Flexible Pavement Design and Subgrade Characteristics Strength

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Abstract- *The Salwala – Gojjar Road OSR-05 under study is located in the Northern part of India (Himachal Pradesh). The said road serves the tourists in commuting to Himalayas, hill stations (Shimla, Manali, Dharamshala, Dalhousie, Chamba, Khajjiar, Kullu and Kasauli) as well as 9 Devi Temples of Himachal Pradesh and Jammu region. In view of this, it has been proposed to develop the road as per standards for augmenting the capacity of design traffic with significant service life.*

In the current study for the flexible new pavement design, we have considered the parameters such as volume of traffic, growth rates and Vehicle Damage Factors (VDF) as well as evaluated soil and materials of the exisitng road to identify the existing strength of pavement and also to characterize the various layers of pavement materials in design.Existing Intermediate lane isstructurally poor in condition as well as heavy distresses, so failing under rutting criteria which shows deficiency in granular layers. Hence it is recommended for reconstruction over entire project road with Design Option-III i.e., Design of Bituminous Pavements with Cemented Sub-Base (CTSB) and Granular Base course (WMM).

I. INTRODUCTION

Design Procedure of Flexible Pavement Design:

Flexible pavement is modelled as an elastic multilayer structure. Stresses and strains at critical locations are computed using a linear layered elastic model (Figure 1). The stress analysis software IITPAVE has been used for computation of stresses and strains in flexible pavements. Tensile strain, ε_t, at the bottom of the bituminous layer and the vertical sub-grade strain ε_v, at top of the sub-grade are conventionally considered as critical parameters for pavement design to limit cracking and rutting in the bituminous layers and non-bituminous layers respectively.

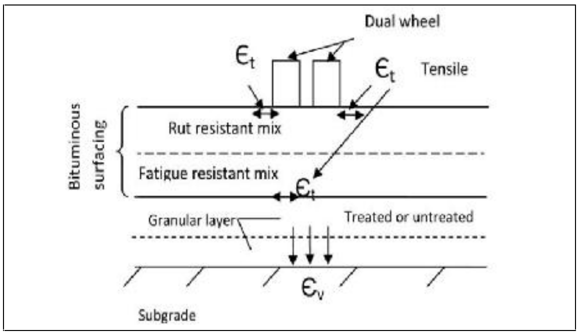

Figure 1: The different stresses and strains in a flexible pavement.

IRC method for pavement design is based on limiting the vertical compressive strain on top of sub-grade which results in permanent deformation of the pavement and the horizontal tensile strain at the bottom of the bituminous layer which results in cracking of the pavement. The relationships governing the above two pavement failure criteria are expressed as:

Fatigue Model: With every load repetition, the tensile strain developed at the bottom of bituminous layer develops micro cracks, which go on widening and expanding till the load repetitions are large enough for the cracks to propagate to the surface over an area of the surface that is unacceptable from long term serviceability of the pavement point of view. The phenomenon is called fatigue of the bituminous layer and the number of load repetitions in terms of standard axles that cause fatigue denotes the fatigue life of the pavement. The two equations for the conventional bituminous mixes designed are given below:

$N_f = 1.6064 x C x\ 10^{-4}\ (1/\varepsilon_t)^{3.89} * (1/MR)^{0.854}$ (80% Reliability)

$N_f = 0.5161 x c\ x\ 10^{-4}\ (1/\varepsilon_t)^{3.89} * (1/MR)^{0.854}$ (90% Reliability)

Where,

N_f = Fatigue life in number of standard axles

ε_t = Maximum tensile strain at the bottom of the bituminous layer, and

M_R = Resilient modulus of the bituminous layer
C = 10M, and $M=4.84(V_b/(V_a+V_b)-0.69)$
Corresponding to the values of V_a and $V_b$as stated above the above equation for 80% reliability is as given ;V_a= 4.5% and V_b = 10.5% and 90% reliability is given V_a= 3.5% and V_b = 11.5% has been considered for Pavement Design as per Clause12.3 IRC 37-2018.

Rutting Model: The model considers the vertical strain in sub-grade as the only variable for rutting, which is a measure of bearing capacity of the sub-grade. The two-rutting equation for 80% and 90% reliability levels are given below;

$N = 4.1656 \times 10^{-8} (1/\varepsilon_v)^{4.5337}$ - (80% Reliability Level)
$N = 1.4100 \times 10^{-8} (1/\varepsilon_v)^{4.5337}$ - (90% Reliability Level)
Where,
N - Number of cumulative standard axles, and
ε_v - Vertical strain in the sub-grade

A bituminous pavement generally consists of bituminous surfacing course and a bituminous base/binder course. Dense Bituminous Macadam (DBM)/Bituminous Macadam (BM) can be used as base/binder courses.

