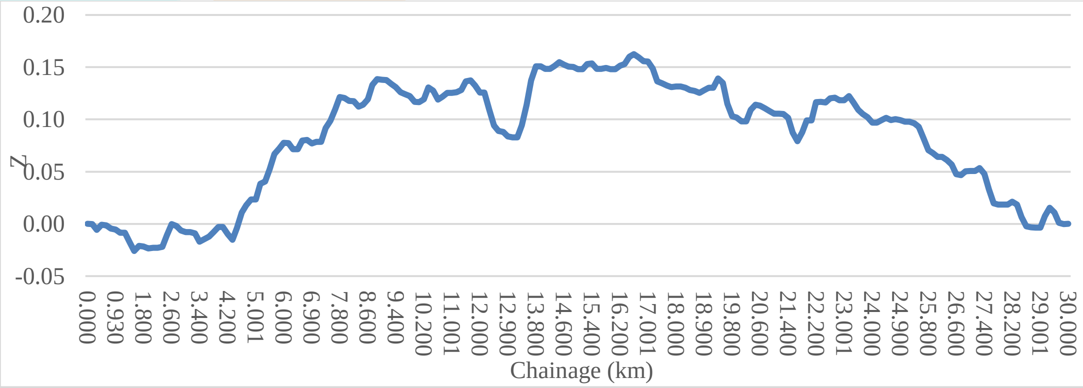


Figure 4: Delineation of Homogeneous Section LHS

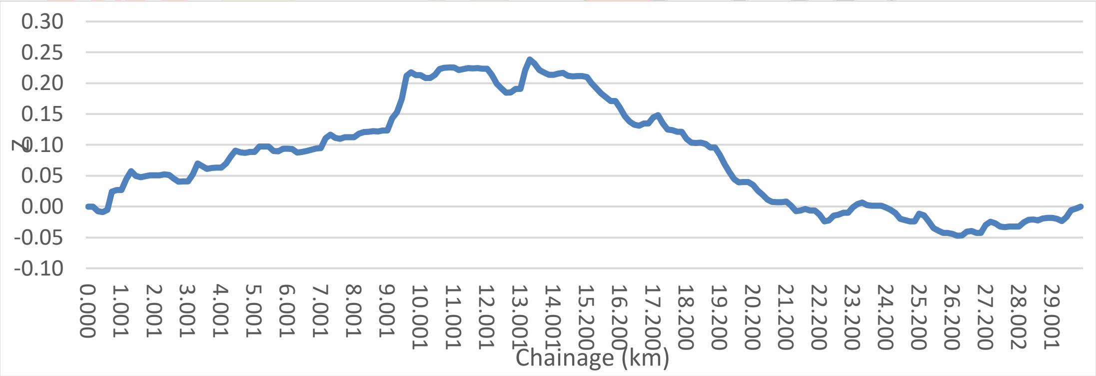


Figure 5: Delineation of Homogeneous Section RHS

Based on the above plot, the following sections given in ***Table 6*** have been identified in each direction

Table 6: Homogeneous sections

LHS

HS. No	From (km)	To (km)	Length (km)	Remarks
1	0.000	2.400	2.400	
2	2.400	4.400	2.000	
3	4.400	5.900	1.500	
4	5.900	8.800	2.900	
5	8.800	11.000	2.200	
6	11.000	13.000	2.000	
7	13.000	14.900	1.900	
8	14.900	16.600	1.700	
9	16.600	18.200	1.600	
10	18.200	20.000	1.800	
11	20.000	20.900	0.900	
	20.900	21.100	0.200	Toll Plaza
	21.100	21.800	0.700	
12	21.800	24.000	2.200	
13	24.000	26.600	2.600	
14	26.600	28.800	2.200	
15	28.800	30.000	1.200	

RHS

HS. No	From (km)	To (km)	Length (km)	Remarks
1	0.000	2.800	2.800	
2	2.800	4.800	2.000	
3	4.800	6.600	1.800	
4	6.600	8.800	2.200	
5	8.800	10.400	1.600	
6	10.400	12.800	2.400	
7	12.800	15.001	2.201	
8	15.001	17.000	1.999	
9	17.000	19.000	2.000	
10	19.000	20.900	1.900	
11	20.900	21.100	0.200	Toll Plaza
	21.100	23.000	1.900	
12	23.000	26.400	3.400	
13	26.400	29.200	2.800	
14	29.200	30.000	0.800	

7.6. *15th Percentile MR layer moduli:* To adopt a conservative approach, the 15th percentile MR values were used in the analysis to ensure that the pavement design accounts for weaker-than-average subgrade conditions, thereby reducing the risk of premature failure. It is derived from a statistical distribution of measured MR values, selecting the value below which 15% of the data points fall. This approach helps in designing pavements that remain functional even under less favorable conditions.

VIII. RESIDUAL LIFE CALCULATIONS:

The residual life of flexible pavement is the remaining number of standard axle load repetitions that the pavement can sustain, after considering the load effects of the traffic it has already carried. It is typically expressed in terms of Million Standard Axles.

- Fatigue Life of Bituminous layer, $\mathbf{N_f = 0.711*10^{-4}\ X\ [1/E_t]3.89*[1/MR]^{0.854}}$
- Rutting Life of Subgrade layer, $\mathbf{N_R = 1.41*10^{-8}\ X\ [1/E_v]^{4.5337}}$

where,

E_t – Horizontal Tensile strain at bottom of bituminous layer

E_v – Vertical compressive strain at top of subgrade layers

The critical strain values were determined using IITPAVE software with the required input parameters as in ***Table 7*** below.

