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GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS

(*Fourth Revision*)

INDIAN ROADS CONGRESS

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ABBREVIATIONS AND SYMBOLS

All the abbreviations and symbols are explained in the guidelines wherever they appeared first in the document. Some of the abbreviations and symbols are listed below:

AASHO	-	American Association of State Highway Officials
AASHTO	-	American Association of State Highway and Transportation Officials
ASTM	-	American Society of Testing and Materials
AUSTROADS	-	Association of Australian and New Zealand Road Transport and Traffic Authorities
BBD	-	Benkelman Beam Deflection
BC	-	Bituminous Concrete
BIS	-	Bureau of Indian Standards
BM	-	Bituminous Macadam
CBR	-	California Bearing Ratio
CFD	-	Cumulative Fatigue Damage
csa	-	Cumulative standard axles
CTB/CT	-	Cement Treated Base (includes all types of cement and chemical stabilized bases)
CTSB	-	Cement Treated sub base (includes all types of cement and chemical stabilized sub-bases)
CVPD	-	Commercial Vehicles Per Day
DBM	-	Dense Bituminous Macadam
FWD	-	Falling Weight Deflectometer
GB	-	Granular Base
GDP	-	Gross Domestic Product
GGRB	-	Gap Graded Mix with Rubberized Bitumen
GSB	-	Granular Sub-base
IRC	-	Indian Roads Congress
IS	-	Indian Standard
ITS	-	Indirect Tensile Strength
kN	-	Kilonewton
LCR	-	Layer Coefficient Ratio
MEPDG	-	Mechanistic Empirical Pavement Design Guide
MIF	-	Modulus Improvement Factor
mm	-	Millimetre
MoRTH	-	Ministry of Road Transport & Highways
MPa	-	Mega Pascal
NHAI	-	National Highways Authority of India
RAP	-	Reclaimed Asphalt Pavement
RF	-	Reliability Factor
SAMI	-	Stress Absorbing Membrane Interlayer
SDBC	-	Semi-Dense Bituminous Concrete
SMA	-	Stone Matrix Asphalt
SP	-	Special Publication
SS2	-	Slow Setting-2 Emulsion
UCS	-	Unconfined Compressive Strength

VDF	-	Vehicle Damage Factor
WBM	-	Water Bound Macadam
WMM	-	Wet Mix Macadam
°C	-	Degree Celsius
A	-	Initial traffic
a	-	Radius of circular contact area
C	-	Adjustment factor for fatigue life of bituminous layer
D	-	Lateral distribution factor
E	-	Elastic modulus of CTB material
E_{CTSB}	-	Elastic modulus of cement treated sub bases
F	-	Vehicle Damage Factor (VDF) used in the design traffic estimation equation
h	-	Thickness of the granular layer
Lat	-	Latitude
M_{RGRAN}	-	Resilient modulus of granular layer
M_{Rm}	-	Resilient modulus of the bituminous mix
M_{RS}	-	Resilient modulus of subgrade soil
$M_{RSUPPORT}$	-	Effective resilient modulus of the layer supporting the granular layer
M_{Rup}	-	28-day flexural strength of the cementitious base
msa	-	Million standard axles
$\mu\varepsilon$	-	Microstrain
N	-	Number of standard axle load repetitions which the cement treated material can sustain
n	-	Design life period, in years
N_{DES}	-	Cumulative number of standard axles to be catered for during the design period of 'n' years
N_f	-	Fatigue life of the bituminous layer
N_{fi}	-	Fatigue life of CTB material which is the maximum repetitions of axle load class 'i' the CTB material can sustain
n_i	-	Expected (during the design life period) repetitions of axle load of class 'i'
N_R	-	Subgrade rutting life
P	-	Number of commercial vehicles per day as per last count
p	-	Contact pressure
r	-	Annual growth rate of commercial vehicles in decimal
T_{20}	-	Temperature at a depth of 20 mm of layer
T_{air}	-	Air temperature
V_a	-	Percent volume of air voids in the mix
V_{be}	-	Percent volume of effective bitumen in the mix
x	-	Number of years between the last count and the year of completion of construction
δ	-	Maximum surface deflection
ε_t	-	Horizontal Tensile Strain
ε_v	-	Vertical compressive strain
σ_R	-	Radial stress at the location
σ_T	-	Tangential stress at the location
σ_Z	-	Vertical compressive stress at the location

GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS

1. INTRODUCTION

1.1 The first guidelines for the design of flexible pavements, published in 1970, were based on (i) subgrade (foundation) strength (California Bearing Ratio) and (ii) traffic, in terms of number of commercial vehicles (having a laden weight of 3 tonnes or more) per day. These guidelines were revised in 1984 considering the design traffic in terms of cumulative number of equivalent standard axle load of 80 kN and design charts were provided for design traffic volumes up to 30 million standard axle (msa) repetitions. The 1970 and 1984 versions of the guidelines were based on empirical (experience based) approach.

1.2 The second revision was carried out in 2001 [1] using semi-mechanistic (or mechanistic-empirical) approach based on the results available from R-6[2], R-56[3] and other research schemes of the Ministry of Road Transport and Highways (MoRTH). The mechanistic-empirical performance models for subgrade rutting and bottom-up cracking in the bottom bituminous layer, developed using the results of these research schemes, were used for the design of flexible pavements. FPAVE software, developed for R-56 research scheme for the analysis of linear elastic layered pavement systems, was used for the analysis of pavements and for the development of thickness design charts. Thickness charts were provided for design traffic levels up to 150 msa.

1.3 The third revision of the guidelines was carried out in 2012[4] to facilitate (i) design of bituminous pavements for traffic volumes more than 150 msa (ii) utilization of new types of pavement materials such as bituminous mixes with modified binders, foam/emulsion treated granular or Reclaimed Asphalt Pavement (RAP) material bases and sub-bases and cement treated sub-bases and bases and stabilized subgrades and (iii) utilization of new construction techniques/practices. Recommendations were made for the use of harder grade binders to resist rutting and top-down cracking in the upper bituminous layer and for fatigue resistant bituminous mixes for the bottom bituminous layer. Mechanistic-empirical performance models were given for rutting in subgrade and bottom-up cracking in bituminous layers for two different levels (80% and 90%) of reliability. Fatigue criteria were included for cement treated bases also.

1.4 The fourth (current) revision has been done based on the feedback received on the performance of bituminous pavements in general and that of bituminous layers in particular. Different provisions made in the third revision of the guidelines have been fine-tuned based on the feedback. Some of the salient features of the fourth revision are: (a) recommendation of better performing bituminous mixes and binders for surface and base/binder courses (b) guidelines for selection of appropriate elastic moduli for bituminous mixes used in the surface and other courses (c) recommendation of minimum thicknesses of granular and cement treated sub-bases and bases and bituminous layers from functional requirements (d) generalization of the procedure for the estimation of the effective resilient modulus/CBR of subgrade (e) provision for the use of geo-synthetics and (f) rationalization of the design approach for stage construction.

1.5 The draft of the basic document was prepared by Late Prof. B.B. Pandey of IIT Kharagpur based on the feedback received during the open house discussion on IRC:37-2012 held at the

National Highways Authority of India (NHAI) headquarters on 9th April, 2016 and subsequent comments received from different practicing professionals, experts and the members of the H-2 committee on the field performance of bituminous pavements and on other design and practical issues. The draft was further edited and modified by a sub-committee consisting of Shri A.V. Sinha, Shri R.K. Pandey and Shri Bidur Kant Jha. The draft was deliberated in various meetings of Flexible Pavement, Airfield & Runways Committee (H-2) and was approved in its meeting held on 29th September, 2018. The Highways Specifications and Standards Committee (HSS) in its meeting held on 23rd October, 2018 approved the document and authorized the Convenor to modify the document subject to written comments received and comments offered during the meeting. Executive Committee of IRC approved the document in its meeting held on 27th October, 2018. Thereafter, meeting was convened by the Convenor, H-2 Committee along with members of drafting Sub-group on 17th November, 2018 at New Delhi to incorporate compliance of the comments in the document. This modified draft was further deliberated and approved in a meeting convened by HSS Convenor & DG (RD) & SS, MoRTH on 19th November, 2018 wherein Secretary General, IRC and member of Sub-group were also present. The Council in its meeting held on 22nd November, 2018 at Nagpur considered and approved the document for printing and releasing.

The composition of H-2 Committee is given below:

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Director General (Road Development) & Special Secretary to Govt. of India	(Singh, B.N.), Ministry of Road Transport & Highways
Secretary General, Indian Roads Congress	Nirmal, Sanjay Kumar

2. SCOPE

2.1 The Guidelines shall apply to the design of new flexible pavements and reconstruction of damaged pavements for roads with a design traffic of two million standard axle (msa) load repetitions or more. For the roads with a design traffic of less than 2 msa, IRC:SP:72[5] shall be adopted for pavement design. For rehabilitation of in-service pavements, overlay design shall be done as per Falling Weight Deflectometer (FWD) method (IRC:115)[6] or Benkelman Beam Deflection (BBD) test method (IRC:81)[7].

2.2 Users of the guidelines are expected to use their skills, experience and engineering judgment and take into consideration the local climatic conditions, cost and availability of materials, their durability and past pavement performance in their respective regions for selecting a suitable pavement composition.

2.3 The guidelines may require revision from time to time in the light of future performance data and technological developments. Towards this end, it is suggested that all the organizations intending to use the guidelines should keep a detailed record of the year of construction, subgrade CBR, soil characteristics, pavement composition and specifications, traffic, pavement performance, overlay history, climatic conditions, etc., and provide feedback to the Indian Roads Congress for further revision.

3. DESIGN PRINCIPLES

3.1 While various sections of these guidelines describe the design procedures in detail, these are supplemented by a discussion on the 'principle and approach to design' given in the **Appendix-A** to this document, which needs to be considered as an integral part of the Guidelines. The Annexes (I to III) given to this document intend to elaborate the finer points of design and support the recommendations by different worked out design examples to help the users in familiarizing themselves with different provisions of the guidelines and for arriving at a safe, economical and performing design.

The philosophy of pavement design involves designing pavements for satisfactory functional and structural performance of the pavement during its intended service life period. Roughness caused by variation in surface profile, cracking of layers bound by bituminous or cementitious materials, rutting (permanent or plastic deformation) of unbound/unmodified or partially modified subgrade, granular layers and bituminous layers are the primary indicators of the functional and structural performance of pavements. Performance of the pavement is explained by performance models which are either (a) purely empirical (only based on past experience) or (b) mechanistic-empirical, in which the distresses/performance are explained in terms of mechanistic parameters such as stresses, strains and deflections calculated using a specific theory and as per a specified procedure. Most of the current pavement design methods follow the mechanistic-empirical approach for the design of bituminous pavements. In these methods, for each of the selected structural distresses, a critical mechanistic parameter is identified and controlled to an acceptable (limiting) value in the design process. The limiting values of these critical mechanistic parameters are obtained from the performance models.

3.2 The mechanistic-empirical design approach, which was used in the second and third revisions of IRC:37, is retained in the current revision as well for the design of flexible pavements. The theory selected for the analysis of pavements is 'linear elastic layered theory' in which the pavement is modeled as a multi-layer system. The bottom most layer (foundation or subgrade) is considered to be semi-infinite, and all the upper layers are assumed to be infinite in the horizontal extent and finite in thickness. Elastic modulus, Poisson's ratio and thickness of each layer are the pavement inputs required for calculation of stresses, strains and deflections produced by a load applied at the surface of the pavement. IITPAVE software, which is an updated version of FPAVE developed for MoRTH Research Scheme R-56 "Analytical design of Flexible Pavement"[3], has been used for the analysis of pavements.

3.3 The vertical compressive strain on top of the subgrade is considered in these guidelines to be the critical mechanistic parameter for controlling subgrade rutting. Horizontal tensile strain at the bottom of the bottom bituminous layer is taken as the causative mechanistic parameter which has to be limited to control bottom-up cracking in bituminous layers. Similarly, to ensure that the Cement Treated Bases (CTB) do not fail by fatigue cracking, tensile strain and tensile stress at the bottom of the CTB are considered to be the critical parameters to control.

3.4 Rutting within bituminous layers caused by accumulated plastic (permanent) deformation in these layers due to repeated application of traffic loads is another major distress which occurs in bituminous pavements. High pavement temperatures and heavy loads can cause early development of unacceptable levels of rut depth in bituminous mixes as the stiffness of the bituminous mix reduces at higher temperatures and the proportion of plastic (irrecoverable) deformation out of the total deformation will be larger under higher temperature and heavier loading conditions. Moisture damage of mixes and brittle cracking resulting from excessive age hardening of bitumen in the upper layers are the other major concerns to be taken into consideration. These distresses are considered by integrating the mix design into the structural design by incorporating the mix volumetric parameters into the performance models and by making suitable recommendations about the choice of binder and mix to be used in different layers.

3.5 For the satisfactory performance of bituminous pavements and to ensure that the magnitudes of distresses are within acceptable levels during the service life period, the guidelines recommend that the pavement sections be selected in such a way that they satisfy the limiting stresses and strains prescribed by the performance models adopted in the guidelines for subgrade rutting, bottom-up cracking of bituminous layer and fatigue cracking of cement treated bases. Additional measures have been suggested in the guidelines by way of integrating the mix design parameters that have a significant bearing on the performance of pavements into the design process. It may be noted that the design of the bituminous mix was integrated into the structural design process even in the second revision (2001 version) of IRC:37 as the strain values used in the fatigue and rutting performance models are computed using the elastic moduli of bituminous mixes and other layer materials. Also, the elastic modulus of the bituminous layer appears in the fatigue performance criterion. Suitable recommendations have also been made in the guidelines for (i) fatigue cracking and moisture damage resistant mixes for the bottom (base) bituminous layer (ii) rut and moisture damage resistant bituminous mixes for the intermediate (binder) bituminous layer (if provided) and (iii) rut, moisture damage, fatigue cracking and age resistant surface course and (iv) drainage layer for removal of excess moisture from the interior of the pavement.

3.6 Performance Criteria

The following performance criteria are used in these guidelines for the design of bituminous pavements.

3.6.1 *Subgrade rutting criteria*

An average rut depth of 20 mm or more, measured along the wheel paths, is considered in these guidelines as critical or failure rutting condition. The equivalent number of standard axle load (80 kN) repetitions that can be served by the pavement, before the critical average rut depth of 20 mm or more occurs, is given by equations 3.1 and 3.2 respectively for 80 % and 90 % reliability levels. The rutting performance model developed initially based on the MoRTH R-6 Research Scheme[2] performance data was subsequently developed into two separate models for two different reliability levels based on the additional performance data collected for MoRTH R-56 Research Scheme[3].

$$N_R = 4.1656 \times 10^{-08} [1/\varepsilon_v]^{4.5337} \quad \text{(for 80 % reliability)} \qquad (3.1)$$

$$N_R = 1.4100 \times 10^{-08} [1/\varepsilon_v]^{4.5337} \quad \text{(for 90 % reliability)} \qquad (3.2)$$

Where,

N_R = subgrade rutting life (cumulative equivalent number of 80 kN standard axle loads that can be served by the pavement before the critical rut depth of 20 mm or more occurs)

ε_v = vertical compressive strain at the top of the subgrade calculated using linear elastic layered theory by applying standard axle load at the surface of the selected pavement system

IITPAVE software is used in these guidelines for the analysis of pavements. For the computation of stresses, strains and deflections in the pavement, thicknesses and elastic properties (elastic modulus and Poisson's ratio) of different layers are the main inputs. Detailed instructions for installation and use of IITPAVE software, which is provided along with these guidelines, are given in **Annex I**. Guidelines for the selection of the elastic modulus and Poisson's ratio values of different pavement layers are given in different sections of the guidelines. For the calculation of vertical compressive strain on top of the subgrade, horizontal tensile strain at the bottom of the bottom bituminous layer and the horizontal tensile strain at the bottom of cement treated base (CTB) layer, the analysis is done for a standard axle load of 80 kN (single axle with dual wheels). Only one set of dual wheels, each wheel carrying 20 kN load with the centre to centre spacing of 310 mm between the two wheels, applied at the pavement surface shall be considered for the analysis. The shape of the contact area of the tyre is assumed in the analysis to be circular. The uniform vertical contact stress shall be considered as 0.56 MPa. However, when fatigue damage analysis of Cement Treated Bases (CTB) is carried out, (using Equations 3.5 to 3.7) the contact pressure used for analysis shall be 0.80 MPa. The layer interface condition was assumed to be fully bound. The materials are assumed to be isotropic.

3.6.2 *Fatigue cracking criteria for bituminous layer*

The occurrence of fatigue cracking (appearing as inter connected cracks), whose total area in the section of the road under consideration is 20 % or more than the paved surface area of the section, is considered to be the critical or failure condition. The equivalent number of standard axle (80 kN) load repetitions that can be served by the pavement, before the critical condition of the cracked surface area of 20 % or more occurs, is given by equations 3.3 and 3.4 respectively for 80 % and 90 % reliability levels. The fatigue performance models given by equations 3.3 and 3.4 were developed under MoRTH R-56 scheme[3] utilizing primarily the R-6 scheme (Benkelman Beam Studies) performance data[2] supplemented by the data available from R-19 (Pavement Performance Studies)[8] and R-56 schemes[3].

$$N_f = 1.6064 * C * 10^{-04} \, [1/\varepsilon_t]^{3.89} * [1/M_{Rm}]^{0.854} \text{ (for 80 \% reliability)} \tag{3.3}$$

$$N_f = 0.5161 * C * 10^{-04} \, [1/\varepsilon_t]^{3.89} * [1/M_{Rm}]^{0.854} \text{ (for 90 \% reliability)} \tag{3.4}$$

Where

$C = 10^M$, and $M = 4.84 \left(\frac{V_{be}}{V_a + V_{be}} - 0.69 \right)$

V_a = per cent volume of air void in the mix used in the bottom bituminous layer

V_{be} = per cent volume of effective bitumen in the mix used in the bottom bituminous layer

N_f = fatigue life of bituminous layer (cumulative equivalent number of 80 kN standard axle loads that can be served by the pavement before the critical cracked area of 20 % or more of paved surface area occurs)

ε_t = maximum horizontal tensile strain at the bottom of the bottom bituminous layer (DBM) calculated using linear elastic layered theory by applying standard axle load at the surface of the selected pavement system

M_{Rm} = resilient modulus (MPa) of the bituminous mix used in the bottom bituminous layer, selected as per the recommendations made in these guidelines.

The factor 'C' is an adjustment factor used to account for the effect of variation in the mix volumetric parameters (effective binder volume and air void content) on the fatigue life of bituminous mixes [9] and was incorporated in the fatigue models to integrate the mix design considerations into the fatigue performance model.

A popular approach used for enhancing the fatigue life of bituminous layers is to make the bottom most bituminous mixes richer in bitumen[10]. Larger binder volume in the mix means an increased thickness of the binder film in the mix and an increase in the proportion of bitumen over any cross-section of the layer normal to the direction of tensile strain. Besides having longer fatigue lives, larger binder volumes will also be beneficial in making the mix more moisture damage resistant due to thicker binder films which also reduce the ageing of the binder. Considering that the bottom bituminous layer will be subjected to significantly lower stresses and lower summer temperatures compared to the upper layers, the chance of rutting of the lower layer will be less.

The recent version of the Asphalt Institute manual for mix design[10] recommends design of the bitumen rich mixes (or rich bottom mixes) at 2 to 2.5 per cent air voids and to compact the rich bottom layer to less than 4 per cent in-place air voids. The recommendations made in these guidelines about the volumetric parameters and the in-place air voids to be achieved are given in para 9.2.