As per Clause 9.1 and Table 9.1 IRC:37-2018, VG30 Bituminous type is recommended for Non National highways with Design Traffic level less than 20 Msa . For design traffic level of 20 MSA and up to 50 MSA the recommended guide lines are using VG 40 grade bitumen.

For snow bound locations, softer binders such as VG10 may be used to limit thermal transverse cracking (especially if the maximum pavement temperature is less than 30°C).

II. SCOPE OF THE WORK

To design the new flexible pavement as per the guidelines of IRC: 37-2018 "Guidelines for the design of flexible pavements" by considering the following aspects
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- To evaluate the strength of the pavement by performing soil and material characterization of the exisitng road.
- To characterize the various layers of pavement materials of existing road.
- To design and recommend the new flexible pavement in accordance the traffic survey predictions of existing project road by considering volume of traffic, growth rates and Vehicle Damage Factors (VDF).
- To design and recommend the new flexible pavement with significant service life and as per standards for augmenting the capacity of design traffic.
- Rehabilitation of existing two/intermediate/single-lane by cost-effective solutions, and designing of new two-lane/Intermediate/Four lane carriageway.

III. PROJECT DESCRIPTION

The state of Himachal Pradesh is located in the Northern part of India. In terms of population, it has only 0.57% of India's total population. Tourism in Himachal Pradesh is a major contributor to the state's economy and growth. The Himalayas attracts tourists from all over the world. Hill stations like Shimla, Manali, Dharamshala, Dalhousie, Chamba, Khajjiar, Kullu and Kasauli are popular destinations for both domestic and foreign tourists. Apart from the tourism perspective people will also visit 9 Devi Temples of Himachal Pradesh and Jammu region and is an all-season Devi Pilgrimage itinerary where devotees visit all major Devi temples in Himachal Pradesh with Vaishnodevi such as Mansadevi, Naina Devi, Chintpurni, Jawalamukhi, Vaishnodevi, Vrajeshwari, Chamundadevi, Kalika Devi and Shakumbhri Devi. Himachal Pradesh is bordered by Jammu and Kashmir on the north, Punjab on the west, Haryana on the southwest, Uttarakhand on the southeast, and Tibet on the east. At its southernmost point, it also touches the state of Uttar Pradesh. Project roads present in Sirmaur district, namely Salwala – Gojjar Road OSR-05. The Details of the project road is given in below Table 1.

S. No	Road Name	District	Design Length (km)	Connecting roads
1	Salwala – Gojjar Road OSR-05	Sirmaur	14+820	OSR-03, Dakpatthar (HP-Uttarakhand Border)

Table 1: Details of Project Road

IV. CASE STUDY

4.1 Investigation on subgrade of Existing pavement (Test pit analysis)

The subgrade investigations were carried out to know the strength properties of the existing soil. Visual inspection of the existing pavement condition was conducted prior to commencement of sub-grade investigation work. The general testing scheme of existing road consists of testing at least two kilometer interval subgrade soil samples for road. Even though same soil strata were found on lengthy homogeneous sections, it was ensured that subgrade strength test pits were dug in the interval of 2 km. While collecting soil samples, various in-situ tests were conducted. The laboratory tests included are summarized in Table 2**.**The pavement composition details (pavement course, material type, and thickness) were also recorded at every test pit.

Discussion on the tests conducted and results obtained are carried out in the following sections.

S. No	Type of Soil Sample	Testing Criteria	
		Description of Test	**Standard Code Applicable**
Existing Subgrade			
i)	**Existing Sub-grade Strength Test Pits**	In-situ Density	IS 2720 (Pat-29)
		In-situ Moisture Content	IS 2720 (Pat-2)
		Dynamic Cone Penetration Test	TRRL (U.K.) vide Road Note No. 31
		Soil Classification	IS 1498
		Grain Size Analysis	IS 2720 (Pat – 4)
		Atterberg Limits	IS 2720 (Pat – 5)
		Laboratory Compaction Test	IS 2720 (Pat – 8)
		4-days soaked CBR	IS 2720(Pat – 16)
		Free swell Index	IS:2720 (Pat-40)
Materials Investigation			
ii)	**Borrow Area Soil**	Soil Classification	IS 1498
		Sieve Analysis	IS 2720 (Pat – 4)
		Atterberg Limits	IS 2720 (Pat – 5)
		Laboratory Compaction Test	IS 2720 (Pat – 8)
		4-day soaked CBR	IS 2720(Pat – 16)
		Free swell Index	IS:2720 (Pat-40)
iii)	**Coarse aggregate samples from crushers/quarries**	Sieve Analysis	IS:2386 (Part-1)
		Flakiness and Elongation Index	IS 2386 (Part – 1)
		Specific Gravity and Water Absorption	IS 2386 (Part – 3)
		Aggregate Impact Value (AIV)	IS 2386 (Part – 4)
		Stripping and Coating test	IS 6241
		Los Angeles Abrasion Value (LAV)	IS 2386 (Part-4)
iv)	**Fine Aggregates (Sand)**	Grain Size Analysis	IS:2386 (Part – 1)
		Designation of zone	IS:383 – 1997
		Specific Gravity	IS 2386 (Part – 3)
		Fineness Modulus	IS 383 – 1997

Table 2: Details of Testing Criteria and codes

4.1.1 Field Tests and Results

Field tests were conducted as per the project requirement to determine the subgrade characteristics and strength. The field testing for subgrade soil includes.