Table 7: Input Parameters for remaining life calculations

No of layers	3
Layer Thickness	As adopted for each Homogeneous section from Test pits
Modulus values	As calculated for each Homogeneous section
Poisson's ratio	0.35, 0.35, 0.35
Wheel set	2
Wheel Load	20KN
Analysis points	4
Tyre Pressure	0.56 Mpa

The residual life of the existing pavement was evaluated for overlay improvement by comparing it with the projected 10th-year design traffic, expressed in terms of MSA. A total of 11 sections on the LHS and 7 sections on the RHS fell below the target life, warranting the need for overlay treatment.

IX. OVERLAY DESIGN:

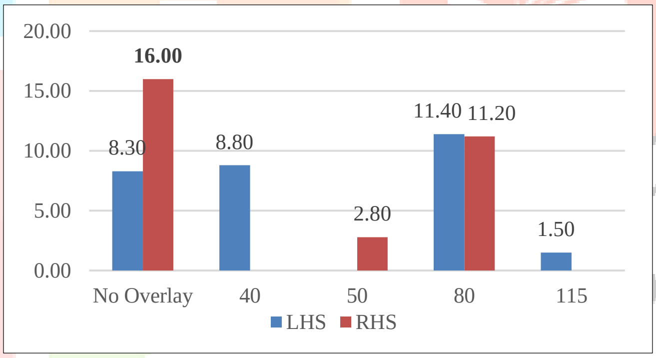
Sections that fell short of the target life were designed for overlay using VG-40 grade bitumen, along with 20 mm milling of the existing surface to prevent the propagation of existing distresses into the new surface course. The enhanced pavement life was calculated by increasing the thickness of the bituminous layers as part of the overlay design. The expected life of the improvements was determined using IITPAVE by comparing it against the target life. This process was iteratively repeated, with incremental increases in overlay thickness, until the newly proposed thicknesses achieved the desired target life. The following input parameters given in ***Table 8*** were considered in the calculation of critical strains:

Table 8: Input Parameters for overlay design

No of layers	4
Layer Thickness	Proposed BT thickness given for layer -1. Thicknesses evolved from Test pit data the respective homogeneous sections were adopted for remaining 3 layers
Modulus values	3000Mpa for newly proposed layer-1. Corrected 15th percentile MR values for respective homo section for remaining 3 layers
Poisson's ratio	0.35, 0.35, 0.35,0.35
Wheel set	2
Wheel Load	20KN
Analysis points	4
Tyre Pressure	0.56 Mpa

X. RESULTS AND DISCUSSIONS:

Following the successful completion of the study, the proposed overlay thicknesses were determined as follows to achieve the 10th-year target life of 30 MSA for the LHS and 28 MSA for the RHS, respectively.



XI. CONCLUSIONS:

This paper presents partial findings from a recently completed study on the Trichy–Madurai section of NH-38, which focused on the structural evaluation of flexible pavements using the Falling Weight Deflectometer (FWD). A total length of 30 km was selected for the study. FWD testing was conducted along these segments to assess the structural condition of the existing pavement. Based on detailed investigations, the following conclusions have been summarized:

i. The current VDF calculations reveal that the impact of multi-axle vehicles on road deterioration is significantly higher on the LHS (Madurai-bound) carriageway, being 16% greater than on the RHS (Trichy-bound). Conversely, the influence of 2-axle and 3-axle vehicles is notably lower on the LHS, 20% less compared to the RHS carriageway

ii. The 10th-year design traffic is higher on the LHS (Madurai-bound) carriageway compared to the RHS (Trichy-bound). Consequently, the LHS carriageway is likely to experience greater deterioration in the future.

iii. It has been observed that the rutting life of the existing pavement significantly exceeds the target life in both directions. On the LHS, the rutting life is 4 to 96 times greater than the target life of 30 MSA, while on the RHS, it is 12 to 93 times higher than the target of 28 MSA life. This confirms that the subgrade is sufficiently robust to sustain the design traffic load anticipated for the 10th year.

iv. Regarding the fatigue life of the existing pavement, 11 out of 15 sections on the LHS carriageway (HS-1, HS-2, HS-3, HS-4, HS-5, HS-6, HS-7, HS-9, HS-10, HS-14, and HS-15) exhibit significantly lower fatigue life compared to the target MSA. Similarly, on the RHS carriageway, 8 out of 14 sections (HS-1, HS-2, HS-3, HS-4, HS-5, HS-13, and HS-14) exhibits lower fatigue life and therefore require strengthening.

v. Based on residual life calculations, overlay strengthening is needed for 21.7 km on the LHS (72% of the total length) and 14 km on the RHS (47% of the total length) of the carriageway.

vi. The overlay has been designed to enhance the strength of the existing pavement by recommending the use of VG-40 grade bitumen along with 20 mm surface milling. Accordingly, on the LHS carriageway (Madurai Bound), a 40 mm thick overlay is proposed over 8.80 km, an 80 mm overlay over 11.40 km, and a 115 mm overlay over 1.5 km of in-service pavement. On the RHS carriageway (Trichy Bound), a 50 mm overlay is recommended for 2.80 km, and an 80 mm overlay for 11.20 km.

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