3.6.3 *Fatigue performance models for Cement Treated Base (CTB)*

3.6.3.1 In the case of pavements with CTB layer, fatigue performance check for the CTB layer should be carried out as per equation 3.5 (based on cumulative standard axle load repetitions estimated using vehicle damage factors), and as per equations 3.6 and 3.7 (cumulative fatigue damage analysis) using axle load spectrum data. It may be noted that 'cement treated' refers to stabilization by different types of cementitious materials such as cement, lime, fly-ash, or a combination thereof. The terms, 'cement treated' and 'cementitious', have been used interchangeably in these guidelines. Equation 3.5 is based on the Australian experience[11] whereas equation 3.6 is as per the recommendations of the Mechanistic-Empirical Pavement Design Guide[12]. Pavement analysis shall be carried out using IITPAVE with a contact stress of 0.8 MPa on the pavement surface to determine the tensile strain (εt) value at the bottom of the CTB layer. The number of standard axle loads derived from equation 3.5 by substituting the computed tensile strain value along with other inputs shall not be less than the design traffic.

$$N = RF\left[\frac{\left(\frac{113000}{E^{0.804}} + 191\right)}{\varepsilon_t}\right]^{12} \tag{3.5}$$

Where,

RF = reliability factor for cementitious materials for failure against fatigue

= 1 for Expressways, National Highways, Sate Highways and Urban Roads and for other categories of roads if the design traffic is more than 10 msa

= 2 for all other cases

N = number of standard axle load repetitions which the CTB can sustain

E = elastic modulus of CTB material (MPa)

ε_t = tensile strain at the bottom of the CTB layer (microstrain).

3.6.3.2 *Cumulative fatigue damage analysis*

The CTB layer is subjected to cumulative fatigue damage by the application of axle loads of different categories and different magnitudes applied over the design life period. The fatigue life

N_{fi} of the CTB material when subjected to a specific number of applications (n_i) of axle load of class 'i' during the design period, is given by equation 3.6. Details of different types of axles, axle load spectrum, repetitions of each load group expected during the design life period, shall be obtained from the analysis of the axle load survey data.

For the purpose of analysis, each tandem axle repetition may be considered as two repetitions of a single axle carrying 50 % of the tandem axle weight as axles separated by a distance of 1.30 m or more do not have a significant overlapping of stresses. Similarly, one application of a tridem axle may be considered as three single axles, each weighing one third the weight of the tridem axle. For example, if a tridem axle carries a load of 45 tonnes, it may be taken to be equivalent to three passes of a 15 tonne single axle.

For analyzing the pavement for cumulative fatigue damage of the CTB layer, contact stress shall be taken as 0.80 MPa instead of 0.56 MPa.

$$log_{10} N_{fi} = \frac{0.972 - \left(\sigma_t / M_{Rup}\right)}{0.0825} \quad (3.6)$$

Where,

N_{fi} = Fatigue life of CTB material which is the maximum repetitions of axle load class 'i' the CTB material can sustain

σ_t = tensile stress at the bottom of CTB layer for the given axle load class.

M_{Rup} = 28-day flexural strength of the cementitious base

σ_t/M_{Rup} = Stress Ratio

The Cumulative Fatigue Damage (CFD) caused by different repetitions of axle loads of different categories and different magnitudes expected to be applied on the pavement during its design period is estimated using equation 3.7.

$$CFD = \Sigma(n_i/N_{fi}) \quad (3.7)$$

Where,

n_i = expected (during the design life period) repetitions of axle load of class 'i'

N_{fi} = fatigue life or maximum number of load repetitions the CTB layer would sustain if only axle load of class 'i' were to be applied

If the estimated CFD is less than 1.0, the design is considered to be acceptable. If the value of CFD is more than 1.0, the pavement section has to be revised.

3.7 Reliability

These Guidelines recommend 90% reliability performance equations for subgrade rutting (equation 3.2) and fatigue cracking of bottom bituminous layer (equation 3.4) for all important roads such as Expressways, National Highways, State Highways and Urban Roads. For other categories of roads, 90 % reliability is recommended for design traffic of 20 msa or more and 80 per cent reliability for design traffic less than 20 msa.

3.8 Analysis of Flexible Pavements

For computing the stresses, strains and deflections, the pavement has been considered in these guidelines as a linear elastic layered system. IITPAVE software, developed for analysis of linear elastic layered systems, has been used in these guidelines for analysis and design of pavements. Details of the IITPAVE software, which is supplied with this document, are given in **Annex-I**. As mentioned previously in these guidelines, the vertical compressive strain on top of subgrade and the horizontal tensile strain at the bottom of the bituminous layer are considered to be the critical mechanistic parameters which need to be controlled for ensuring satisfactory performance of flexible pavements in terms of subgrade rutting and bottom-up cracking of bituminous layers. Similarly, the horizontal tensile stress and horizontal tensile strain at the bottom of the CTB layer are considered to be critical for the performance of the CTB bases. **Figs. 3.1** to **3.6** show different flexible pavement compositions for which the locations at which different critical mechanistic parameters should be calculated are shown. The critical locations are indicated as dots in the figure. **Table 3.1** presents the standard conditions recommended in these guidelines for the pavement analysis.

Theoretical calculations suggest that the tensile strain near the surface close to the edge of the wheel can be sufficiently large to initiate longitudinal surface cracking followed by transverse cracking much before the flexural cracking of the bottom layer occurs, if the mix tensile strength is not adequate at higher temperatures[13] [14].

Table 3.1 Standard Conditions for Pavement Analysis using IITPAVE

Analysis Conditions	
Material response model	Linear elastic model
Layer interface condition	Fully bonded (all layers)
No. of Wheels	Dual wheel
Wheel loads	20 kN on each single wheel (two wheels)
Contact stress for critical parameter analysis	0.56 MPa for tensile strain in bituminous layer and vertical compressive strain on subgrade; 0.80 MPa for Cement treated base
Critical Mechanistic Parameters	
Bituminous layer	Tensile strain at the bottom
Cement treated base	Tensile stress and tensile strain at the bottom
Subgrade	Compressive strain at the top
Note: (a) Only the absolute values of strains/stresses (without the + or – sign) should be used in the performance equations (b) For pavements with strong bases and/or thin bituminous layers, it is possible that the strain at the bottom of the bituminous layer may be compressive instead of tensile.	

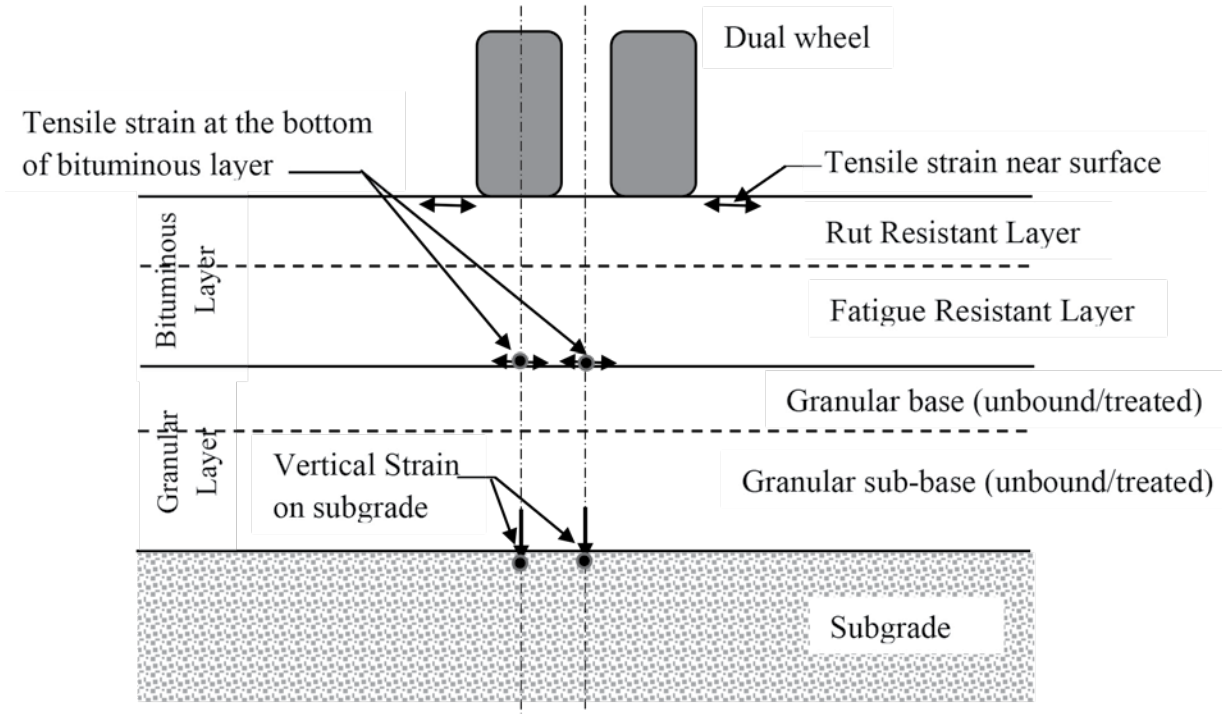

Fig. 3.1 A Pavement Section with Bituminous Layer(s), Granular Base and GSB Showing the Locations of Critical Strains

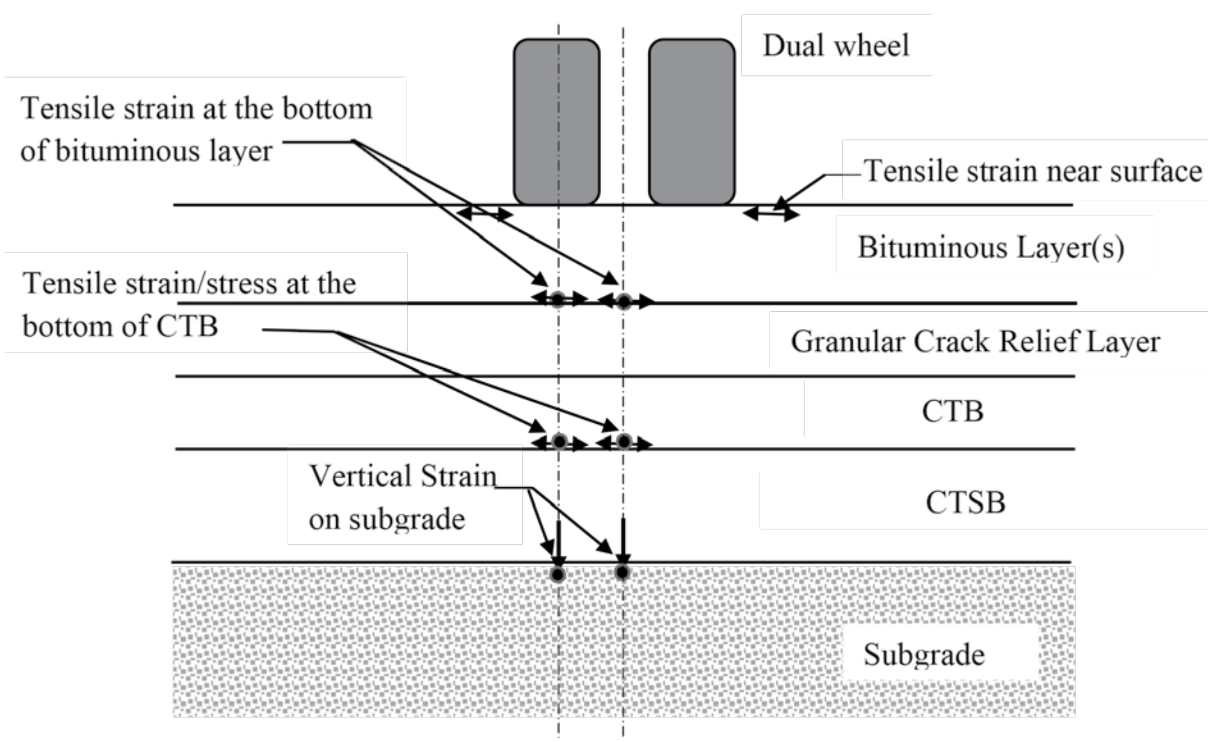

Fig. 3.2 A Pavement Section with Bituminous Layer(s), Granular Crack Relief Layer, CTB, and CTSB Showing the Locations of Critical Strains/Stresses

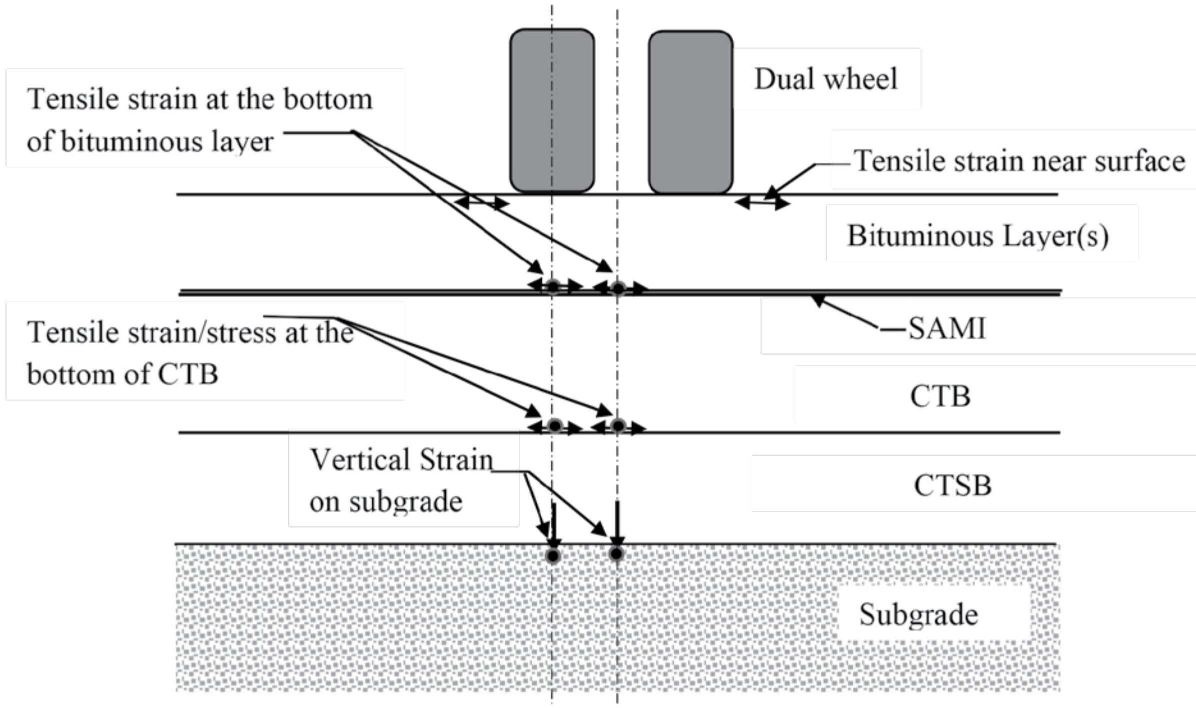

Fig. 3.3 A Pavement Section with Bituminous Layer(s), SAMI Crack Relief Layer, CTB, and CTSB Showing the Locations of Critical Strains/Stresses

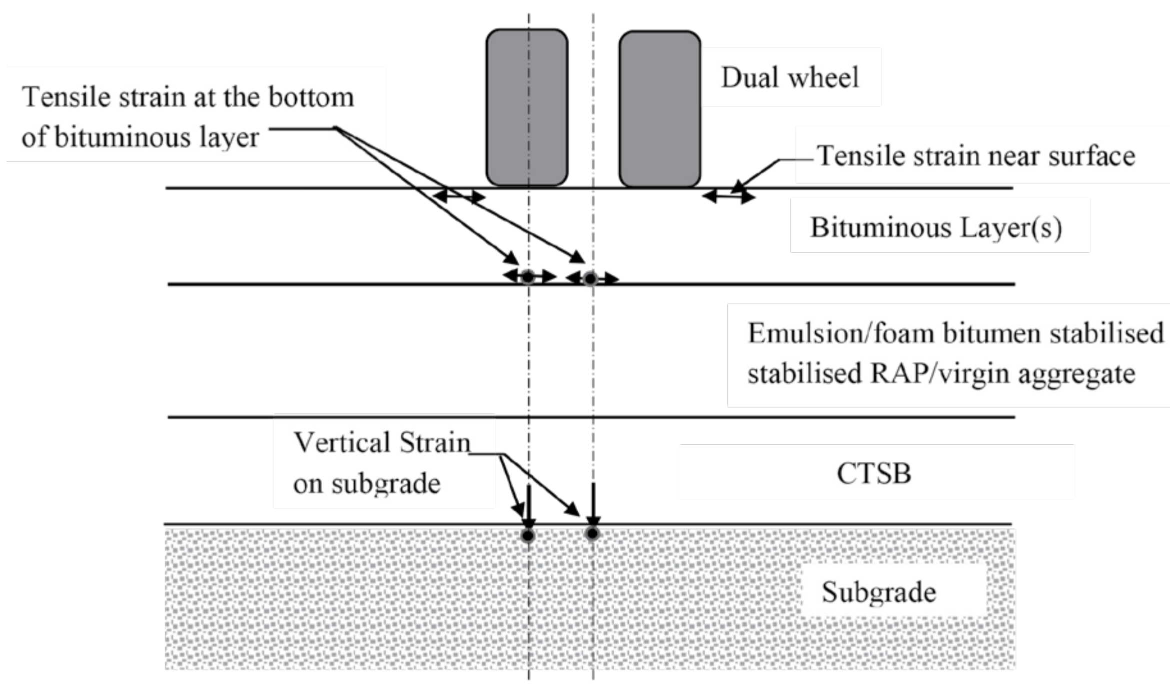

Fig. 3.4 A Pavement Section with Bituminous Layer(s), Emulsion/Foam Bitumen Stabilised RAP/ Virgin Aggregate Layer and CTSB Showing the Locations of Critical Strains

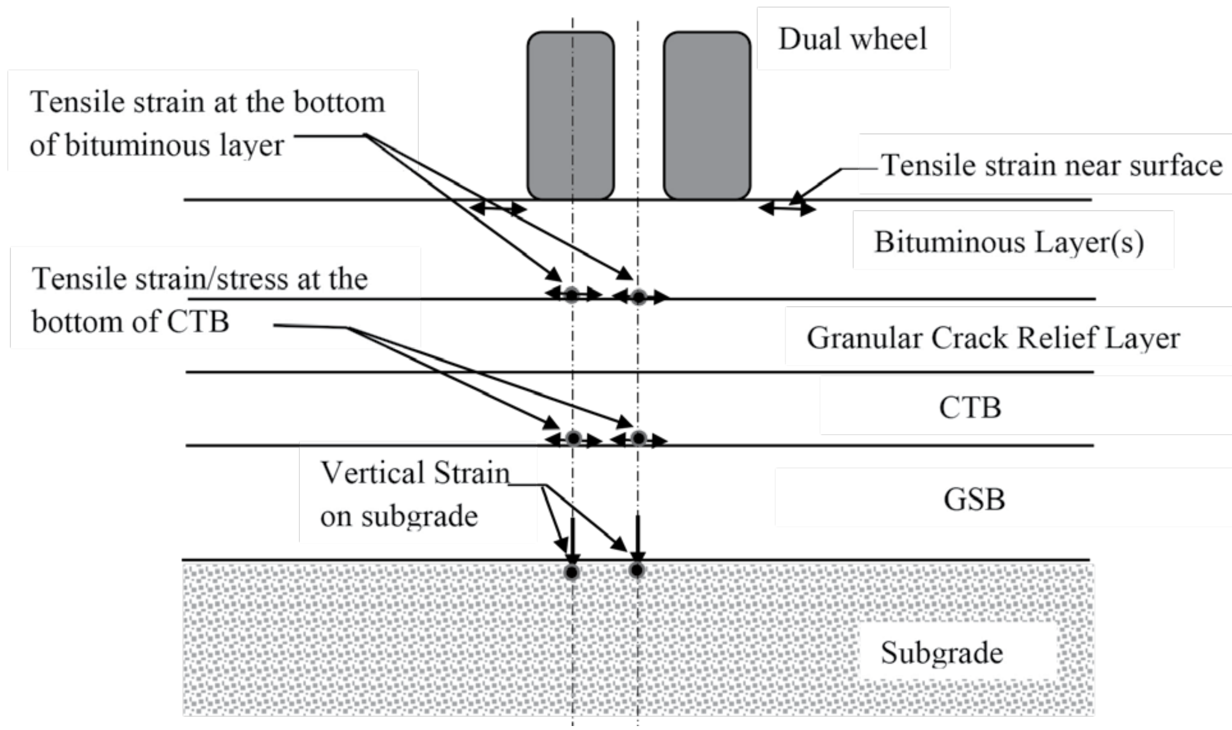

Fig. 3.5 A Pavement Section with Bituminous Layer(s), Granular Crack Relief Layer, CTB, and GSB Showing the Locations of Critical Strains/Stresses