- In-situ density and moisture content at each test pit
- Field CBR using Dynamic Cone Penetration test at few locations.

(a) Field Density & Moisture Content

In-situ density (field density) and moisture content were determined as per the standards enlisted in Table **.** Field density was used to evaluate the degree of compaction and existing subgrade CBR at field density state. The details of field dry density (FDD) and field moisture content (FMC) test results are givenTable **.**

(b) Dynamic Cone Penetration Test

Dynamic Cone Penetration tests were conducted at subgrade strength test pit locations to assess in-situ CBR on existing alignment soil, which will be below sub-grade level. The CBR value was calculated based on different soil layers encountered. The slope change in the graph (Penetration *Vs* Number of Blows) indicates the interface of two layers of different penetration resistance. From the graph, thickness of layer and slope (penetration mm/blow) were calculated. The following equation was used to calculate the layer DCP-CBR value for each layer.

$$log_{10}CBR = 2.465 - 1.12 \times log_{10}(mm/blow)$$

These layered CBR values have been converted to overall CBR value using Japanese formula for the purpose of DCP CBR.

$$OverallCBR = \left\{ \sum layerthickness \times (DCP-CBR)^{1/3} \Big/ \sum layerthickness \right\}^3$$

Dynamic Cone Penetration test results showing penetration of cone in cm and number of blows at each pit are plotted. The DCP- Equivalent CBR along the project road. The DCP- Equivalent CBR values may not be comparable with laboratory CBR as the penetration of DCP cone may widely affect with several factors like filed moisture, layer underneath of subgrade and any obstacles (Boulder/ tree routes). The DCP-CBR value may increase with decreasing of in-situ moisture and vice versa. If any boulder/ stone is obstructed the penetration, the DCP-CBR value will be higher. In-light of all, the DCP-CBR may not be comparable with laboratory 4-days soaked CBR values. The summaries of DCPT-CBR values are included inTable **.**

The graphical representation of filed dry density Vs Maximum dry density presented in **Figure** . Some of the photographs are given in Figure .

S.No	**Existing Chainage (km)**	**Side**	**DCP-CBR (%)**	**FMC (%)**	**FDD(g/cc)**
OSR-05					
1	0+200	LHS	8.0	14.6	1.823
2	2+200	LHS	7.0	11.3	1.655
3	4+800	LHS	11.0	8.7	1.483
4	6+400	LHS	13.4	13.4	1.567
5	8+400	LHS	5.8	12.4	1.800
6	10+400	LHS	9.8	10.8	1.856
7	12+400	LHS	9.0	7.6	1.859
8	14+400	LHS	7.4	11.6	1.635
Average			**8.9**	**11.3**	**1.709**

Table 3: Statistical Summary of Field Test Results

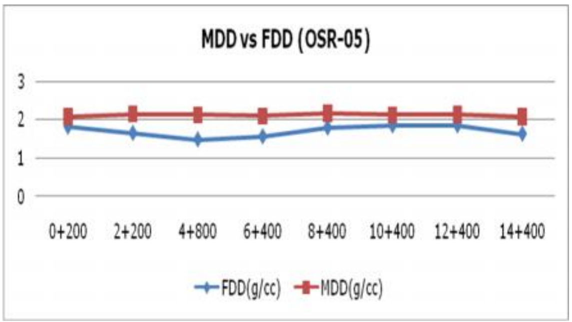


Figure 2: Illustrative Summary of FDD & MDD along the Project Road

Photo showing Performance FMC Test on OSR-05.	Photo showing Performance DCP Test on OSR-05 Road.	Photo showing the of Pavement Composition

Figure 3: Photographs showing Filed Investigations along the theProject Road

4.1.2 Laboratory Tests and Results

The laboratory testing for subgrade includes:

- Characterization (Grain size analysis, Atterberg limits and free Swell Index) at each of the subgrade strength test pit
- Laboratory moisture-density characteristics (For subgrade strength test pits)
- Laboratory 4-day soaked CBR test and unsoaked CBR (For subgrade strength test pits)

More than 50 kg of soil sample was collected in damp proof bag(s) from each test pit for testing purposes. The details like location/ chainage & other identification marks were recorded for the sample bags. These bags were double packed with care so that no damage would occur while transporting to the laboratory for conducting the tests mentioned in Table .