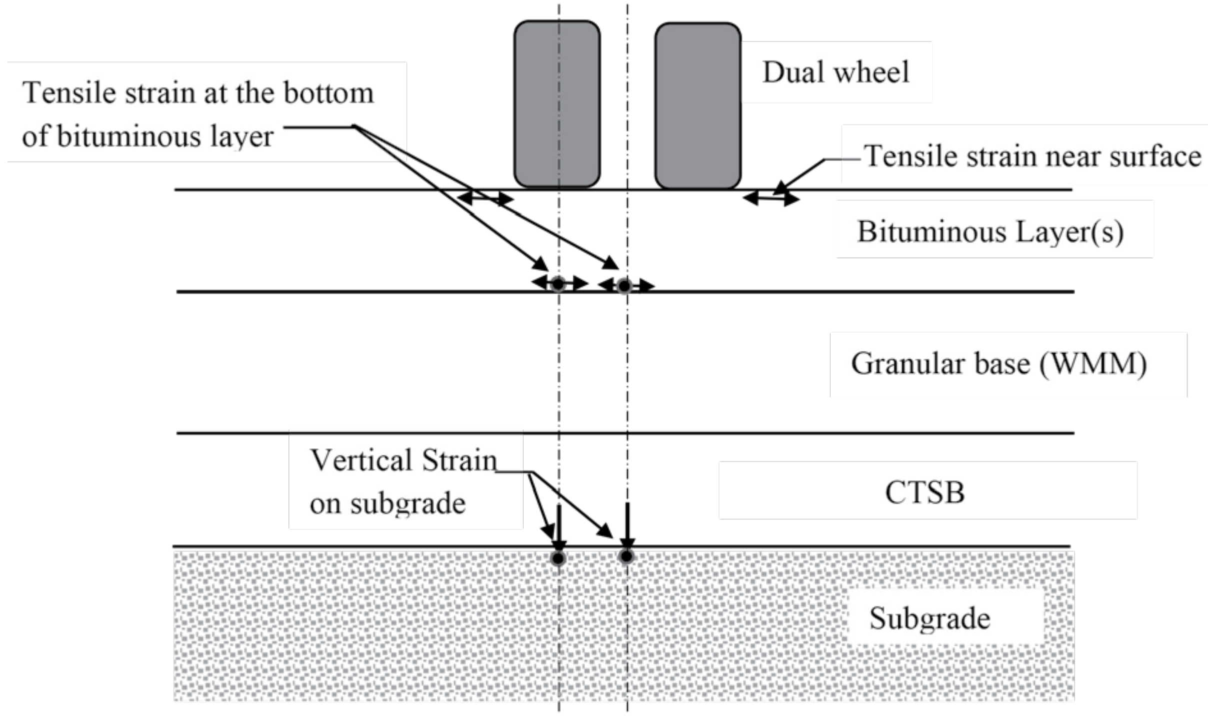

Fig. 3.6 A Pavement Section with Bituminous Layer(s), Granular Base (WMM) and CTSB Showing the Locations of Critical Strains

4. TRAFFIC

4.1 General

This section covers the guidelines for the estimation of design traffic for new roads. The guidelines consider that the structural damage to the pavement i.e., fatigue cracking in the bound layers and rutting in the subgrade is caused by the applied traffic loads. The relative structural damage caused to the pavement by different types of axles carrying different axle loads is considered using Vehicle Damage Factors (VDFs) in the estimation of design traffic.

4.1.1 The design traffic is estimated in these guidelines in terms of equivalent number of cumulative standard axles (80 kN single axle with dual wheels). For estimating the factors required to convert the commercial traffic volumes into equivalent repetitions of the standard axle, it is necessary to measure the axle load spectrum relevant for the stretch of road under consideration. Axle load spectrum data are especially required for the design of pavements having layers treated/stabilised using cementitious materials such as cement, lime, fly ash, etc., for estimating the cumulative fatigue damage expected to be caused to the cement treated base by different axle load groups. The following inputs are required for estimating the design traffic (in terms of cumulative standard axle load repetitions) for the selected road for a given design period.

(i) initial traffic (two-way) on the road after construction in terms of the number of commercial vehicles (having the laden weight of 3 tonnes or more) per day (cvpd)

(ii) average traffic growth rate(s) during the design life period

(iii) design life in number of years

(iv) spectrum of axle loads

(v) factors for estimation of the lateral distribution of commercial traffic over the carriageway

4.1.2 Only the commercial vehicles having gross vehicle weight of 3 tonnes or more are considered for the structural design of pavements.

4.1.3 Estimation of the present day average traffic should be based on the seven-day 24-hour traffic volume count made in accordance with IRC:9[15].

4.2 Traffic Growth Rate

4.2.1 For estimating the cumulative traffic expected to use the pavement over the design period, it is necessary to estimate the rate(s) at which the commercial traffic will grow over the design period. The growth rates may be estimated as per IRC:108[16]. Typical data required for estimation of the growth rates(r) are:

(i) past trends of traffic growth and

(ii) demand elasticity of traffic with respect to macro-economic parameters (like the gross domestic product and state domestic product) and the demand expected due to specific developments and land use changes likely to take place during the design life period.

4.2.2 Traffic growth rates shall be established for each category of commercial vehicles. In the absence of data for estimation of the annual growth rate of commercial vehicles or when the estimated growth rate is less than 5 per cent, a minimum annual growth rate of 5 per cent should be used for commercial vehicles for estimating the design traffic.

4.3 Design Period

4.3.1 The design period to be adopted for pavement design is the time span considered appropriate for the road pavement to function without major rehabilitation. It is recommended that a design period of 20 years may be adopted for the structural design of pavements for National Highways, State Highways and Urban Roads. For other categories of roads, a design period of 15 years is recommended. Pavements for very high density corridors (more than 300 msa) and expressways shall preferably be designed as long-life pavements. Otherwise, for such corridors, the pavement shall be designed for a minimum period of 30 years. The commercial traffic, converted into equivalent repetitions of the standard axle, and adjusted for directional distribution, lateral distribution over the carriageway width, etc., is the design traffic.

4.3.2 *Design traffic considerations for stage construction*

Stage construction of pavement may be adopted in projects where the growth of traffic is uncertain or future traffic volumes are expected to increase substantially due to future developments. Stage construction may also be adopted in projects for which subsequent maintenance is mandated on ‘performance basis’. For projects in which stage construction is adopted, the base and sub-base layers shall be designed for the full design period. Stage construction is not allowed for pavements with cement treated bases and sub-bases.

Stage-1 bituminous layer(s) of the pavement should be designed for more traffic than estimated for the initial (first) stage design period (or traffic) so that the pavement will have at least 40 % life remaining after stage-1 period (traffic). Assuming that the pavement life consumed increases linearly with traffic, the design traffic for stage-1 shall be taken as 1.67 times the design traffic estimated for stage-1 period. If designed and constructed for only the stage-1 design traffic, the pavement, especially the bituminous layer, may not have adequate the structural condition and is lightly to develop full depth cracking and thus may not be suitable for periodical maintenance measures such as patching, crack sealing and micro-surfacing. The requirement of the second stage pavement shall be determined after evaluation of the structural condition of the pavement by Falling Weight Deflectometer (FWD) method as per IRC:115[6] or by Benkelman Beam Deflection (BBD) method as per IRC:81[7]. An example of the design carried out following the concept of stage construction is given in **Annex-II**.

4.4 Vehicle Damage Factor

4.4.1 The guidelines use Vehicle Damage Factor (VDF) for the estimation of cumulative repetitions of standard axle load for the thickness design of pavements. In the case of pavements with CTB layer, in addition to the fatigue performance check carried out as per equation 3.5 based on cumulative standard axle load repetitions (estimated using VDF), CFD analysis also should be carried out following the approach given by equations 3.6 and 3.7 using axle load spectrum data.

4.4.2 Vehicle Damage Factor (VDF) is a multiplier to convert the given number of commercial vehicles having different axle configurations and different axle weights into an equivalent number of standard axle load (80 kN single axle with dual wheels) repetitions.

4.4.3 For converting one repetition of a particular type of axle carrying a specific axle load into equivalent repetitions of 80 kN single axle with dual wheel, equations 4.1 to 4.4 may be used. Since the axle load equivalence factors reported from the AASHO Road Test for flexible as well as rigid pavements are not significantly different for heavy duty pavements, it is assumed that the VDF values estimated for checking subgrade rutting and bituminous layer fatigue cracking can be used for checking the fatigue damage of cemented bases also.

Single axle with single wheel on either side

$$\left(\frac{Axle\ load\ in\ kN}{65}\right)^4 \tag{4.1}$$

Single axle with dual wheel on either side

$$\left(\frac{Axle\ load\ in\ kN}{80}\right)^4 \tag{4.2}$$

Tandem axle with dual wheel on either side

$$\left(\frac{Axle\ load\ in\ kN}{148}\right)^4 \tag{4.3}$$

Tridem axle with dual wheel on either side

$$\left(\frac{Axle\ load\ in\ kN}{224}\right)^4 \tag{4.4}$$

Some tandem axles have only one (single) wheel on each side of the axle. In such cases, each axle of the tandem axle set may be considered as two separate single axles (with single wheels) and Equation 4.1 may be used for estimation of the equivalent axle load repetitions. Similarly, if the axle spectrum has a tridem axle with single wheels, it may be considered as three separate single axles having single wheels.

4.4.4 Multi-axle vehicles may consist of different combinations of axle classes considered in equations 4.1 to 4.4. The VDF should be arrived at by carrying out axle load surveys on the existing roads for a minimum period of 24 hours in each direction. The minimum sample size of commercial vehicles to be considered for the axle load survey is given in **Table 4.1**. Care should be taken to ensure that there is no bias in the selection of the vehicles for the survey. The vehicles to be surveyed should be selected randomly irrespective of whether they are loaded or empty. On some sections of roads, there may be a significant difference between the axle loads of commercial vehicles plying in the two directions of traffic. In such situations, the VDF should be evaluated separately for each direction.

Table 4.1 Minimum Sample Size for Axle Load Survey

Commercial Traffic Volume (CVPD)	Min.% of Commercial Traffic to be Surveyed
< 3000	20 per cent
3000 to 6000	15 per cent (subject to a minimum of 600 cvpd)
> 6000	10 per cent (subject to a minimum of 900 cvpd)

4.4.5 *Axle load spectrum*

For the analysis of the axle load spectrum and for calculation of VDFs, the axle load data may be classified into multiple classes with class intervals of 10 kN, 20 kN and 30 kN for single, tandem and tridem axles respectively.

4.4.6 For small projects, in the absence of weigh pad, the axle loads of typical commercial vehicles plying on the road may be estimated approximately from the type of goods carried. Where information on the axle loads is not available and the proportion of heavy vehicles using the road is small, the indicative values of Vehicle Damage Factor given in **Table 4.2** can be used. These indicative VDF values have been worked out based on typical axle load spectrums and taking into consideration the legal axle load limits notified in the Gazette of India dated 16th July, 2018.

Table 4.2 Indicative VDF values

Initial (Two-Way) Traffic Volume in Terms of Commercial Vehicles Per Day	Terrain	
	Rolling/Plain	Hilly
0-150	1.7	0.6
150-1500	3.9	1.7
More than 1500	5.0	2.8

4.5 Lateral Distribution of Commercial Traffic over the Carriageway

4.5.1 *Lateral distribution*

Lateral distribution of commercial traffic on the carriageway is required for estimating the design traffic (equivalent standard axle load applications) to be considered for the structural design of pavement. The following lateral distribution factors may be considered for roads with different types of the carriageway:

4.5.1.1 Single-lane roads

Traffic tends to be more channelized on single-lane roads than on two-lane roads and to allow for this concentration of wheel load repetitions, the design should be based on the total number (sum) of commercial vehicles in both directions.

4.5.1.2 Intermediate lane roads of width 5.50 m

The design traffic should be based on 75 per cent of the two-way commercial traffic.

4.5.1.3 Two-lane two-way roads

The design should be based on 50 per cent of the total number of commercial vehicles in both the directions.

4.5.1.4 Four-lane single carriageway roads

40 per cent of the total number (sum) of commercial vehicles in both directions should be considered for design.

4.5.1.5 Dual carriageway roads

The design of dual two-lane carriageway roads should be based on 75 per cent of the number of commercial vehicles in each direction. For dual three-lane carriageway and dual four-lane carriageway, the distribution factors shall be 60 per cent and 45 per cent respectively.

4.6 Computation of Design Traffic

4.6.1 The design traffic, in terms of the cumulative number of standard axles to be carried during the design period of the road, should be estimated using equation 4.5.

$$N_{Des} = \frac{365 \times [(1+r)^n - 1]}{r} \times \mathrm{A} \times \mathrm{D} \times \mathrm{F} \quad (4.5)$$

Where,

N_{Des} = cumulative number of standard axles to be catered for during the design period of 'n' years

A = initial traffic (commercial vehicles per day) in the year of completion of construction (directional traffic volume to be considered for divided carriageways where as for other categories of the carriageway, two-way traffic volume may be considered for applying the lateral distribution factors)

D = lateral distribution factor (as explained in para 4.5)

F = vehicle damage factor (VDF)

n = design period, in years

r = annual growth rate of commercial vehicles in decimal (e.g., for 6 per cent annual growth rate, r = 0.06). Variation of the rate of growth over different periods of the design period, if available, may be considered for estimating the design traffic

The traffic in the year of completion of construction may be estimated using equation 4.6.

$$A = P(1 + r)^x \quad (4.6)$$

Where,

P = number of commercial vehicles per day as per last count.

x = number of years between the last count and the year of completion of construction.

4.6.2 For single carriageway (undivided) roads, the pavement may be designed for design traffic estimated based on the larger of the two VDF values obtained for the two directions. For divided carriageways, different pavement designs can be adopted for the two directions of traffic depending on the directional distribution of traffic and the corresponding directional VDF values in the two directions.

5. PAVEMENT COMPOSITIONS

A flexible pavement considered in these guidelines essentially consists of three functional layers above the subgrade. These are: sub-base, base and bituminous layers. Detailed discussion on subgrade and each of the pavement layers is presented in the subsequent sections of the guidelines. The sub-base and base layers may be (a) granular, (b) cement treated or (c) a combination of granular and cement treated materials. Base layer can also be a foam bitumen or emulsion treated granular/Reclaimed Asphalt Pavement (RAP) material/combination of RAP and aggregate layer. When CTB is used, a crack relief layer is to be mandatorily provided, either as an aggregate interlayer or as a stress absorbing membrane inter-layer (SAMI). The bituminous layer comprises two different types of materials, categorized as bituminous base and surfacing. If the base bituminous layer is constructed in two layers, these are generally termed as binder and base bituminous layers. Unless specified otherwise, each functional layer can be constructed in one or more layers. The same elastic properties (elastic/resilient modulus and Poisson's ratio) may be considered for all the sub-layers of a functional layer for the analysis of the pavement using linear elastic layered theory (IITPAVE software). Granular sub-base layers (filter and drainage layers) and granular base layer are considered (unless specified otherwise) as a single layer in the analysis of the pavement. Similarly, bituminous base (binder and base) and surfacing course are considered as a single layer. Aggregate (granular) crack relief layer shall be considered as a separate layer in the analysis. The SAMI crack relief treatment (if used) over the CTB layer shall not be considered in the structural analysis.

6. SUBGRADE

6.1 General

On new roads, the aim should be to construct the pavement as far above the water table as economically practicable. The difference between the bottom of subgrade level and the level of water table/high flood level should, generally, not be less than 1.0 m or 0.6 m in case of existing roads which have no history of being overtopped. In water logged areas, where the subgrade is within the zone of capillary saturation, consideration should be given to the installation of suitable capillary cut-off as per IRC:34 at appropriate level underneath the pavement.

The top 500 mm of the prepared foundation layer immediately below the pavement, designated as subgrade, can be made up of in-situ material, select soil, or stabilized soil forming the foundation for the pavement. It should be well compacted to derive optimal strength and to limit the rutting caused due to additional densification of the layer during the service life. It shall be compacted to attain a minimum of 97 per cent of the laboratory maximum dry density obtained corresponding to heavy compaction as per IS:2720 Part-8[17] for Expressways, National Highways, State Highways, Major District Roads and other heavily trafficked roads. When the subgrade is formed using a material which is stronger than the upper 500 mm of embankment soil or when the subgrade itself is prepared in two separate layers with significantly different strengths, the effective combined contribution of the subgrade and the embankment layers has to be considered for design. The principle to be used for the estimation of the effective strength or mechanical property is discussed in para 6.4. As previously mentioned in these guidelines, the elastic/resilient moduli of different pavement layers are the main inputs for the analysis and design of pavements. Since the measurement of resilient modulus of soil requires sophisticated equipment, the same is generally estimated from the California Bearing Ratio (CBR) value of the material. The following sections present the details of the compaction effort and moisture content to be used for preparing the specimens in the laboratory for evaluating the CBR value or resilient modulus value of the soil.

6.2 Selection of Dry Density and Moisture Content for Laboratory Testing of Subgrade Material

6.2.1 The laboratory test conditions should represent the field conditions as closely as possible. Compaction in the field is done at a minimum of 97 per cent of the laboratory maximum density obtained at optimum moisture content. In the field, the subgrade undergoes moisture variation depending on different local conditions such as water table depth, precipitation, soil permeability, drainage conditions and the extent to which the pavement is impermeable to moisture. In high rainfall areas, lateral infiltration through the unpaved shoulder, median, porous and cracked surface may have a significant effect on the subgrade moisture condition. The California Bearing Ratio (CBR) of the subgrade soil, for the design of new pavements and reconstruction, should be determined as per IS:2720 Part-16[18] at the most critical moisture condition likely to occur at the site. The test should be performed on remoulded samples of soils in the laboratory. The pavement thickness should be based on 4-day soaked CBR value of the soil, remoulded at placement density (minimum 97 % of maximum Proctor compaction test density) and optimum moisture content ascertained from the compaction curve. In areas with less than 1000 mm rainfall, four-day soaking may be too severe a condition for a subgrade well protected with thick bituminous layer and the strength of the subgrade soil may be underestimated. If data is available about the seasonal variation of moisture, the moulding moisture content for the CBR test can be selected based on the field data. The test specimens should be prepared by static compaction to obtain the target density.

6.2.2 *Frequency of tests and design value*

If the type of soil used in different stretches of the subgrade varies along the length of the pavement, the CBR value of each type of soil should be the average of at least three specimens prepared using that soil. 90th percentile subgrade CBR value should be adopted for the design of high volume roads such as Expressways, National Highways, State Highways and Urban Roads. For other categories of roads, the design can be done based on the 80th percentile CBR value if the design traffic is less than 20 msa and based on 90th percentile CBR if the design traffic is 20 msa or more.

6.3 Resilient Modulus of the Subgrade

Resilient modulus, which is measured taking into account only the elastic (or resilient) component of the deformation (or strain) of the specimen in a repeated load test is considered to be the appropriate input for linear elastic theory selected in these guidelines for the analysis of flexible pavements. The resilient modulus of soils can be determined in the laboratory by conducting the repeated tri-axial test as per the procedure detailed in AASHTO T307-99[19]. Since these equipment are usually expensive, the following relationships may be used to estimate the resilient modulus of subgrade soil (MRS) from its CBR value[20, 21].

$$M_{RS} = 10.0 * CBR \quad \text{for } CBR \leq 5\,\% \tag{6.1}$$

$$M_{RS} = 17.6 * (CBR)^{0.64} \quad \text{for } CBR > 5\,\% \tag{6.2}$$

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.

6.4 Effective Modulus/CBR for Design

6.4.1 Sometimes, there can be a significant difference between the CBR values of the soils used in the subgrade and in the embankment layer below the subgrade. Alternatively, the 500 mm thick subgrade may be laid in two layers, each layer material having different CBR value. In such cases, the design should be based on the effective modulus/CBR value of a single layer subgrade which is equivalent to the combination of the subgrade layer(s) and embankment layer. The effective modulus/CBR value may be determined as per the following procedure which is a generalization of the approach presented earlier in an Indian Roads Congress publication[22].

(i) Using IITPAVE software, determine the maximum surface deflection (δ) due to a single wheel load of 40,000 N and a contact pressure of 0.56 MPa for a two or three layer elastic system comprising of a single (or two sub-layers) of the 500 mm thick subgrade layer over the semi-infinite embankment layer. The elastic moduli of subgrade and embankment soils/layers may be estimated from equations 6.1 and 6.2 using their laboratory CBR values. Poisson's ratio (μ) value may be taken as 0.35 for all the layers.

(ii) Using the maximum surface deflection (δ) computed in step (i) above, estimate the resilient modulus M_{RS} of the equivalent single layer using equation 6.3.

$$M_{RS} = \frac{2(1-\mu^2)pa}{\delta} \tag{6.3}$$

Where,

p = contact pressure = 0.56 MPa

a = radius of circular contact area, which can be calculated using the load applied (40,000 N) and the contact pressure '*p*' (0.56 MPa) = 150.8 mm

μ = Poisson's ratio

It is the effective resilient modulus (M_{RS}) value and not the CBR that is used in the design. However, if required, the CBR value can be reported using equations 6.2 and 6.3. A worked out example for the estimation of the effective resilient modulus/CBR is given in **Annex-II**.

In case the borrow material is placed over a rocky foundation, the effective CBR may be larger than the CBR of the borrow material. However, only the CBR of the borrow material shall be adopted for the pavement design. Additionally, proper safeguards should be taken against the development of pore water pressure between the rocky foundation and the borrow material.

If the embankment consists of multiple layers of materials having different CBR values, multi-layer analysis can be carried out using IITPAVE software and the effective resilient modulus can be estimated using the concept discussed above.

6.4.2 For the purpose of design, the resilient modulus (M_{RS}), thus estimated, shall be limited to a maximum value of 100 MPa.

6.4.3 The effective subgrade CBR should be more than 5 % for roads estimated to carry more than 450 Commercial Vehicles Per Day (CVPD) (two-way) in the year of construction.

7. SUB-BASES

7.1 General

The sub-base layer serves three functions: (i) to provide a strong support for the compaction of the granular base (WMM/WBM) layer (ii) to protect the subgrade from overstressing and (iii) to serve as drainage and filter layers. The sub-base layers can be made of granular material which can be unbound or chemically stabilized with additives such as cement, lime, flyash and other cementitious stabilizers. The thickness of the sub-base, whether bound or unbound, should meet these functional requirements. To meet these requirements, minimum sub-base thicknesses have been specified in the following paragraphs.

7.2 Granular (Unbound) Sub-Base Layer

7.2.1 Sub-base materials may consist of natural sand, moorum, gravel, laterite, kankar, brick metal, crushed stone, crushed slag, reclaimed crushed concrete/reclaimed asphalt pavement, river bed material or combinations thereof meeting the prescribed grading and physical requirements. When the granular sub-base material consists of a combination of different materials, mixing should be done mechanically by either using a suitable mixer or adopting the mix-in-place method. Granular sub-base (GSB) should conform to the MoRTH Specifications for Road and Bridge Works[23].

If the thickness of the sub-base layer provided in the design permits, the sub-base layer shall have two sub layers; drainage layer and the filter layer. The upper layer of the sub-base functions as a drainage layer to drain away the water that enters through surface cracks. The lower layer of the sub-base should function as the filter/separation layer to prevent intrusion of subgrade soil into the pavement. The aggregate gradations recommended for the drainage layer are granular sub-base gradations III and IV of MoRTH Specifications[23]. The gradations I, II, V and VI specified for GSB by MoRTH[23] are recommended for filter/separation layer.

If the design thickness of the granular sub-base is less than or equal to 200 mm, both drainage and filter layers cannot be provided separately (considering the minimum thickness requirements given in 7.2.2). For such cases, a single drainage-cum-filter layer with GSB gradation V or VI of MoRTH Specifications may be provided.

The filter and drainage layers should be designed as per IRC:SP:42[24] and IRC:SP:50[25]. It is necessary to extend both drainage and filter layers to full width up to the slope of the embankment to have efficient drainage. Commercially available synthetic geo-composite, grid lock geo-cell with perforated vertical faces filled with aggregates meeting the requirement as specified in IRC:SP:59[26] can also be used to function as both filter/separation and drainage layers. Its strengthening effect can be considered in the pavement design in accordance with the provisions of IRC:SP:59.

When GSB layer is also provided below the median in continuation with that of the pavement, a non-woven geo-synthetic may be provided over the GSB in the median part so that the fines percolating through the median do not enter into the GSB and choke it.

7.2.2 *Minimum thicknesses of granular sub-base layers*

Irrespective of the design traffic volume, the following minimum thicknesses of granular sub-base layers may be provided.

(i) The minimum thickness of drainage as well as filter layer shall be 100 mm (i.e., minimum thickness of each of these two layers is 100 mm).

(ii) The minimum thickness of the single filter-cum-drainage layer shall be 150 mm from functional requirement.

(iii) The minimum thickness of any compacted granular layer should preferably be at least 2.5 times the nominal maximum size of aggregates subject to a minimum of 100 mm.

(iv) The total thickness of the granular sub-base layer should be adequate to carry the construction traffic that may ply on the GSB. This thickness requirement may be worked out to satisfy the subgrade rutting limiting strain criterion given by equation 3.1 or 3.2 (as applicable for the classification of highway and design traffic). The design traffic for this purpose can be worked out based on the expected number of operations of dumpers and other construction vehicles on the GSB layer to carry material for the construction of granular or cement treated base layer. The indicative values of construction traffic loading and the procedure given in the worked out example in **Annex-II** can be used for the estimation of the construction traffic operating over the granular sub-base if more accurate and practical estimation cannot be done.

(v) The sub-base thickness should be checked for the design traffic worked out as per the above mentioned procedure or 10,000 standard axle repetitions, whichever is more.

(vi) The two-layer system (subgrade and GSB) should be analyzed by placing a standard load over it (dual wheel set of 20,000 N each acting at 0.56 MPa contact pressure) and computing (using IITPAVE) the maximum subgrade vertical compressive strain. The GSB thickness should be varied until the computed strain is less than or equal to the limiting subgrade vertical compressive strain, given by equations 3.1 or 3.2 (as applicable).

The worked out example given in **Annex-II** illustrates the estimation of GSB thickness from construction traffic consideration.

7.2.3 *Resilient modulus of GSB layer*

The elastic/resilient modulus value of the granular layer is dependent on the resilient modulus value of the foundation or supporting layer on which it rests and the thickness of the granular layer. A weaker support does not permit higher modulus of the upper granular layer because of the larger deflections caused by loads result in de-compaction in the lower part of the granular layer. Equation 7.1[20] may be used for the estimation of the modulus of the granular layer from its thickness and the modulus value of the supporting layer.

$$M_{RGRAN} = 0.2(h)^{0.45} * M_{RSUPPORT} \quad (7.1)$$

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)

As stated previously in these guidelines, the granular base and granular sub-base are considered as a single layer for the purpose of analysis and a single modulus value is assigned to the combined layer. Thus, when the pavement has the combination of granular base and granular sub-base, the modulus of the single (combined) granular layer may be estimated using equation 7.1 taking the M_{RGRAN} as the modulus of the combined granular layer and $M_{RSUPPORT}$ as the effective modulus of the subgrade. However, when a cement treated or emulsion/foam bitumen treated base layer is used over the granular sub-base, both the layers have to be considered separately in the analysis and separate modulus values have to be assigned for the GSB and the treated base layers. Equation 7.1 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.

For the granular layers reinforced using geo-synthetic materials, IRC:SP:59[26] suggests Layer Coefficient Ratios (LCR) and Modulus Improvement Factors (MIF) which can be used to estimate the improvement in the modulus value of the granular layer due to geo-synthetic reinforcement. These values are to be obtained from detailed field and laboratory investigations as discussed in IRC:SP:59. IRC:SP:59 suggests the estimation of the moduli values of the un-reinforced granular base and sub-base layers separately and to obtain the moduli of the reinforced granular (sub-base and base) layers by applying suitable modification factors. The un-reinforced GSB modulus value estimated from equation 7.1 can be adjusted using appropriate LCR and MIF factors for obtaining the modulus value of the reinforced GSB.

Poisson's ratio of the granular sub-base may be taken as 0.35.

7.3 Cementitious (Cement Treated) Sub-base (CTSB) Layer

7.3.1 *General*

The material used for cementitious (cement treated) sub-base may consist of soil, river bed materials, natural gravel aggregates, reclaimed concrete aggregates, crushed aggregates or soil aggregate mixture modified with different cementitious materials such as cement, lime, lime-flyash, commercially available stabilizers, etc. The recommended aggregate gradation[27] for the CTSB material is Grading IV of Table 400-1 of MoRTH Specifications.

The terms, 'cementitious' and 'cement treated', are used interchangeably in these guidelines. If the CTSB material, which typically is a coarse/open graded material, is disturbed and shows signs of instability before the next layer is laid, the same may be restored by treating it with cement or bitumen emulsion. If soil stabilized with cementitious material is used as a sub-base, commercially available geo-composites can be used to serve as a drainage cum filter/separation layer.

The recommended minimum thickness for CTSB layer is 200 mm.

7.3.2 *Mechanical properties of CTSB material*

The Elastic Modulus (E) of the CTSB material may be estimated from the Unconfined Compressive Strength (UCS) of the material. The cementitious sub-base (CTSB) should have a 7-day UCS of 1.5 to 3.0 MPa as per IRC:SP:89[28]. Third point loading test flexural modulus E_{CGSB} of 28-day cured CTSB material can be estimated using equation 7.2[11].

$$E_{CTSB} = 1000 * UCS \tag{7.2}$$

Where,

UCS = 28-day unconfined compressive strength (MPa) of the cementitious granular material. It should be ensured that the average laboratory strength value should be more than 1.5 times the required (design) field strength.

E_{CTSB} = Elastic modulus (MPa) of 28-day cured CTSB material

For typical cement treated granular sub-base materials, the E_{CGSB} can vary from 2000 to 6000 MPa. Since the sub-base acts as a platform for the construction vehicles carrying 30 to 35 tonnes of construction material, low strength cemented sub-base would crack under the heavy construction traffic and a design value of 600 MPa is recommended for the analysis and design of pavements with CTSB layers. CTSB with grading IV of IRC:SP:89[28] having strength in the range 0.75-1.5 MPa is not recommended for major highways but it can be used for roads with design traffic less than 10 msa. When the CTSB with UCS in the range of 0.75 to 1.5 MPa is used its modulus value may be taken as 400 MPa as specified in IRC:SP:89 (Part II)[27].

Poisson's ratio value of CTSB layer may be taken as 0.25.

The cemented sub-base shall be cured for a minimum of three days before the construction of the next layer. In case sufficient strength of cementitious sub-base is not achieved as per the requirement of IRC:SP:89[28] in 3 days of curing, 7 days curing shall be done for sub-base before the construction of the upper layer can be started.

8. BASES

8.1 Unbound Base Layer

The unbound base layer consists of wet mix macadam, water bound macadam, crusher run macadam, reclaimed concrete, etc., conforming to MoRTH Specifications[23]. Wet mix macadam may also consist of blast furnace slag mixed with crushed stone meeting the MoRTH Specifications. The thickness of the unbound granular layer shall not be less than 150 mm except for the crack relief layer placed over cement treated base for which the thickness shall be 100 mm.

When both sub-base and the base layers are made up of unbound granular layers, the composite resilient modulus of the granular base can be estimated using equation 7.1 taking M_{RGRAN} as the modulus of the combined (GSB + Granular base) granular layer in MPa, 'h' as the combined thickness (mm) of the granular sub-base and base and $M_{RSUPPORT}$ as the effective modulus (MPa) of the subgrade.

For the granular base placed on CTSB layer, the resilient modulus may be taken as 300 MPa and 350 MPa for natural gravel and crushed rock respectively.

Poisson's ratio of granular bases and sub-bases may be taken as 0.35.

As done in the case of granular sub-base, IRC:SP:59 recommends the adjustment of the un-reinforced granular base modulus using LCR or MIF factors. The modulus value of the un-reinforced granular base (which can then be adjusted using LCR or MIF factors for analyzing the pavement) may be estimated using equation 7.1 taking M_{RGRAN} as the modulus of the granular base and $M_{RSUPPORT}$ as the 'effective modulus of the un-reinforced GSB'. The effective modulus of the un-reinforced sub-base can be estimated in the same manner in which the effective subgrade modulus is estimated. In this case, the two-layer system of (a) granular sub-base of selected thickness (whose modulus is estimated using equation 7.1) and (b) the subgrade with 'effective' modulus, is converted into an equivalent granular sub-base of infinite thickness whose effective modulus is to be determined by multiple trials.

When both granular base and granular sub-base are reinforced, the modulus of the un-reinforced granular base can be estimated using Equation 7.1 taking M_{RGRAN} as the modulus of the granular base and $M_{RSUPPORT}$ as the 'effective modulus of the reinforced granular sub-base' calculated as discussed in the previous section. In this case, the two-layer system of (a) reinforced granular sub-base of selected thickness (whose modulus value has been estimated by adjusting the unreinforced granular sub-base using LCR or MIF factors) and (b) effective subgrade is converted into an equivalent reinforced granular sub-base layer of infinite thickness whose modulus can be determined by multiple trials.

8.2 Cementitious Bases (CTB)

8.2.1 Cemented base layers consist of aggregates, reclaimed asphalt material, crushed slag, crushed concrete aggregates or soil-aggregate mixture stabilized with chemical stabilizers such as cement, lime, lime-fly ash or other commercially available stabilizers which can produce mix of requisite strength. Flexural strength of the cemented base is critical to the satisfactory performance of a bituminous pavement. Cementitious bases shall be prepared by plant mixing or by a mechanized in-situ mixing process. The aggregate gradation for CTB shall be as given in table 400-4 of MoRTH Specifications[23]. The CTB material shall have a minimum Unconfined Compressive Strength (UCS) of 4.5 to 7 MPa as per IRC:SP:89 in 7/28 days. While the conventional cement stabilized material should attain this strength in seven days, granular materials and soil-aggregate mixture stabilized with lime, pozzolanic stabilizers, lime-fly ash etc., should meet the above strength requirement in 28 days since the strength gain in such materials is a slow process. As considered in the case of sub-bases, average laboratory strength values should be 1.5 times the required minimum (design) field strength. The cementitious base material must also meet the durability criteria given in Para 8.2.4.

For the functional requirement, the thickness of cement treated bases shall not be less than 100 mm. The procedure to be followed for the estimation of the thickness of the CTB layer required to cater to the construction traffic has been illustrated in **Annex-II**.

The elastic modulus of cementitious bases depends upon the quality of materials. Low grade aggregates such as moorum and kankar may give lower modulus at lower cement contents. Fine grained soil may require larger quantity of cementitious additive for higher strength and may develop wider cracks upon curing. Equation 7.2 may be used for estimating the elastic modulus of E_{CTB} from the 28-day Unconfined Compressive Strength (UCS) of CTB material.

Poisson's ratio value of CTB material may be taken as 0.25.

Strength of cementitious layers keeps on rising with time and an elastic modulus of 5000 MPa may be considered for analysis of pavements with CTB layers having 7/28 day unconfined compression strength values ranging between 4.5 to 7 MPa. While the conventional cement treated layer should attain the above strength in 7 days, lime and lime-flyash stabilised granular materials and soils should achieve the strength in 28 days since the strength gain in such materials is slow (IRC:SP:89).

Curing of cemented bases shall be done for a minimum period of seven days before the commencement of the construction of the next upper layer for achieving the required strength as described in IRC:SP:89[27] and curing should start immediately by spraying bitumen emulsion/ wet jute mat or periodical mist spray of water without flooding or other methods.

8.2.2 *Flexural strength (modulus of rupture) of CTB material*

The modulus of rupture (M_{RUP}) or flexural strength of the CTB material is required for carrying out fatigue damage analysis of the cement treated base. The values of modulus of rupture (MPa) for cementitious bases may be taken as 20 per cent of the 28-day UCS value (MPa)[12] subject to the following limiting (maximum) values:

Cementitious stabilized aggregates	- 1.40 MPa
Lime-flyash-soil	- 1.05 MPa
Soil-cement	- 0.70 MPa

A relationship between UCS, Indirect Tensile Strength (ITS) and Flexural Strength, if developed for the materials being used in a project, will be useful for quality control since ITS is easy to determine on the cores taken from the field. Flexural Strength is approximately 1.5 times the ITS value for cement bound aggregates.

8.2.3 While the minimum size of the sample of the beam for cement stabilized aggregate for flexure test should be 100 mm x 100 mm x 500 mm, the beam size for flexural tests for stabilized soil with hydraulic binders (cement, lime, lime-flyash and other commercially available cementitious binders) can be 50 mm x 50 mm x 250 mm to 75 mm x75 mm x 375 mm. Third point loading shall be applied at a rate of 1.25 mm per minute, which is the same as that used in the CBR test.

8.2.4 *Durability criteria*

The minimum quantity of cementitious material in the bound base layer should be such that in a wetting and drying test (BIS: 4332 Part-IV[29], the loss of weight of the stabilized material does not exceed 14 per cent after 12 cycles of wetting and drying. In cold and snow bound regions like Arunachal Pradesh, Jammu & Kashmir, Ladakh, Himachal Pradesh etc., durability should also be evaluated by freezing and thawing test and the loss of weight should be less than 14 per cent after 12 cycles as per BIS:4332 Part-IV.

8.3 Crack Relief Layer

In case of pavements with CTB, a crack relief layer, provided between the bituminous layer and the cementitious base, delays the reflection of crack from the CTB layer in to the bituminous layer. The crack relief layer may consist of dense graded crushed aggregates of 100 mm thickness conforming to MoRTH[23] Specifications for Wet Mix Macadam (WMM) or the Stress Absorbing Membrane Interlayer (SAMI) of elastomeric modified binder applied at the rate of 10 – 12 kg/10 m^2 covered with 0.1 m^3 of 11.2 mm aggregates. For the pavement analysis, the

SAMI layer is not considered as a structural layer, i.e., it shall not be included in the pavement composition for pavement analysis.