Sl. No.	Existing Chainage (km)	Side (LHS / RHS)	Description of Soil	Gradation: Percent by weight retained the sieve (IS:2720 Part- IV)				Clay and Silt Content (%)	Atterberg Limits (IS:2720 Part - V)			Soil Classification	Modified Proctor Test		4-Days Soaked CBR (%)	Free Swell Index (%)	Degree of Compaction (%)
				4.75 mm	2.00 mm	425 micron	75 micron		Liquid Limit (LL) %	Plastic Limit (PL) %	Plasticity Index (PI) %		OMC (%)	Max. dry density (gm/cc)			
1	0+200	LHS	Gravelly Sand with Clay and Silt	26.40	16.90	17.80	15.40	23.50	29.2	17.4	11.8	SC	10.8	2.07	18.9	15	88.1
2	2+200	LHS	Gravelly Sand with Silt	40.60	11.50	15.50	14.30	18.10	25.2	NP	NP	SM	10.8	2.15	19.0	10	74.0
3	4+800	LHS	Gravelly Sand with Silt	33.60	15.80	14.80	15.40	20.40	25.5	NP	NP	SM	9.8	2.12	20.5	10	76.2
4	6+400	LHS	Gravelly Sand with Clay and Silt	30.40	17.30	15.10	16.90	20.30	28.4	16.7	11.7	SC	11.0	2.09	19.1	15	86.4
5	8+400	LHS	Sandy Gravel with Silt	43.60	9.50	12.40	15.80	18.70	25.2	NP	NP	GM	9.8	2.17	19.5	10	73.2
6	10+400	LHS	Gravelly sand with Silt	30.80	17.40	16.80	14.50	20.50	26.4	NP	NP	SM	10.9	2.12	12.0	10	97.3
7	12+400	LHS	Gravelly Sand with Silt	37.30	13.90	14.80	12.60	21.40	26.6	NP	NP	SM	11.2	2.13	19.1	10	87.3
8	14+400	LHS	Gravelly Sand with Clay and Silt	15.20	20.20	15.80	16.40	32.40	29.8	17.8	12	SC	10.3	2.07	11.0	30	79.0

Table 4: Summary of Existing Subgrade Soil Test Results

(a) Soil Classification and Distribution

The Physical properties of Existing Subgrade soil test pit results are given in Table 4below.

4.2 Input Parameters for Pavement Design

The performance and design of pavements is affected by several factors which includes the following primly discussed parameters as described below.

4.2.1 Design Life of Pavement Layers

The flexible pavement design has been carried out in accordance with IRC: 37-2018. As per clause 4.3.1 of IRC: 37-2018, a design life of 15 years has been considered for flexible pavement design.

4.2.2 Design MSA

(a) Annual Average Daily Traffic

The base-year Annual Average Daily Traffic (ADT) is classified into commercial, passenger and non-motorized traffic based on the classified traffic volume count surveys carried out on the project road. The details of Annual Average Daily Traffic (AADT) are given in the relevant traffic chapter and summary of commercial vehicles in the base year of 2019 is given in below.

Road No	Mini Bus/Bus	LCV	2 Axle	3 Axle	MAV
OSR-05	88	184	82	84	36

Table 5: Road wise Annual Average Daily Traffic (Base year, 2019)

(b) Traffic Growth Rates

The mode-wise percentage growth factors derived on the basis of traffic demand estimates are adopted for projection of traffic data and the same is presented in relevant traffic report. The project roads are in Sirmaur district thus growth rates are used accordingly.

The summary of commercial traffic growth rates is given in the Table 6. The cumulative standard axle repetitions were calculated. As per clause 4.2.2 of IRC:37-2018 if the annual growth rate is less than 5% a minimum annual growth rate of 5 per cent should be used for commercial vehicles for estimating the design traffic.

Year	Bus	LCV	2-Axle	3-Axle	MAV
2019-2023	7.28	9.21	5.00	5.00	5.00
2024-2028	6.19	7.83	5.00	5.00	5.00
2029-2033	5.26	6.66	5.00	5.00	5.00
2034-2038	5.00	5.66	5.00	5.00	5.00

Table 6: Traffic Growth Rates (in percentage)

(c) Lane Distribution Factor

The lane distribution factors adopted for the project are as given below:

- 2-lane: 50 percent of the number of commercial vehicles in both directions (as per clause 4.5 of IRC: 37-2018).