The resilient modulus of a well-graded granular layer depends upon the gradation and the confinement pressure to which it is subjected to under the application of wheel load. A typical value of 450 MPa[30, 31] is used for the sandwiched aggregate layer for the analysis of pavement. The granular crack relief layer shall be compacted to 100% of the modified Proctor compaction maximum density.

Poisson's ratio of the granular crack relief layer may be taken as 0.35.

8.4 Bitumen Emulsion/Foamed Bitumen Treated Reclaimed Asphalt Pavement (RAP) Base

Reclaimed Asphalt Pavement (RAP) material with or without virgin aggregates, treated with foamed bitumen or bitumen emulsion can be used as the base layer. The minimum thickness of the emulsion/foam bitumen stabilised RAP layer shall be 100 mm.

The resilient modulus of the material with bitumen emulsion (SS2)/foamed bitumen shall be taken as 800 MPa though values as high as 3000 MPa have also been achieved on tests conducted on 150 mm diameter specimens. VG30 bitumen is recommended for preparation of the foamed bitumen used for stabilizing the RAP/RAP-aggregate material.

Indirect Tensile Strength of 102 mm diameter Marshall specimen of the bitumen emulsion/ foamed bitumen treated material determined as per ASTM:D 6931[32] should have a minimum value of 100 kPa after soaking and 225 kPa in dry condition at a deformation rate of 50 mm/ minute at 25°C[33].

The recommended Poisson's ratio is 0.35.

9. BITUMINOUS LAYERS

9.1 General

A bituminous pavement generally consists of bituminous surfacing course and a bituminous base/binder course. For high traffic volume roads with a design traffic of more than 50 msa, (a) Stone Matrix Asphalt (SMA)[34], (b) Gap Graded Mix with Rubberized Bitumen (GGRB) [35] and (c) Bituminous Concrete (BC) with modified binders, are recommended for surfacing course for durable, ageing resistant and crack resistant surface courses. For the Stone Matrix Asphalt (SMA) mix (recommended for high traffic volume roads) also, use of modified binders is preferred as it is expected that mixes with modified binders will result in longer service life and will be more resistant to ageing. For roads (national highways as well as non-national highways) with design traffic in the range of 20 to 50 msa, and for national highways with less than 20 msa design traffic, BC with VG40 bitumen can be used for the surface course in addition to the aforementioned mixes. For highly stressed areas or roads in high rainfall areas and junction locations, mastic asphalt mix can be used as an alternative surface course.

For non-National Highway roads with less than 20 msa design traffic, besides SMA, GGRB and BC (with modified binders), the mixes recommended for surface course are Bituminous

Concrete, Semi Dense Bituminous Concrete (SDBC), Pre-Mix Carpet (PMC), Mix Seal Surfacing (MSS) and Surface Dressing (SD) with unmodified binders. The thin bituminous layers such as PC, MSS and SD shall not be considered as part of the bituminous layer for analysis of the pavement.

Dense Bituminous Macadam (DBM) mix with VG40 binder and confirming to IRC and MoRTH Specifications, shall be the material used for base/binder courses for roads with 20 msa or more design traffic. Dense Bituminous Macadam (DBM)/Bituminous Macadam (BM) can be used as base/binder courses for roads with design traffic less than 20 msa. These guidelines recommend VG30/VG40 bitumen for design traffic less than 20 msa and VG40 bitumen for the DBM layers for design traffic of 20 msa or more. For expressways and national highways, even if the design traffic is less than 20 msa, VG40 or modified bitumen shall be used for surface course and VG40 bitumen shall be used for the DBM.

In view of the overlap in the viscosity ranges specified in IS:73[36] for VG30 and VG40 bitumen, it is recommended that the VG40 bitumen used in the surface, binder and base bituminous courses shall have a minimum viscosity of 3600 Poise at 60°C temperature to safeguard against rutting. 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).

If the total thickness of the bituminous layers is less than 40 mm, VG30 bitumen may be used for the BC/SDBC layers even if VG40 bitumen may be more appropriate from pavement temperature consideration. Thin pavements will deflect more under the traffic loads and stiffer VG40 mixes may not have adequate flexibility to undergo such large deflections. The summary of bituminous mixes and binders recommended in the present guidelines is presented in **Table 9.1**.

Table 9.1 Summary of Bituminous Layer Options Recommended in these Guidelines

S. No.	Traffic Level	Surface course		Base/Binder Course	
		Mix type	Bitumen type	Mix type	Bitumen type
1	>50 msa	SMA	Modified bitumen or VG40	DBM	VG40
		GGRB	Crumb rubber modified bitumen		
		BC	With modified bitumen		
2	20-50 msa	SMA	Modified bitumen or VG40	DBM	VG40
		GGRB	Crumb rubber modified bitumen		
		BC	Modified bitumen or VG40		
3	<20 msa[1]	BC/SDBC/PMC/MSS/ Surface Dressing (besides SMA, GGRB and BC with modified binders)	VG40 or VG30	DBM/BM	VG40 or VG30

[1]For expressways and national highways, even if the design traffic is less than 20 msa, VG40 or modified bitumen shall be used for surface course and VG40 bitumen shall be used for the DBM

Special cases:

- Mastic Asphalt can also be used for roads in high rainfall areas and junction locations
- BC/SDBC with VG30 is recommended if total bituminous layer requirement is less than 40 mm.
- VG10 bitumen may be used in the snow bound locations.

9.2 Resilient Modulus of Bituminous Mixes

Resilient modulus of bituminous mixes depends upon the grade of binder, frequency/load application time, air voids, shape of aggregate, aggregate gradation, maximum size of the aggregate, bitumen content, etc. Indicative maximum values of the resilient moduli of different bituminous mixes with different binders are given in **Table 9.2** for reference.

The modulus values given in **Table 9.2** are based on a number of laboratory tests conducted on bituminous mix specimens as per ASTM:4123[37] upgraded now to ASTM: D7369-09[38]. ASTM: D7369-09 essentially retains most of the features of ASTM 4123 but recommends that Poisson's ratio also be measured. ASTM:4123 permits the use of assumed Poisson's ratio values. These guidelines recommend measurement of the resilient modulus at a temperature of 35°C as per ASTM:4123[37] with an assumed Poisson's ratio value of 0.35. A loading pulse of 0.1 second duration followed by a rest period of 0.9 second is adopted. Bituminous mixes undergo reduction in air void content, harden with time and the modulus value will increase due to ageing effect and the actual modulus values could be more than those given in **Table 9.2**. For the measurement of the resilient modulus of DBM, 150 mm diameter specimens should be used because of the larger size of aggregates used in the DBM mixes.

The modulus value of bituminous mixes prepared with modified bitumen varies widely depending upon the modifier, duration of blending, quantity of admixtures and the extent of air blowing of the base bitumen. These mixes may have lower resilient modulus value than those of the mixes prepared with unmodified bitumen. The lower resilient modulus values of mixes with modified binders are due to the larger proportion of elastic/resilient deformation/strain possible with modified mixes. The smaller resilient modulus values do not necessarily indicate that modified binder mixes will have inferior performance compared to unmodified mixes. In fact, mixes with modified binders are, in general, expected to have better fatigue and rutting performance and durability compared to conventional mixes.

As mentioned previously in these guidelines, all the bituminous layers in the pavement shall be considered as one layer in the analysis of the pavement and will be assigned the same elastic properties (elastic/resilient modulus and Poisson's ratio). Considering the possibility that the resilient moduli of Stone Matrix Asphalt[34], GGRB[35] and BC mixes with modified binders will be less than those obtained for dense graded mixes (BC and DBM) prepared with unmodified binders such as VG40, and taking into consideration that these surface mixes are expected to give much better performance than the conventional dense graded mixes with unmodified binders, these guidelines recommend that the bituminous layer (combination of all the bituminous layers) shall be assigned the modulus value of the DBM mix (bottom DBM mix if two DBM layers are used) for analysis and design.

The design of pavement shall be carried out based on the actual values obtained with field designed DBM/BM mix subject to the maximum values indicated in **Table 9.2** for the selected mix (DBM/BM mixes with selected unmodified binder) for an average annual pavement temperature of 35°C. For the climatic conditions prevailing in the plains of India, the Average Annual Pavement Temperature is expected to be close to 35°C. If the resilient modulus value of the specimens prepared using the field bottom (base) bituminous mix is more than the corresponding maximum value indicated in **Table 9.2** for 35°C, the value given in the table shall be used for the analysis and design.

Modified binders are not recommended for the DBM layers due to the concern about the recyclability of DBM layers with modified binders.

Table 9.2 Indicative Values of Resilient Modulus (MPa) of Bituminous Mixes

Mix type	**Average Annual Pavement Temperature °C**				
	20	**25**	**30**	**35**	**40**
BC and DBM for VG10 bitumen	2300	2000	1450	1000	800
BC and DBM for VG30 bitumen	3500	3000	2500	2000	1250
BC and DBM for VG40 bitumen	6000	5000	4000	3000	2000
BC with Modified Bitumen (IRC:SP:53)	5700	3800	2400	1600	1300
BM with VG10 bitumen	500 MPa at 35°C				
BM with VG30 bitumen	700 MPa at 35°C				
RAP treated with 4 per cent bitumen emulsion/ foamed bitumen with 2-2.5 per cent residual bitumen and 1.0 per cent cementitious material.	800 MPa at 35°C				

Note: For the purpose of the design

a) Resilient modulus shall be measured at 35°C temperature as per ASTM 4123.

   For snowbound areas resilient modulus shall be measured at 20°C. The design modulus shall be the smaller of the measured modulus and the value indicated in the table.

b) The same indicative maximum modulus values are recommended for BC (surface course) as well as DBM (binder/base course) with unmodified binders.

c) The resilient modulus values for surfacing courses with modified bitumen shall be taken to be the same as the resilient modulus values indicated for DBM.

The following empirical relationships (equations 9.1 and 9.2) between resilient modulus and indirect tensile strength test of different bituminous mixes and are recommended for arriving at a reasonable estimation of the resilient modulus value. The user agencies are encouraged to provide feedback for their further refinement. Also refer to Clause 14.

Resilient Modulus of 150 mm diameter DBM specimens at 35°C

$$M_r = 11.088 \times ITS - 3015.80 \qquad (9.1)$$

$$(R^2 = 0.68)$$

Resilient Modulus of 102 mm diameter specimens with elastomeric polymer modified binder mixes at 35°C

$$M_r = 1.1991 \times ITS + 1170 \qquad (9.2)$$

$$(R^2 = 0.89)$$

Where,

ITS = Indirect Tensile Strength in kPa,

M_r = Resilient Modulus in MPa

A Poisson's ratio value of 0.35 is recommended for the bituminous layer for analysis of the pavement.

The DBM layer may be constructed in a single layer or in two layers depending upon the design thickness requirement. When only one layer of DBM is used, DBM-1/DBM-2 (selected depending on the thickness of the layer) may be constructed with a suitable surface course. When the DBM is laid in two layers, the sequence of bituminous layers from the bottom to top is: DBM-1, DBM-2 and a suitable surface course.

For longer life of bituminous pavements, to avoid moisture induced distresses and for better bottom-up fatigue resistance, bitumen rich DBM bottom layer is recommended in these guidelines. The rich bottom mixes are typically designed to have more binder volume by selecting lower design air void content which yields more design binder content than normal. It is also a common practice to compact the rich bottom bituminous mixes to smaller in-place air voids. The increased compaction adopted for these mixes will result in mixes with good aggregate interlocking and will make the mixes stiffer. The increased compaction will also reduce the mix rutting that might be produced in the mix by secondary compaction under traffic load stresses. In view of the increased binder requirement, the upper limit of the voids filled with the bitumen (VFB) criterion for such bottom rich mixes designed as per the following criteria, shall be 80%.

For the single layer DBM, the recommended target air void content for mix design is 3.5%. The 3.5% air void content and the corresponding volume of effective binder content shall be used for calculating the fatigue life using equations 3.3 and 3.4. It shall be compacted to 4.5% or lower air void content in the field.

For the bottom DBM layer of two-layer DBM construction, the recommended target air void content for mix design of the bottom DBM layer is 3.0%. The 3.0% air void content and the corresponding volume of effective binder content shall be used for calculating the fatigue life using equations 3.3 and 3.4. The mix shall be compacted to an air void content of 4% or lower in the field.

In case the parameters indicated above for rich bottom mixes are not achievable for any reason, the actual air void content achieved and the corresponding effective binder volume are to be used for estimation of fatigue life.

The recommendations for mix design and field compaction of the other bituminous layers (upper DBM layer of a two-layer DBM system and surface layer) shall be as per the prevailing applicable guidelines.

While it is easy to achieve a target air void content (V_a) during mix design, the corresponding effective binder content at which the V_a is achieved (for a given laboratory compaction effort) may vary widely depending on the variations in aggregate gradation, specific gravity of aggregates and the absorption of bitumen by aggregates. If water absorption of aggregates is 2.0 %, percentage of bitumen absorption into the surface pores of aggregates can vary from 0.8% to 1.6% (by weight of aggregates). Hence V_a and V_{be} must be correctly estimated as per Asphalt Institute Mix Design, MS-2[10].

The minimum thicknesses of different bituminous layers shall be as per relevant MoRTH and IRC specifications. In the case of pavements with Cement Treated Bases (CTB) for traffic exceeding 20 msa, the combined total thickness of surface course and base/binder course shall not be less than 100 mm irrespective of the actual thickness requirement obtained from structural consideration.

10. LONG-LIFE PAVEMENTS

A pavement having a life of fifty years or longer is generally termed as a long-life pavement or perpetual pavement. In the Indian context, pavements with a design traffic of 300 msa or more may be designed as long-life pavements. As per Asphalt Institute, MS-4, 7th edition[39], if the tensile strain caused by the traffic in the bituminous layer is less than 70 microstrain (considered to be the endurance limit of the material), the bituminous layer will never crack. Similarly, if the vertical subgrade strain is less than 200 microstrain, there will be practically very little rutting in the subgrade. For the climatic conditions prevailing in the plains of India, where the Average Annual Pavement Temperature may be close to 35°C, the corresponding limiting strains may be taken as 80 and 200 microstrains respectively. Thus, long-life pavement design involves selecting a suitable pavement layer combination which can keep the horizontal tensile strain and vertical compressive strain limited to the afore-mentioned limiting strain values corresponding to endurance condition. Different layers of the long life pavement have to be designed and constructed in such a way that only the surface course would need replacement from time to time. A design example is given in **Annex-II**.

11. PAVEMENT DESIGN PROCEDURE

11.1 Steps involved in the Pavement Design

11.1.1 *Selecting a trial composition*

In selecting the pavement composition, the designer should be guided by the expected functional requirements of the layers in a high performing pavement, such as a strong subgrade, a well-drained sub-base strong enough to withstand the construction traffic loads, a strong crack, rutting and moisture damage resistant bituminous base and a bituminous surfacing that is resistant to rutting, top-down cracking and to damages caused by exposure to environment.

11.1.2 *Bituminous mix design and the mix resilient modulus*

Sourcing of the material ingredients for the mix has to be decided and the physical requirements and properties of the sourced materials should be checked for their conformity with the provisions of applicable Specifications and these Guidelines. The right proportioning of the mix ingredients or the design mix should be arrived at by trials and testing. Where the resilient modulus is required to be tested in accordance with the procedures recommended in these Guidelines, the samples of the design mix should be appropriately tested as specified. Where the resilient modulus is required to be derived indirectly by using empirical equations given in these Guidelines or are to be adopted as per a certain recommended value, the modulus should be selected/determined accordingly and used for design subject to the compliance with the conditions specified in these Guidelines. In case the resilient modulus determined in this manner exceeds the limiting values specified in these Guidelines, the latter value has to be adopted. In case, it is less than the limiting value, the actual value should be adopted in the design.

11.1.3 *Selecting layer thickness*

The selection of trial thicknesses of various layers constituting the pavement should be based on the designers' experience and subject to the minimum thicknesses recommended in these Guidelines and in other relevant Specifications (when there is no specific recommendation in these guidelines) from functional and constructability considerations.

11.1.4 *Structural analysis of the selected pavement structure*

This is to be done by running the IITPAVE software or any other linear elastic layer programme using as inputs the layer thicknesses, the layer moduli, the layer Poisson's ratio values, the standard axle load of 80 kN distributed on four wheels (20 kN on each wheel), and a tyre pressure as 0.56 MPa. For carrying out fatigue damage analysis of cement treated bases, the axle load under consideration and a contact pressure of 0.80 MPa will be considered. The program will output the stresses, strains and deflections at selected critical locations in the pavement from which the values of critical mechanistic parameters can be identified for design. A soft copy of the IITPAVE software is attached as part of this document. Details about IITPAVE and instructions for its installation and use are given in **Annex-I**. **Table 11.1** gives the details of different inputs to be considered for the analysis.

11.1.5 *Computing the allowable strains/stresses*

The allowable strains in the bituminous layer and subgrade for the selected design traffic are to be estimated using the fatigue and rutting performance (limiting strain) models given in these guidelines. The inputs to the models are the design period of pavement in terms of cumulative standard axles, the resilient modulus value of the bottom layer bituminous mix, and the volumetric proportions (air voids and effective binder) of the mix. For estimating the limiting tensile strain in the CTB layer, the elastic modulus of the CTB material is an input.

11.1.6 *Doing the iterations*

A few iterations may be required by changing the layer thicknesses until the strains computed by IITPAVE are less than the allowable strains derived from performance models.

11.1.7 *Check for cumulative fatigue damage*

Where cementitious bases are used in the pavement, the cumulative fatigue damage analysis is required to be done as done in the case of rigid pavement design to make sure that the cumulative damage caused by the expected axle load spectrum does not exceed unity.

11.1.8 The minimum thicknesses, as specified in the guidelines, shall be provided to ensure intended functional requirement of the layer.

11.2 The design procedures are explained through illustrative worked out design examples given in **Annex-II** of this document.

11.3 In the case of relatively low traffic volume roads, with design traffic not exceeding 50 msa, and in situations where investigations prior to design are not feasible on account of exigencies, a thickness design catalogue is provided in these Guidelines to help the highway authorities in expeditious project approval and procurement. It needs to be borne in mind that the design assumptions made in the preparation of the catalogues need to be fulfilled in actual execution. In case there are deviations from these assumptions, the design should be revisited following the procedure explained in para 11.1 above.

11.4 For design traffic lower than 2 msa, the recommendations of IRC:SP:72[5] may be used.

11.5 The designer is expected to apply his/her judgment and experience in the choice of pavement materials and layer thickness as a number of options of pavements are suggested in the guidelines.

Table 11.1 Recommended Material Properties for Structural Layers

Material Type	**Elastic/Resilient Modulus (MPa)**	**Poisson's Ratio**
Bituminous layer with VG40 or Modified Bitumen	3000 or tested value (whichever is less)	0.35
Bituminous layer with VG30	2000 or tested value (whichever is less)	0.35
Cement treated base	5000	0.25
Cold recycled base	800	0.35
Granular interlayer	450	0.35
Cement treated sub-base	600	0.25
Unbound granular layers	Use Eq. 7.1	0.35
Unbound granular base over CTSB sub-base	300 for natural gravel 350 for crushed aggregates	0.35 0.35
Subgrade	Use Eq. 6.1 or 6.2	0.35

12 PAVEMENT STRUCTURAL DESIGN CATALOUGES

12.1 The pavement structural catalogues presented in these guidelines for design traffic levels up to 50 msa are intended for initial cost estimation and for guidance only. For all roads with more than 2 msa design traffic, the design shall be carried out using site specific inputs to satisfy the mechanistic-empirical performance models given in these guidelines which may require analysis of different trial pavement sections using IITPAVE software. The individual layer thicknesses shown in the catalogues are only for illustration and the actual optimal requirement of layer thicknesses shall be evolved based on detailed analysis. Practical considerations and durability of the selected layers should always be kept in mind.