(d) Vehicle Damage Factor

The axle load survey, the spectrum of axle loads and the numbers of equivalent 8.16 t standard axles for the different categories of commercial vehicles have been determined on the basis of the axle load surveys. The equations for computing equivalency factor for single, tandem and tridem axles given below is used as directed in the IRC:37-2018 for converting different axle load repetitions into equivalent standard axle load repetitions;

- Single axle with single wheel on either side
  $= \{\text{axle load in kN} / 65\}^4$
- Single axle with dual wheel on either side
  $= \{\text{axle load in kN} / 80\}^4$
- Tandem axle with single wheel on either side
  $= \{\text{axle load in kN} /148\}^4$
- Tridem axle with dual wheel on either side
  $= \{\text{axle load in kN} /224\}^4$

The relationship is referred to as the ‘Fourth Power Rule’, which states that the damaging effect of an axle load increases as the fourth power of the weight of an axle. In order to convert axle loads from survey data into ESAL, each axle of each category of vehicle is multiplied by equivalency factor of that type of axle. The output is the ‘damage’ caused by that particular axle on the pavement. Damages by all axles are then added to find the cumulative damage by that type of vehicle. The VDF is calculated by using the following equation:

$$\text{VDF} = \frac{\text{Cumulative Damage}}{\text{Sample Size}}$$

As such axle load survey was not conducted for this road as a part this project (HPRIDC) but the same survey has been carried out in other project (HPRAMS) by the consultants and based on past on past experience values on other roads the adopted values are presented below.

Road No	Mini Bus/Bus	LCV	2 Axle	3 Axle	MAV
OSR-05	1.2	1.5	5.2	5.6	7.7

Table 7: **Summary of adopted VDF values**

(e) Design Traffic (Cumulative Number of Standard Axles)

The traffic loading in terms of the cumulative number of standard axles for the given period has been computed using the following relationship as given in IRC: 37-2018:

$$N = \frac{365 \times \left\{ (1+r)^n - 1 \right\}}{r} \times A \times D \times F$$

Where,

N = Cumulative number of standard axles to be catered for the design life in terms of msa.

r = Annual growth rate of commercial vehicles

n = Design life in years

A = Initial traffic in the year of completion of construction in terms of number of commercial vehicles per day exceeding 3 tonnes

D = Lane distribution factor

F = Vehicle Damage Factor

The traffic in the year of completion is estimated by using following formula:

A =P (1r/100)x

Where,

P - Number of commercial vehicles as per count

x - Number of years between the count and the year of

Completion of construction.

By using stated above equation obtained section wise design traffic was given in Table 8.

Road No	Design Start Ch (Km)	Design End Ch (Km)	Length (km)	Lane Configuration	Design Period (Years)	MSA	Design MSA
OSR-05	0+000	14+820	14+820	2Lane	15	8.62	10

Table 8: Summary of Design traffic

4.3 Subgrade CBR for New Pavement Design

Extensive survey was conducted to locate the potential sources of borrow area soils required for the construction of embankment and subgrade with in the reasonable lead distance.

Borrow samples are collected for OSR05.Identified borrow areas have been tested and their 4-days laboratory soaked CBR at OMC has been determined and presented in the **Chapter-2**. On analyzing the test results of all borrow areas, selected earth of two borrow areas are found suitable for subgrade construction. There is a significant difference between CBR value used in the subgrade and in the embankment. As per Clause-6.4.1 of IRC: 37-2018 the effective CBR value has been determined and presented in Table 9.

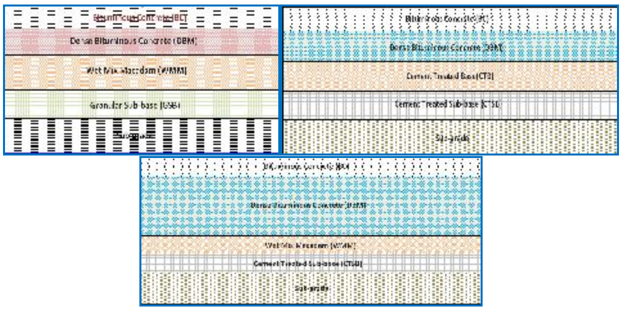
S.No	Road No	Effective Design CBR (%) as per Clause-6.4.1 of IRC:37-2018	Adopted Design CBR (%)
1	OSR-05	18.26%	10%

Table 9: Design Subgrade CBR

As per clause 6.4.2 of IRC: 37-2018 maximum resilient modulus value for Design should be limited to 100Mpa. However**10%** CBR is adopted for Design Purpose.

4.4 Design of New Flexible Pavement

Design of new flexible pavement applies to widening portion of existing carriageway, new two/intermediate/four-lane carriageway and reconstruction stretches (if any). IRC: 37-2018 is referred as design guide for flexible pavement design, and the same is explained in the earlier section. Three design options are consider for evaluating cost-effective design solution, and the options are as follows. The proposed Pavement Composition for New Construction/Reconstruction road is detailed in Figure 4 and Table 10.