12.2 Catalogues have been given for the following six categories of pavements: (a) bituminous surface course with granular base and sub-base (b) bituminous surface course with CTSB, CTB and granular crack relief layer (c) bituminous surface course with CTSB, CTB and SAMI (d) bituminous surface course with CTSB and emulsion/foam bitumen stabilised RAP/ virgin aggregate (e) bituminous surface course with GSB, CTB and granular crack relief layer and (f) bituminous surface course with CTSB and granular base course.

12.3 The catalogues have been developed considering 80% reliability subgrade rutting and fatigue cracking performance models for design traffic up to 20 msa, and using 90% models for higher traffic levels. It may be noted that for expressways, national highways, state highways and urban roads, 90% reliability should be adopted irrespective of the design traffic.

Resilient moduli of 2000 MPa (VG30 binder mix for BC as well as DBM) and 3000 MPa (VG40 binder mix for BC as well as DBM) were considered for less than 20 msa and 20 to 50 msa categories respectively. It may be noted that, for expressways and national highways, even if the design traffic is less than 20 msa, VG40 bitumen shall be used for surface as well as DBM layers.

In the absence of axle load spectrum data, in the development of the design catalogues, the CTB layer was checked only for one fatigue criterion given by equation 3.5. However, it is essential to check the CTB thickness with project specific axle load spectrum as mentioned in these guidelines.

The values of RF factor used in Equation 3.5 are taken as 2 for design traffic less than 10 msa and as 1 for design traffic of 10 msa or more.

The mix volumetric parameters used are: V_a of 3.5% and V_{be} of 11.5% and fatigue equation 'C' factor of 2.35 for pavement cases (2), (3) and (5) mentioned in 12.2. For (1), (4) and (6) pavement cases, for design traffic of 5, 10 and 20 msa, the V_a, V_{be} and 'C" factor values considered are 4.5%, 10.5% and 1.12 and for 20, 30, 40 and 50 msa, the values are 3.5%, 11.5% and 2.35 respectively.

Figs. 12.1 to **12.48** present the design catalogues developed for the six pavement composition types mentioned in section 12.2. The catalogues were developed based on the assumptions discussed in the above section. Example calculations for 10% effective subgrade CBR case are presented in **Annex-III**. Some of the thicknesses (especially those of bituminous layers) given in the thickness templates have been selected based on the minimum thickness requirement of bituminous layer for pavements with CTB base.

A large number of pavement design options are possible for less than 50 msa design traffic with varying values of (a) design traffic (b) effective subgrade CBR (c) reliability (d) mix volumetric parameters and the corresponding 'C' factor (e) pavement composition (f) classification of highway and (g) binder used with the base (DBM/BM) mix. It must be noted that the catalogues are for the specific inputs considered for developing them. The designers or the user agencies are advised to get familiarized with the use of IITPAVE software (for which very detailed guidelines are given in **Annex-I**) so that pavements can be designed with any selected combination of inputs.

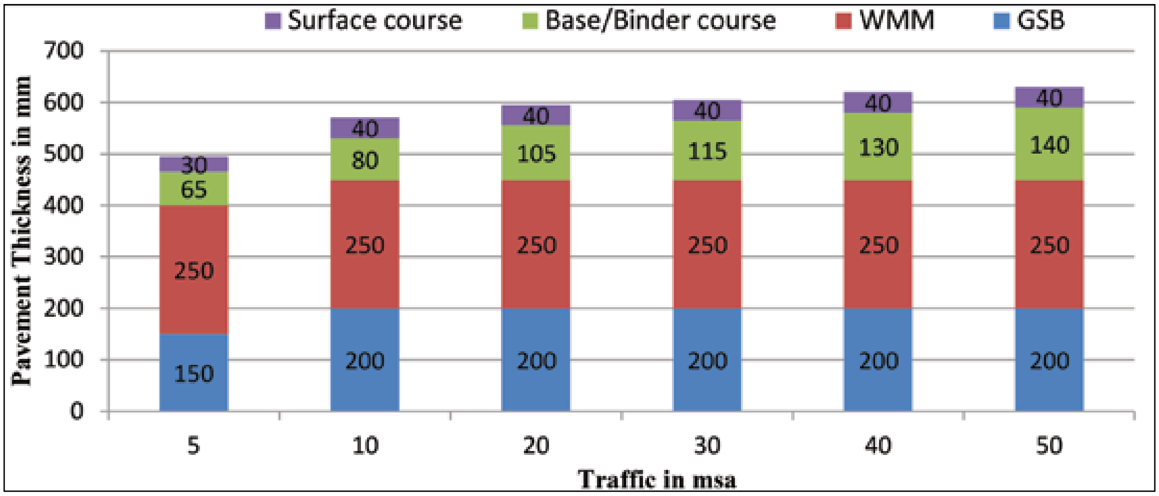

Fig. 12.1 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 5% (Plate-1)

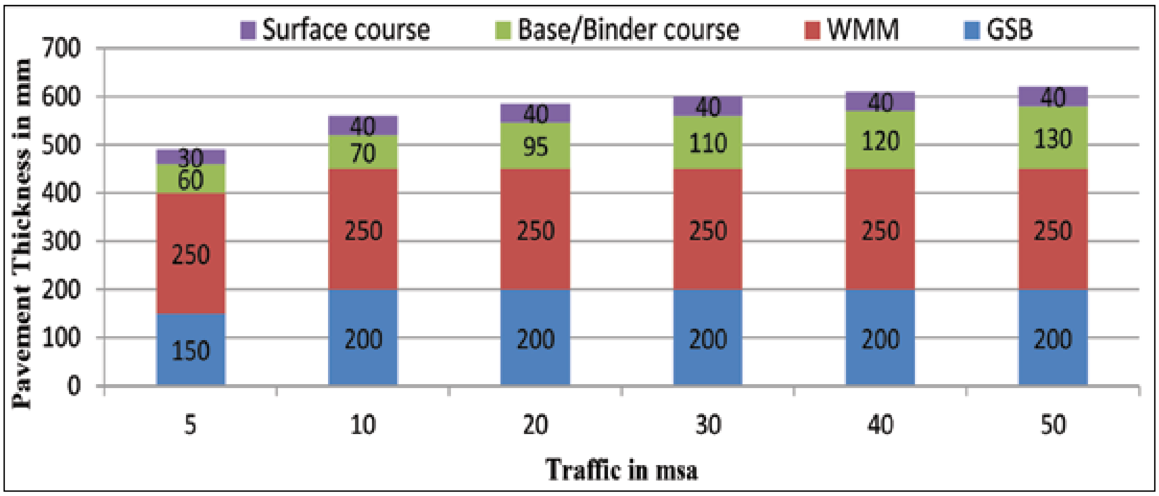

Fig. 12.2 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 6% (Plate-2)

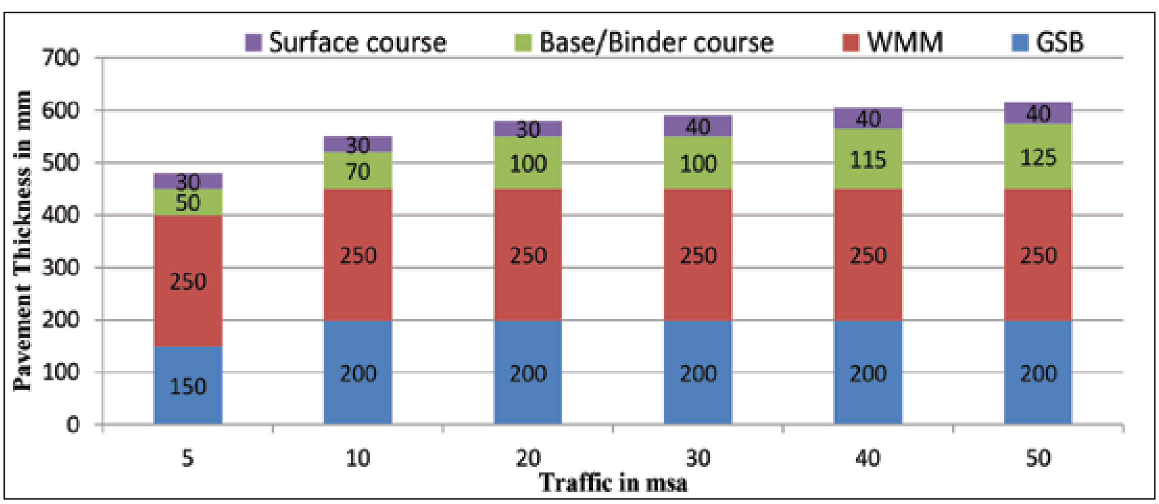

Fig. 12.3 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 7% (Plate-3)

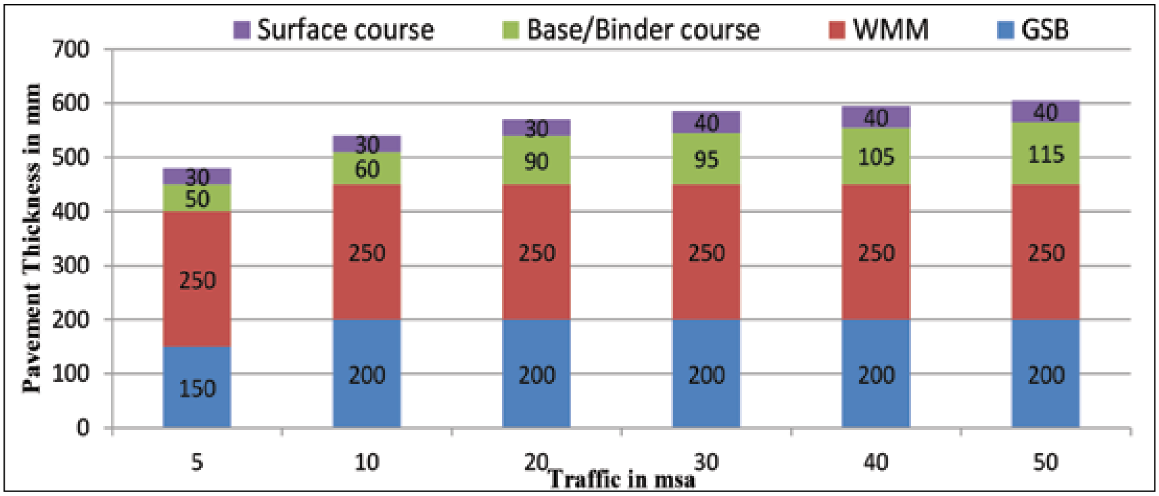

Fig. 12.4 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 8% (Plate-4)

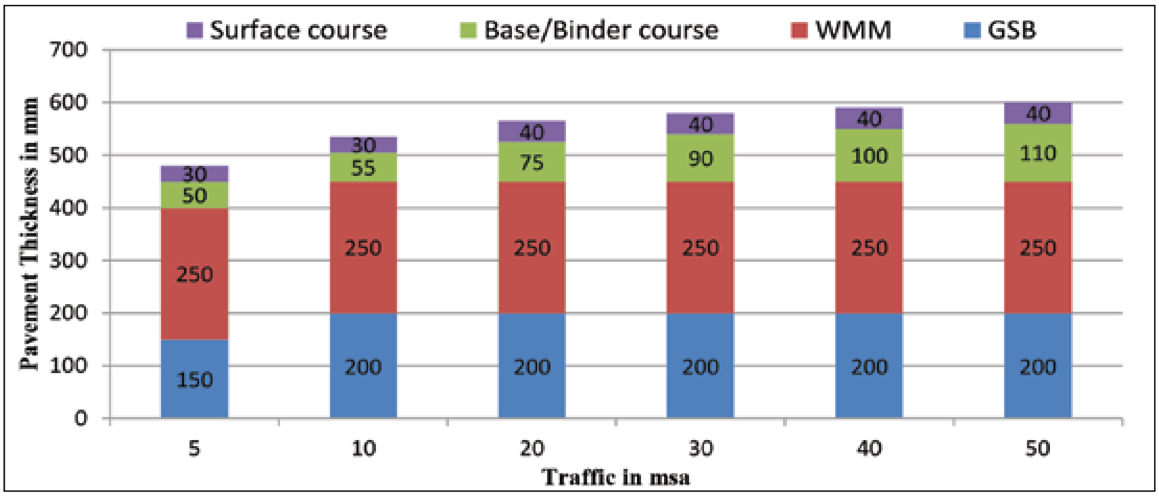

Fig. 12.5 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 9% (Plate-5)

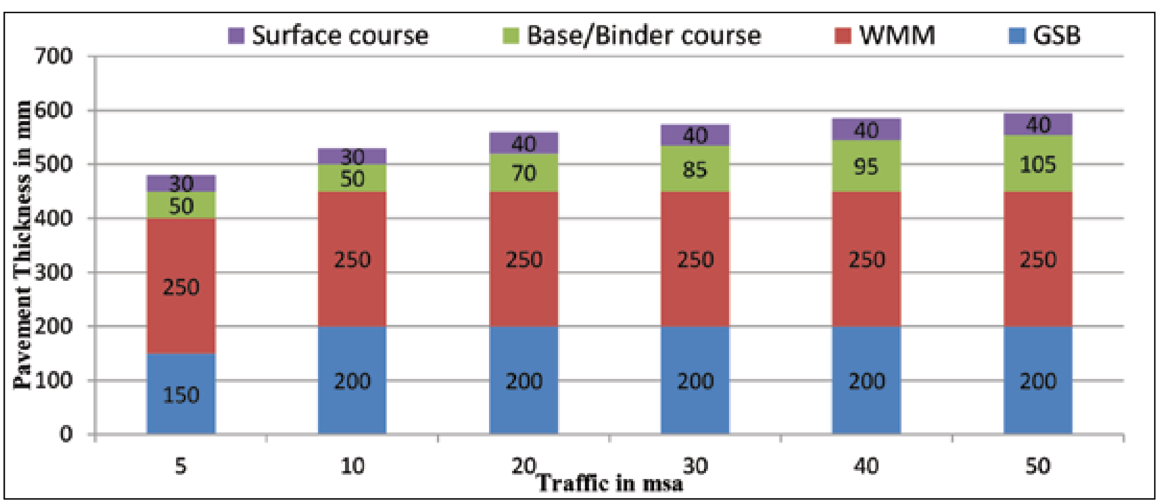

Fig. 12.6 Catalogue for Pavement with Bituminous Surface Course with Granular Base and sub-base - Effective CBR 10% (Plate-6)

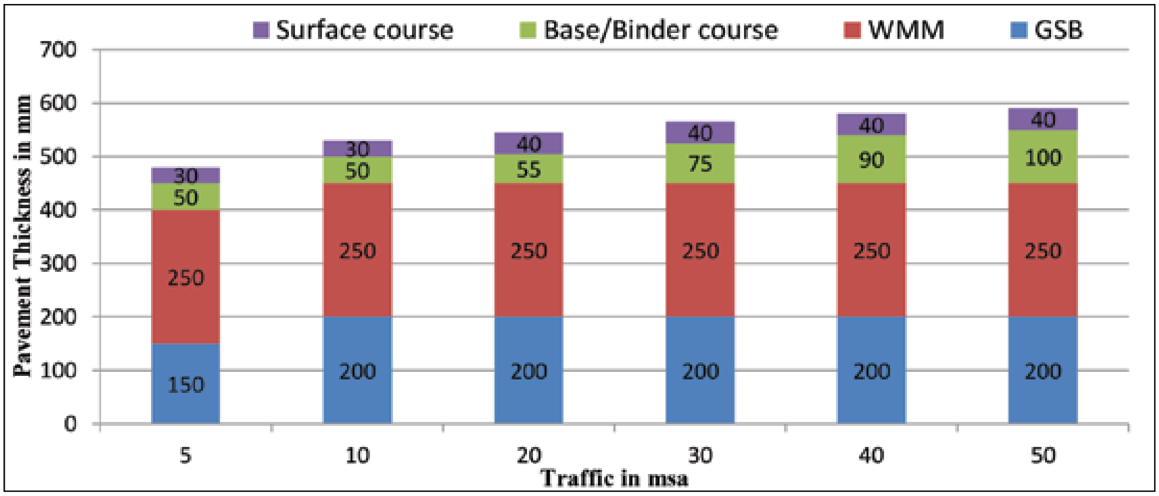

Fig. 12.7 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 12% (Plate-7)

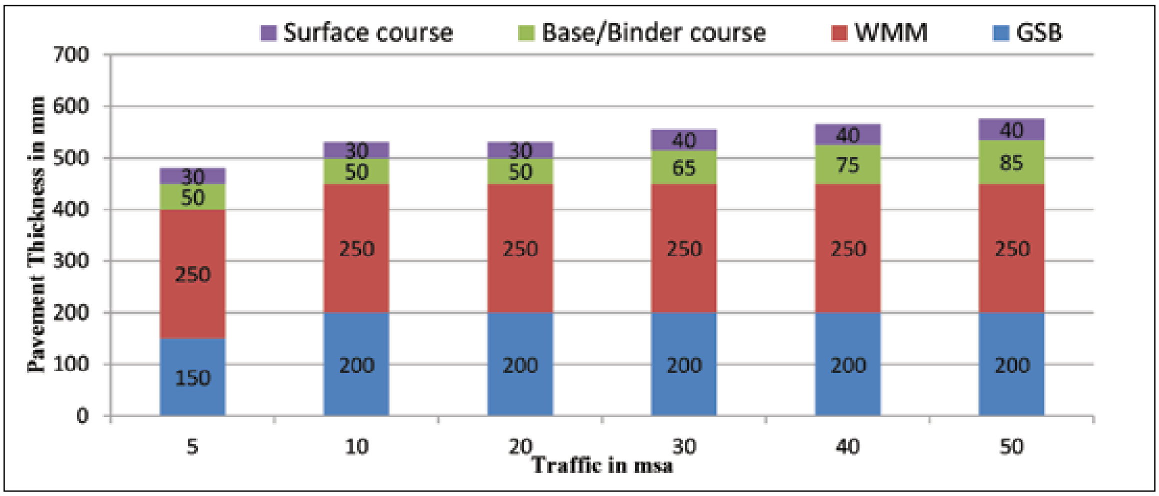

Fig. 12.8 Catalogue for Pavement with Bituminous Surface Course with Granular Base and Sub-base - Effective CBR 15% (Plate-8)