(a) (b) (c)

Figure 4:(a)Design Option I: Design of conventional flexible pavement **(b)Design Option II:** Design of bituminous pavements with cemented base and cemented sub-base with Aggregate Inter Layer (AIL) and**(c)Design Option III:** Design of Bituminous pavements with CTSB and Granular base course as per IRC 37-2018.

To provide scientific design thickness Mechanistic–Empirical pavement design" procedure is followed as described in IRC: 37-2018 with the help of IITPAVE software.

Note: In sub base layer a minimum thickness of 200mm of Granular Sub-base/ Cement Treated sub-base is proposed in different pavement options.

- As per Clause 7.2.1 of IRC: 37-2018 if the thickness of the Granular sub-base layer is equal to 200mm then grade V or VI of MoRTH specifications should be used in sub-base layers.
- As per clause 7.3.1 of IRC: 37-2018 the recommended aggregate gradation for the Cement treated sub-base material is Grading-IV of Table 400-1 of MoRTH specifications.

For current project road Design Traffic is 10MSA with Design CBR of 10%. Design option for all road sections is tabulated below.

Design Traffic (Msa)	**10**				
Design CBR (%)	**10**				
Bituminous Type	**VG30**				
New Pavement Option-I (conventional) with VG-30					
Crust Composition	**BC**	**DBM**	**WMM**	**GSB**	
	30 mm	50 mm	250 mm	200 mm	
New Pavement Option-II (AIL, CTB & CTSB)					
Crust Composition	**BC**	**DBM**	**AIL**	**CTB**	**CTSB**
	30 mm	50 mm	100 mm	120 mm	200 mm
New Pavement Option-III (WMM & CTSB)					
Crust Composition	**BC**	**DBM**	**WMM**	**CTSB**	
	30 mm	50mm	**150**	**200**	

Table Error! No text of specified style in document.**0: Proposed Pavement Composition for New Construction/Reconstruction**

4.5 Rehabilitation of existing Pavements

Falling Weight Deflectometer (FWD) is an impulse-loading device in which a transient load is applied to the pavement and the deflected shape of the pavement surface is measured. The working principle of a typical FWD is illustrated in**Figure**

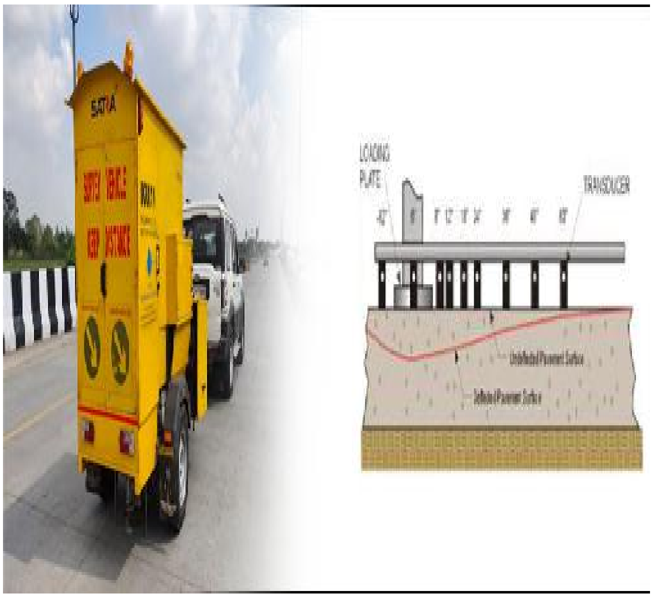


Figure 5 : Falling Weight Deflectometer

Impulse load is applied by means of a falling mass, which is allowed to drop vertically on a system of springs placed over a circular loading plate. The deflected shape of the pavement surface is measured using displacement sensors which are placed at different radial distances starting with the centre of the load plate.

Principle Used

A falling mass in the range of 200 kg is dropped from a height of fall in the range of 100 to 600 mm to produce load pulses of desired peak load of 40 kN (+/- 4 kN) and duration of 20-30 ms. The target peak load of 40 kN (+/- 4 kN) applied on bituminous pavements corresponds to the load on dual wheel set of a 80 kN standard axle load and duration of 20-30 ms simulates traffic moving at a speed of 60 KmpH. The corresponding peak vertical surface deflections at different radial locationsare measured and recorded. The target peak load can be decreased suitably if the peak maximum (central) deflection measured with 40 kN load exceeds the measuring capacity of the deflection transducer. If the applied peak load differs from 40 kN within the above mentioned range, the measured deflections have to be normalized linearly during the analysis to correspond to the standard target load of 40 KN. Sufficient number of deflection transducers shall be used to adequately capture the shape of deflection bowl. Six to nine velocity transducers (geophones) are generally adequate for measuring surface deflections of flexible pavements.