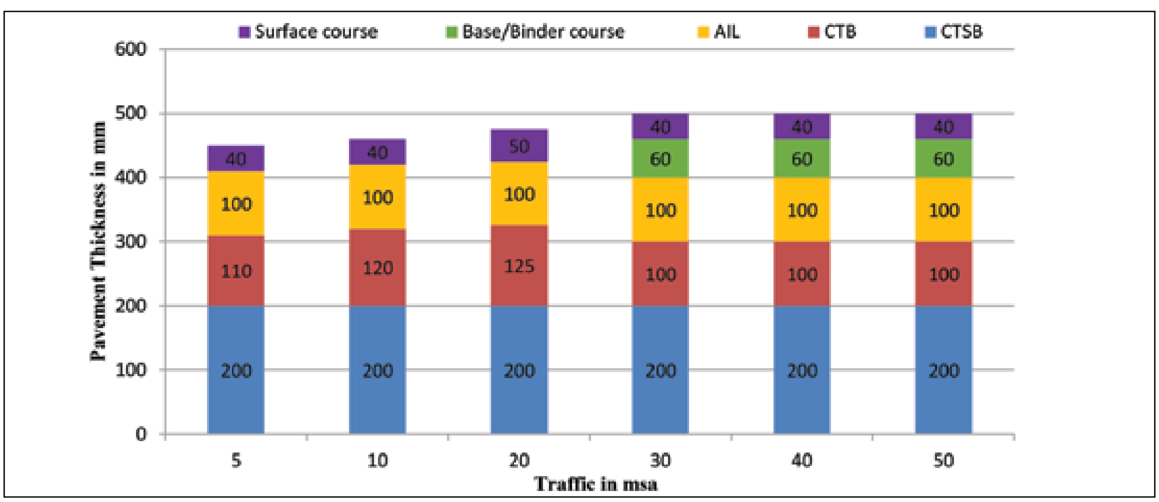

Fig. 12.9 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 5% (Plate-9)

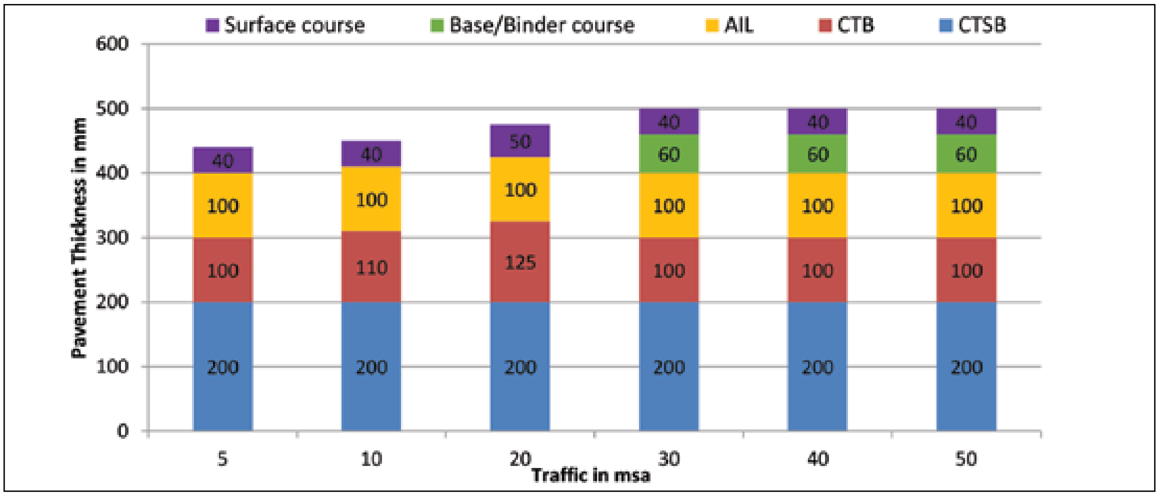

Fig. 12.10 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 6% (Plate-10)

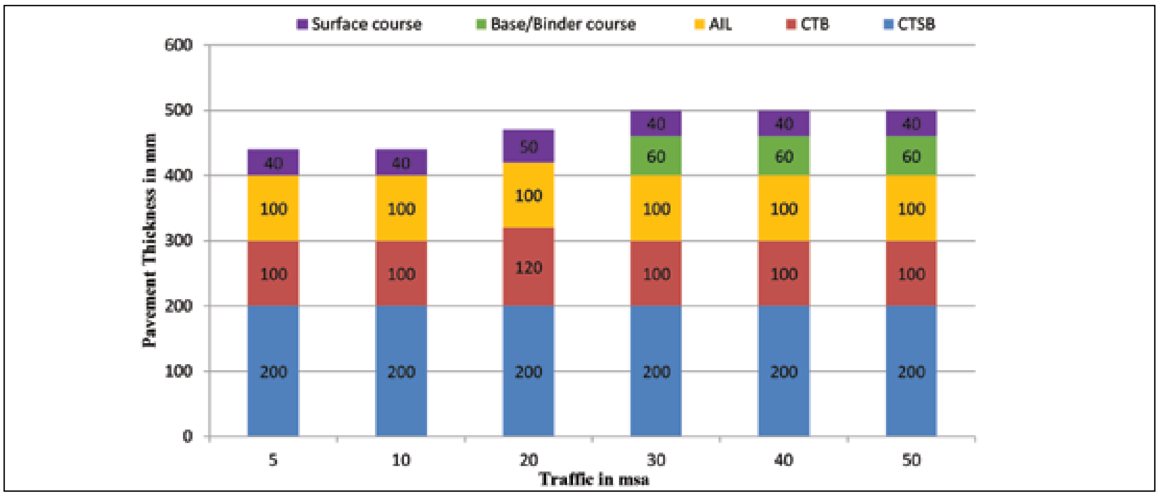

Fig. 12.11 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 7% (Plate-11)

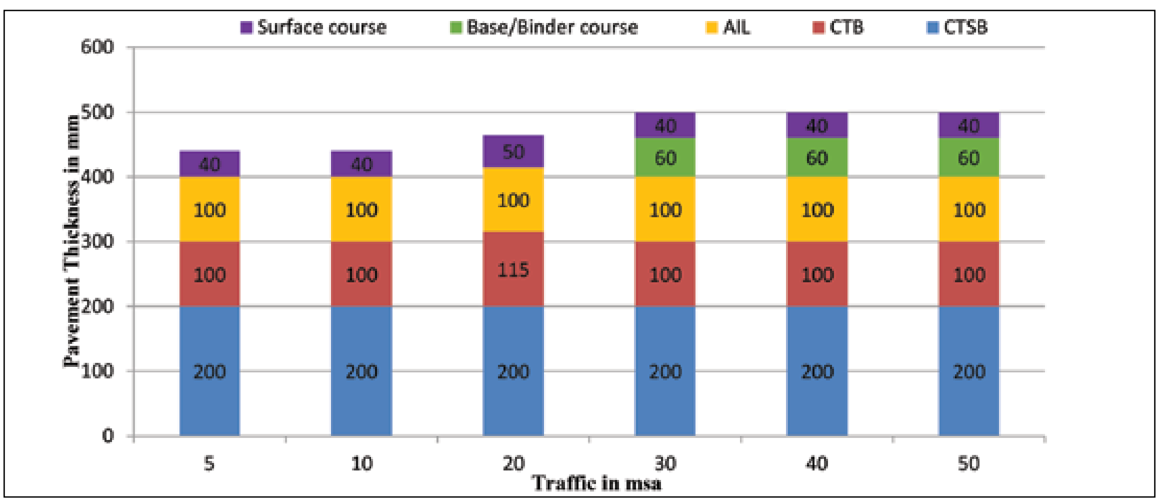

Fig. 12.12 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 8% (Plate-12)

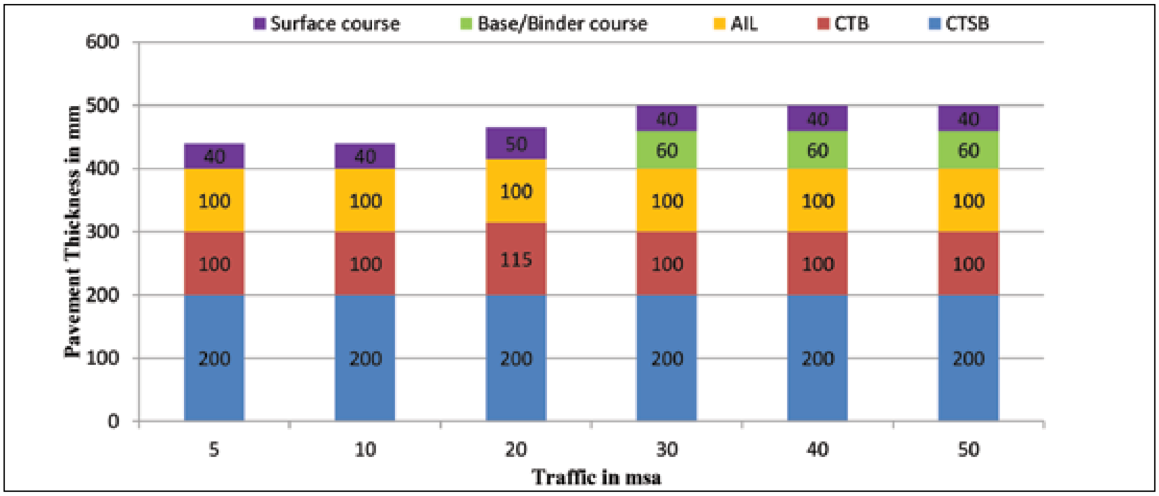

Fig. 12.13 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 9% (Plate-13)

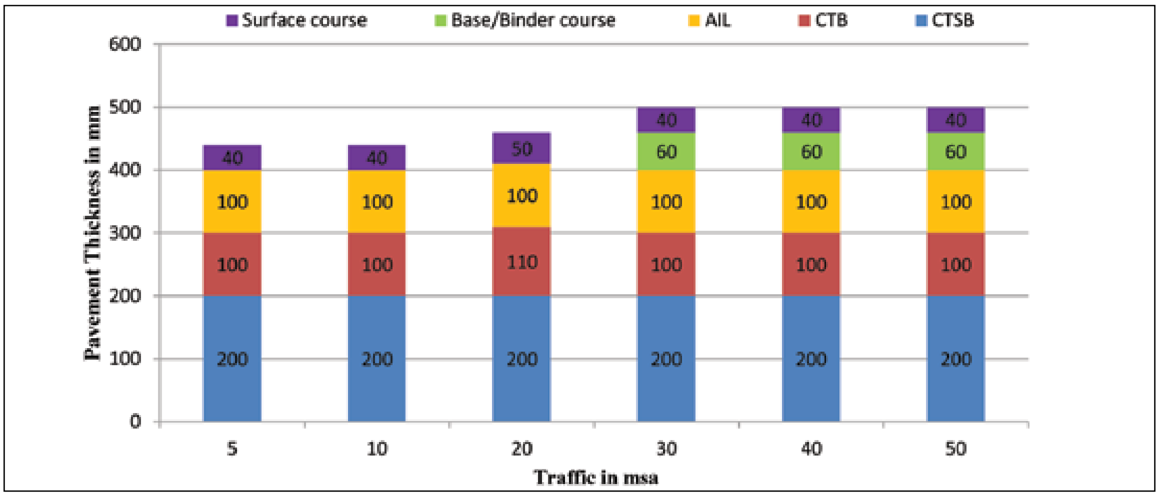

Fig. 12.14 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 10% (Plate-14)

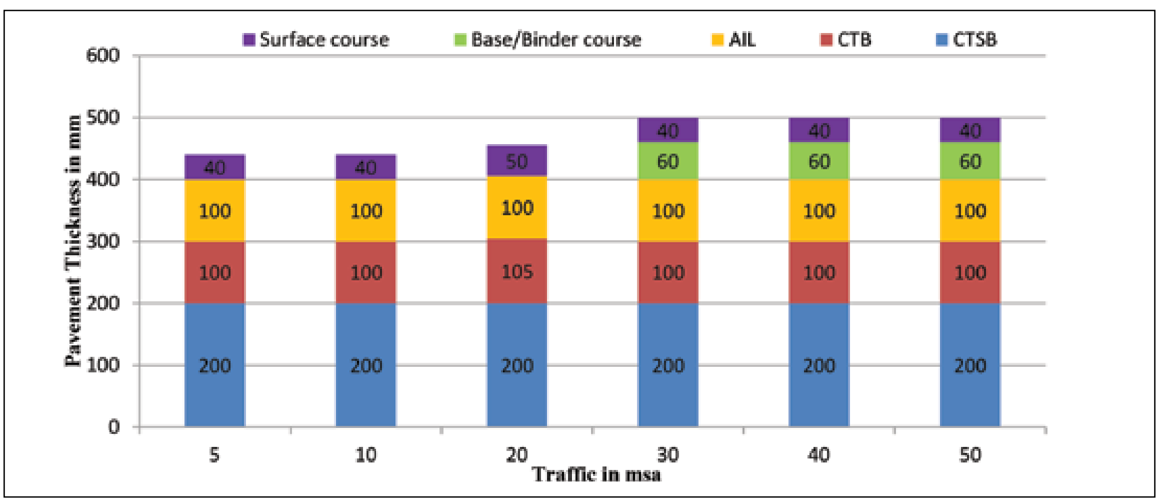

Fig. 12.15 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 12% (Plate-15)

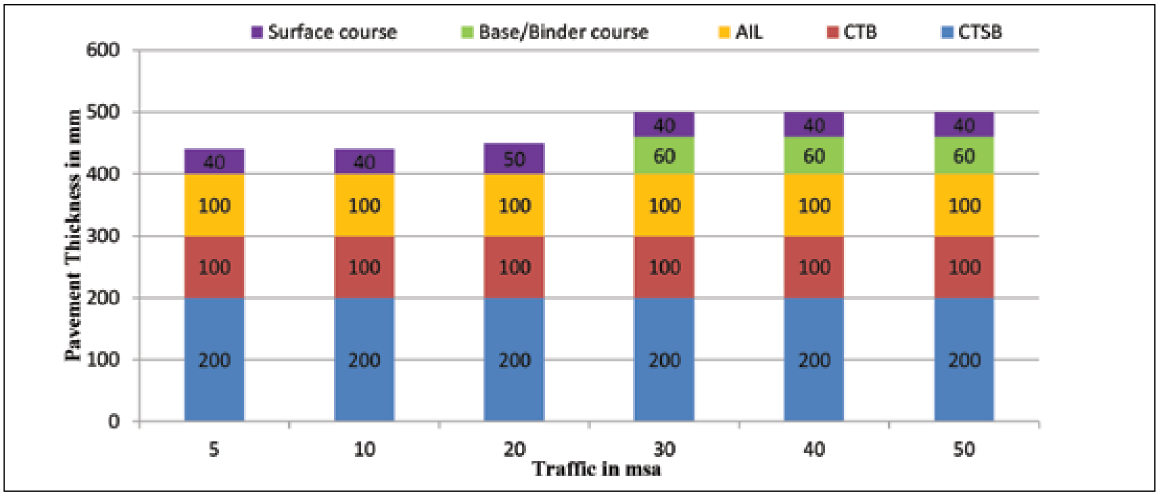

Fig. 12.16 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer - Effective CBR 15% (Plate-16)

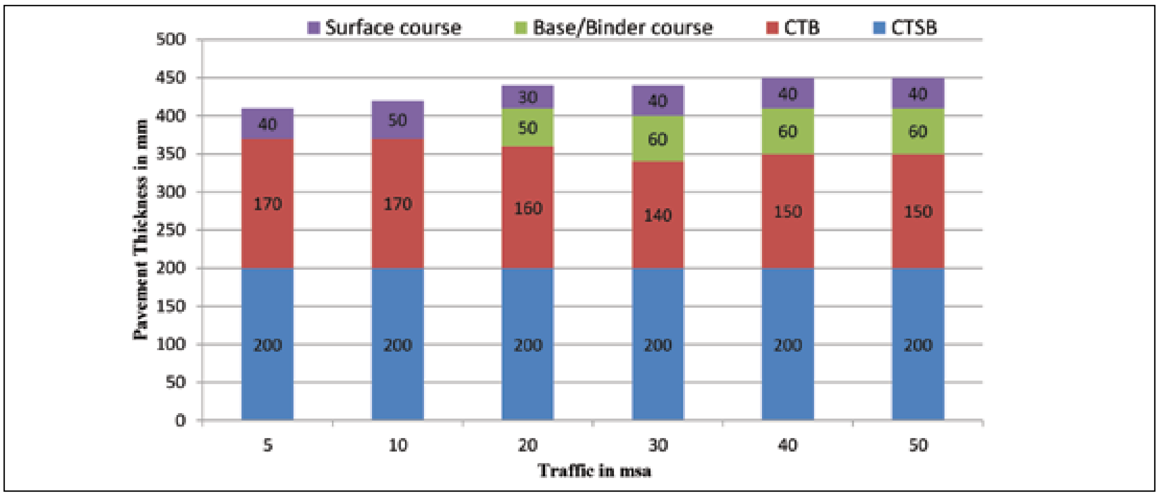

Fig. 12.17 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 5% (Plate-17)

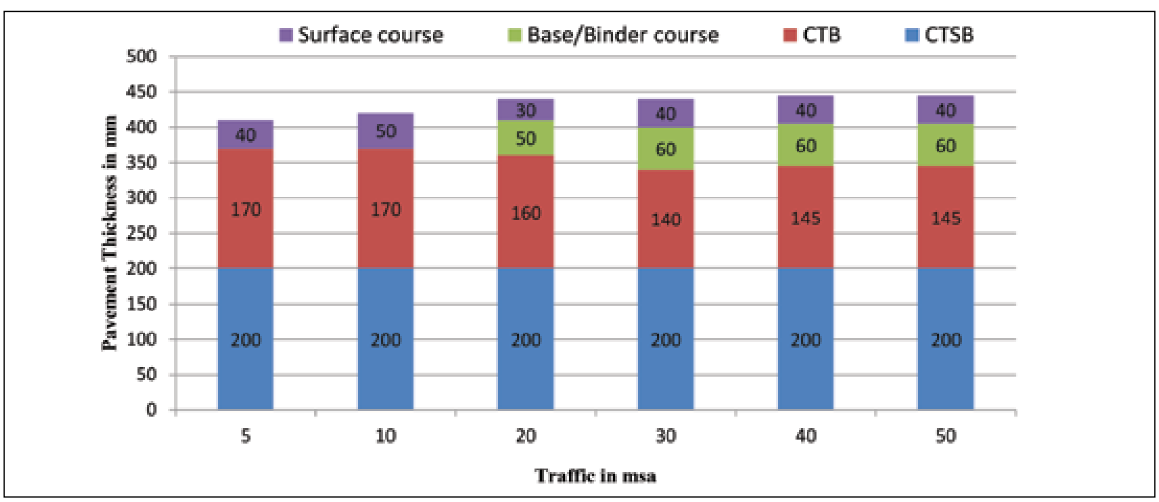

Fig. 12.18 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 6% (Plate-18)

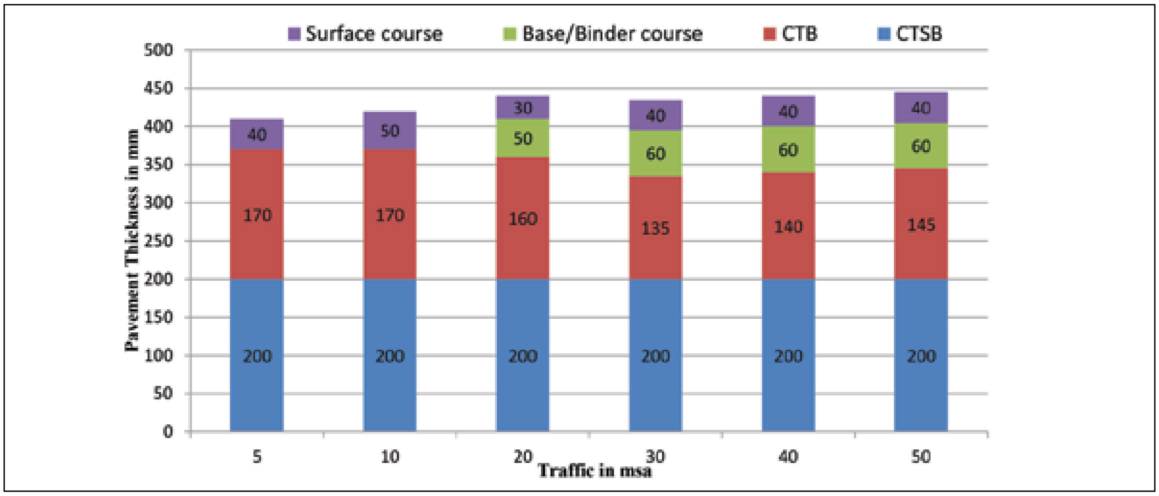

Fig. 12.19 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 7% (Plate-19)