For the present work, the defined mass has been dropped form a height of 70mm to produce the target load and deflections have been measured from 7 deflection transducers placed at 0, 200, 300, 450, 600, 900, and 1200 mm.

Procedure

The FWD measurements have been carried out at spacing of as given IRC: 115-2014 respectively in the provided stretches.

Deflections measured by the FWD equipment are influenced by pavement temperature. Measurements made when the pavement temperature is different than standard temperature has be corrected. The deflection measurements, pavement temperature, subgrade soil & deflection, and other information collected during the deflection study have been recorded.

Following procedure has been followed for measurement of FWD:

- The test location is marked on the field.
- The loading plate along with the deflection sensors have been lowered at the test location.
- The target load has been applied and the deflections have been measured for 3 times.
- The first load is considered as a seating load and the values are not adopted for analysis.

Temperature Measurement

The standard temperature for doing the experiment is 35^{o}C. Since it is not possible to conduct the test at the standard temperature, a correction factor has to be applied for the deflection. The correction factor is determined by knowing the temperature at the time of the survey. The pavement temperature during the survey has been measured for every one hour by drilling a hole of 40mm in the pavement and filling it with glycerol.

Correction for Temperature

The stiffness of bituminous layer is highly susceptible to temperature and hence consequently the surface deflections of a given pavement will vary depending on the temperature of the constituent bituminous layers. If the depth of the BT surface is more than 40mm, then correction factor has to be applied. If the depth is less i.e., if it is a thin bituminous surfacing like premix carpet and surface dressing, then no correction is required. Correction for temperature variation on deflection values measured at temperature other than 35ºC should be calculated by the formula provided in IRC 115.

The key points to consider are:

- Pavement temperature range applicable for correction factor – 20^{0}C to 45^{0}C.
- FWD shall not be carried for pavement temperature more than 45^{0}C
- Temperature correction not required for following cases
  - bituminous layers (depth < 40 mm)
  - “Poor” sections
  - Where average daily temperature is < 20^{o}C for more than 4 months

Correction for Seasonal Variation

Moisture content affects the strength of subgrade and granular subbase/base layers. The extent to which the strength is affected will depend on the nature of subgrade soil, gradation and nature of fines in the granular layers, etc. For the purpose of applying these guidelines, it is intended that the pavement layer moduli values should pertain to the period when the subgrade is at its weakest condition. As per IRC: 115-2014, granular layers and subgrade will be in its weakest condition during the post-monsoon season. Since survey was conducted during winter season seasonal correction as per Equation 6 and Equation 9 of IRC: 115-2014 was applied.

Calculations and Interpretation

The falling weight Deflectometer (FWD) Survey completed after the raw deflection data to normalised 40KN.

KGPBACK (or) PVD – Genetic Algorithm based software for back calculating the layer moduli

- Three layer system (Bituminous, Granular & Subgrade)
- Data required:

Existing layer thickness – the existing layer thicknesses of various stretches have been obtained from the Client based on the construction records.

Layer moduli range – the layer moduli adopted for the analysis is shown in next section.
Poisson's ratio:

- BT-0.35, Granular-0.35, Subgrade-0.35 (For Back calculation and IITPAVE analysis without considering overlay)

Range of Moduli

The adopted moduli values are given below table.

Type of Layer	**Lower and Upper Limit (Mpa)**
Subgrade	The Existing CBR (%) is 10 to 15.And the current project road adopted E- Values for Sub grade is 76.8 and 100.
Granular	IRC: 37-2018, $M_{R\_gran}=0.2*h^{0.45}*M_{R\_sub}$ h= thickness of granular layer For Lower limit M_{R_sub}= E-Subgrade Upper limit M_{R_sub}= E-subgrade(Refer lower and upper limit for subgrade given above) Or Standard As per IRC 115-2014 is 100 to 500.
Bituminous	E-Values are taken based on Existing Pavement Condition As per IRC 115-2014 given below. Bitumen Layer – Distressed Condition (Fair to Poor) 400 to 1500. Bitumen Layer - Thick Layer without Cracking 750 to 3000 (Current project road)

Table Error! No text of specified style in document.**1: E-Modulus Ranges**

Existing Crust Composition

The current project road existing crust compositions are given below table

S. No	**Location (Km)**	**Direction**	**Pavement Composition (mm)**				**Total Thickness (mm)**
			Bituminous Layer	**Granular Base (WBM)**	**Old BT (mm)**	**Natural GSB / Hard Morrum**	
1	0+200	LHS	75	200		-	275
2	2+200	LHS	50	100		125	275
3	4+800	LHS	15	150		-	165
4	6+400	LHS	50	150		-	200
5	8+400	LHS	50	250		-	300
6	10+400	LHS	50	150		-	200
7	12+400	LHS	50	100		150	300
8	14+400	LHS	50	100		-	150
Average			**48.8**	**150.0**		**137.5**	**233.1**

Table 12 : Existing Pavement Composition

Chainage (Km)	Length (Km)	Bituminous Moduli (Mpa)	Granular Moduli (Mpa)	Sub Grade Moduli (Mpa)	Fatigue Strain	Rutting Strain	Remaining Life(MSA)
0.000-2.000	2.000	1703.75	169.95	86.60	0.0004502	0.0009555	2.04
2.000-4.000	2.000	1451.59	138.01	86.65	0.0005782	0.0010910	1.12
4.000-6.000	2.000	1464.06	130.25	86.65	0.0005523	0.0009437	1.68
6.000-8.000	2.000	1445.87	155.00	86.65	0.0005422	0.0014590	0.30
8.000-10.000	2.000	1490.30	164.74	86.65	0.0004930	0.0009623	1.98
10.000-14.800	2.800	1480.00	162.50	86.65	0.0005203	0.0014430	0.32

Table Error! No text of specified style in document.**3: Remaining Life of Existing Pavement**

Overlay Design

The design of overlays for the existing carriageway pavement has been carried out taking into account the traffic, strength of the existing pavement based on detailed pavement investigation including FWD Testing. The strengthening (overlay) requirements for the existing road pavement have been worked out based on IRC: 115-2014. Below Table gives the details of overlay thicknesses for each road section at respective design traffic.

Chainage (Km)	Length (Km)	Bituminous Moduli (Mpa)	Granular Moduli (Mpa)	Sub Grade Moduli (Mpa)	Overlay Thickness	Fatigue Strain	Rutting Strain	Design Life(MSA)
0.000-2.000	2.000	1703.75	169.95	86.60	50mm	0.0003260	0.0006619	10.80
2.000-4.000	2.000	1451.59	138.01	86.65	90mm	0.0003326	0.0005734	12.15
4.000-6.000	2.000	1464.06	130.25	86.65	70mm	0.0003332	0.0005599	11.98
6.000-8.000	2.000	1445.87	155.00	86.65	100 mm	0.0003075	0.0006507	11.66
8.000-10.000	2.000	1490.30	164.74	86.65	80mm	0.0003260	0.0005625	12.84
10.000-14.800	4.800	1480.00	162.50	86.65	100 mm	0.0003001	0.0006478	11.90

Table 1: Overlay Design of Existing Pavement

The strengthening (overlay) requirements for the existing road pavement have been worked out based on IRC: 115-2014. Road sections for rehabilitation are mentioned in below table.

From Ch(Km)	To Ch (Km)	Length (Km)	BC (mm)	DBM (mm)
0.000	14.820	14.820	40	60

Table 2: Recommended Rehabilitation (Overlay) Strategy of project road

V. CONCLUSIONS AND RECOMMENDATION

5.1 Conclusion for New Flexible Pavement design

OSR-05: Existing Intermediate lane isstructurally poor in condition as well as heavy distresses, so failing under rutting criteria which shows deficiency in granular layers. Hence it is recommended for reconstruction over entire project road with Design Option-III i.e., Design of Bituminous Pavements with Cemented Sub-Base (CTSB) and Granular Base course (WMM)

New Pavement Option					
Crust Composition	BC	DB M	WMM	CTSB	Sub Grade
	40 mm	50m m	150 mm	200 mm	500m m

Table 16: Conclusions of Flexible pavement proposal on Project Road

5.2 Conclusion for Subgrade characteristics

It is evident that majority of existing subgrade soils throughout the project corridor is Gravelly Sand with Clay and Silt. Liquid Limit (LL) ranges 25.2% to maximum upto 29.8% and plasticity Index ranges 11.7% to NP. The test results obtained from Modified Proctor Test (IS: 2720- Part VIII) gave Max. Dry density 2.07 g/cc to 2.17 g/cc and Optimum Moisture Content varies between 9.8% to 11.2%.The MDD achieved here is greater than the minimum required density of 1.75g/cc. The CBR value of the 4 days soaked specimens of existing SG soil ranges from 11.0% to 20.5%.

However, all the measured parameters of Una Region Roads (LL, PI and FSI) are well within the acceptable limits of MoRTH guidelines (50%, 25% and 50% respectively) to use it in road construction.

5.3 Strength Parameters:

From the results of degree of compaction it is evident that there is variance between MDD and FDD, the same is converted in to degree of compaction. The degrees of compaction along the project roads are ranges are 70% to 88% with an average of 81%. It is noticed that the existing sub grade slightly lacks in desired compaction level. Less amount of degree of compaction may also of the reason for settlement and traffic loading over the time.

REFERENCES

[1] Highway Safety Manual
[2] IRC: 37-2018
[3] IRC: 115-2014
[4] IRC: SP:44-2011-Highway Safety Codes