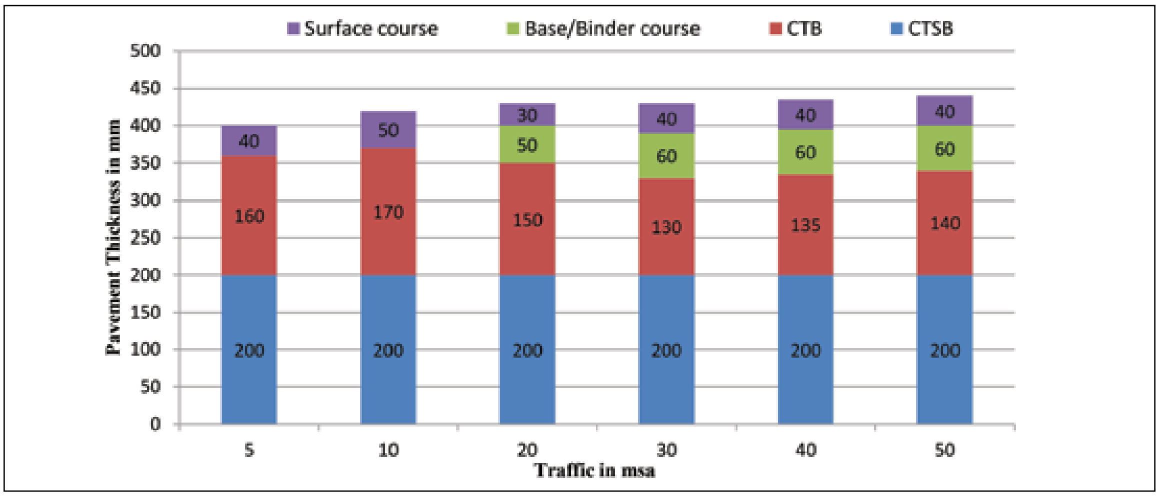

Fig. 12.20 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 8% (Plate-20)

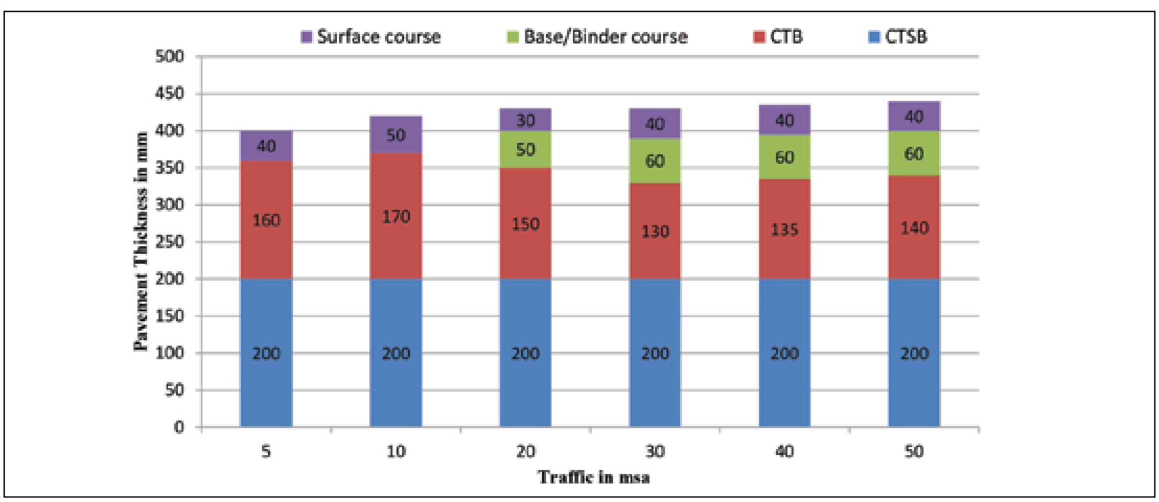

Fig. 12.21 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 9% (Plate-21)

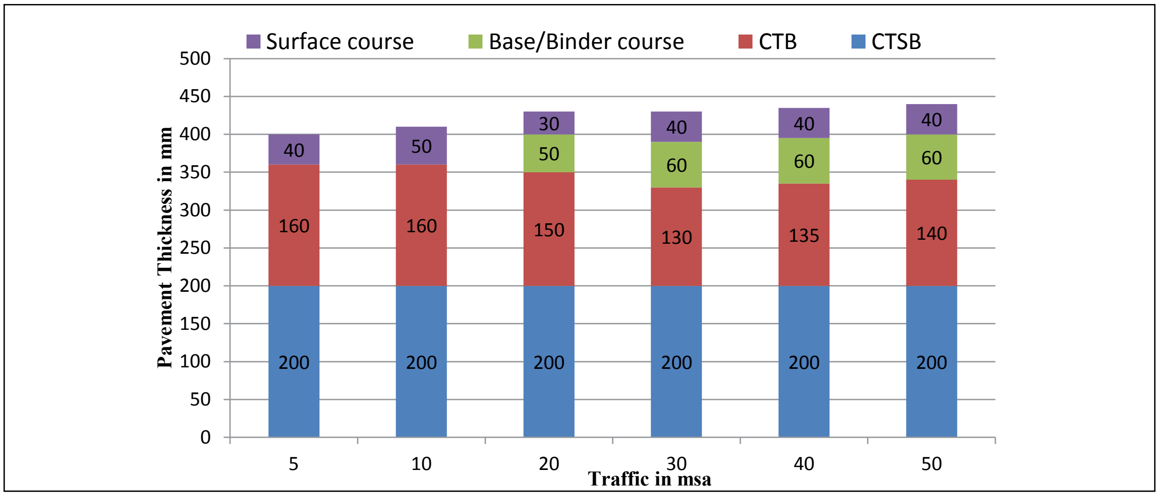

Fig. 12.22 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 10% (Plate-22)

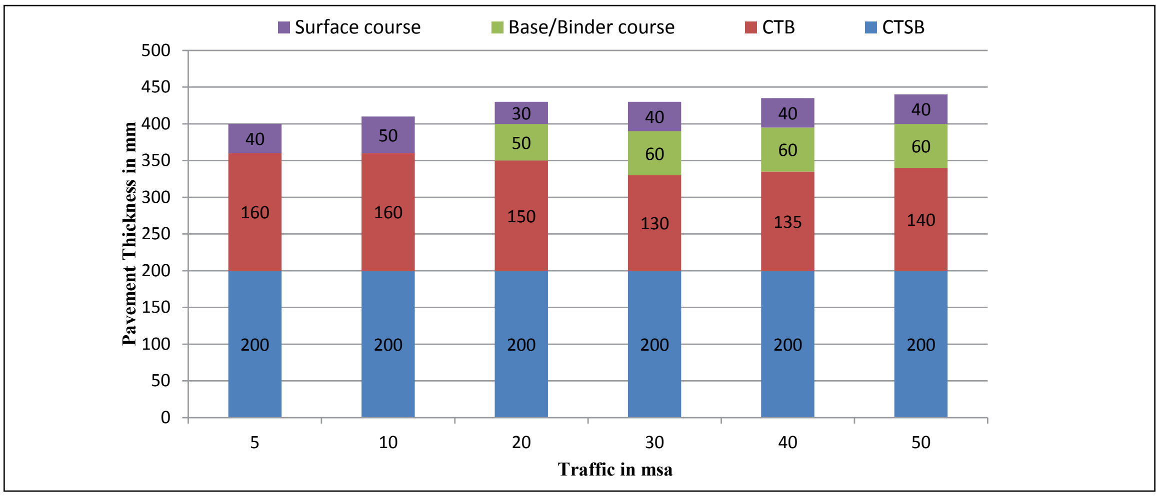

Fig. 12.23 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 12% (Plate-23)

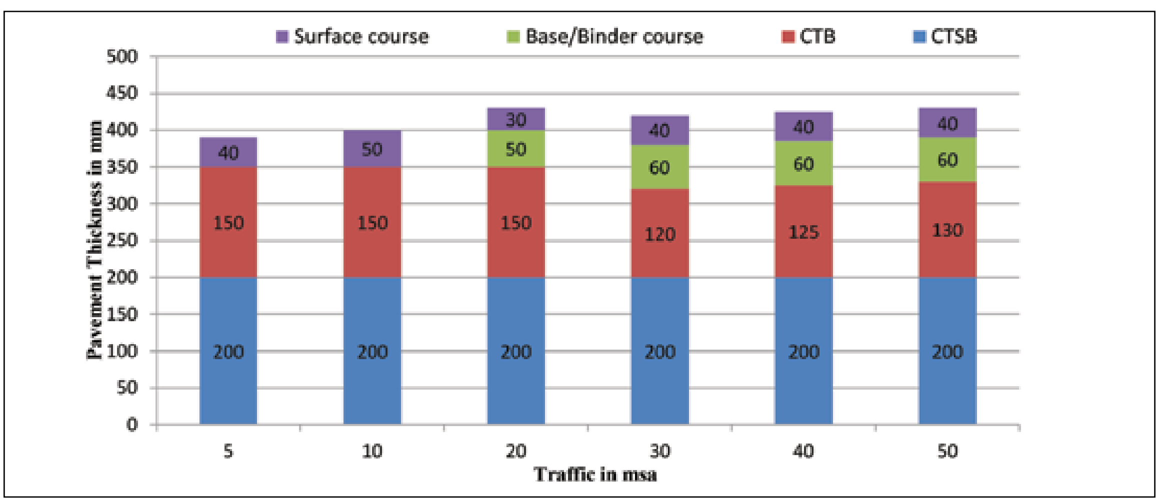

Figure 12.24 Catalogue for Pavement with Bituminous Surface Course with CTSB, CTB and SAMI - Effective CBR 15% (Plate-24)

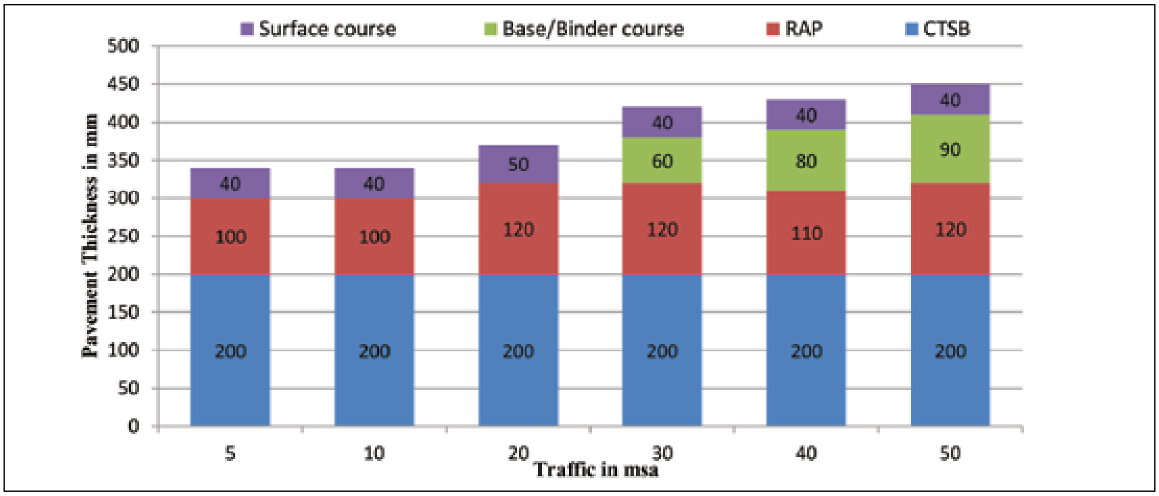

Fig. 12.25 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 5% (Plate-25)

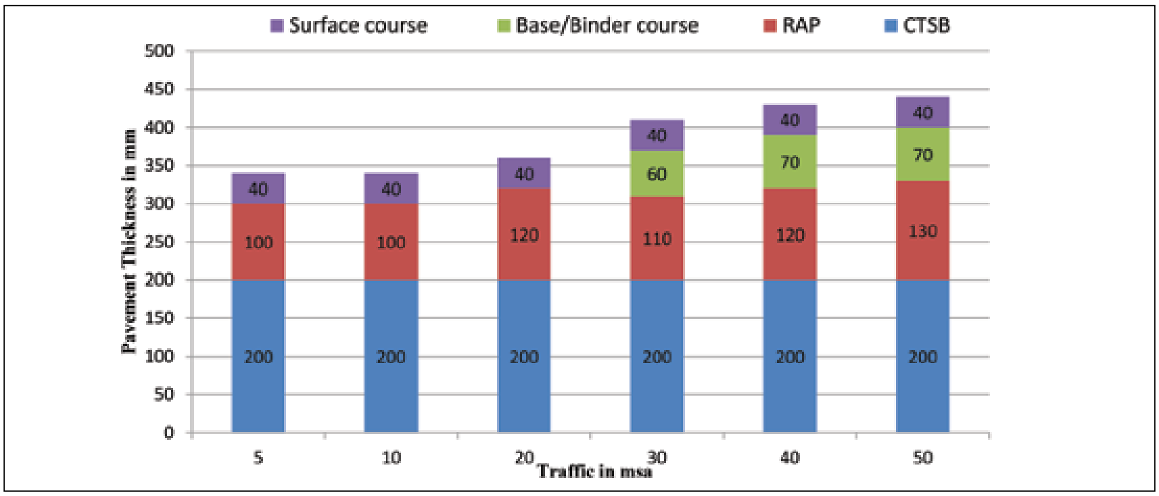

Fig. 12.26 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 6% (Plate-26)

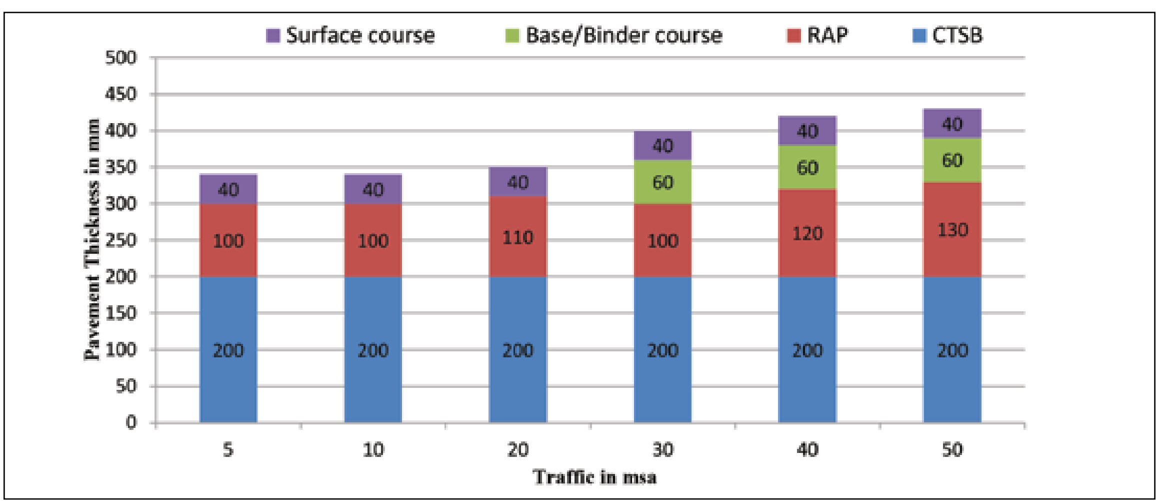

Fig. 12.27 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 7% (Plate-27)

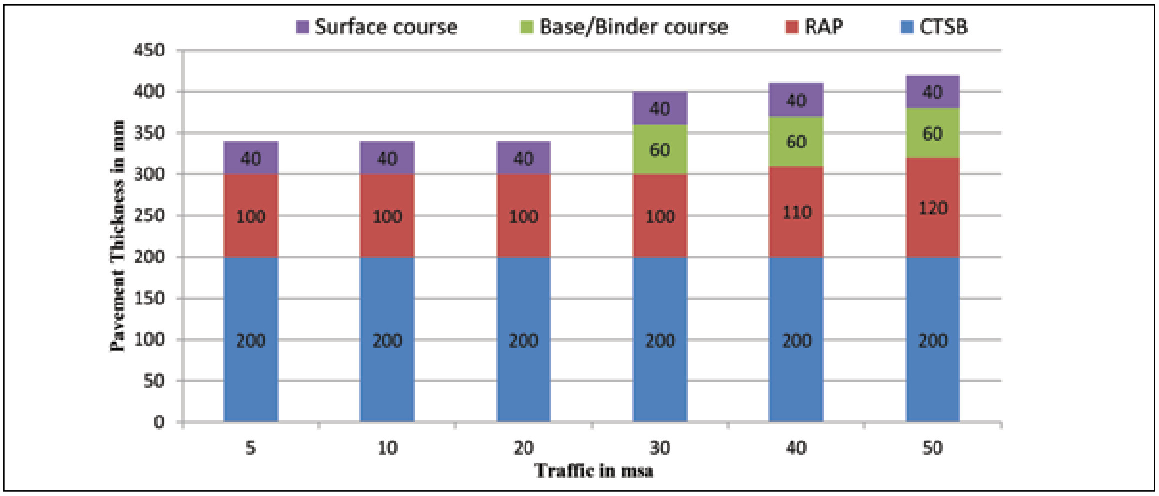

Fig. 12.28 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 8% (Plate-28)

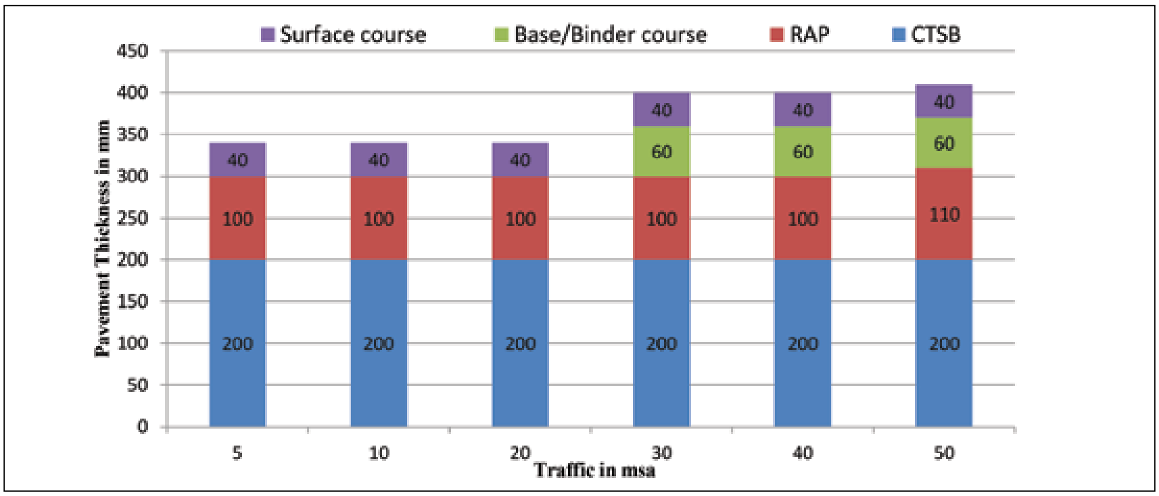

Fig. 12.29 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 9% (Plate-29)

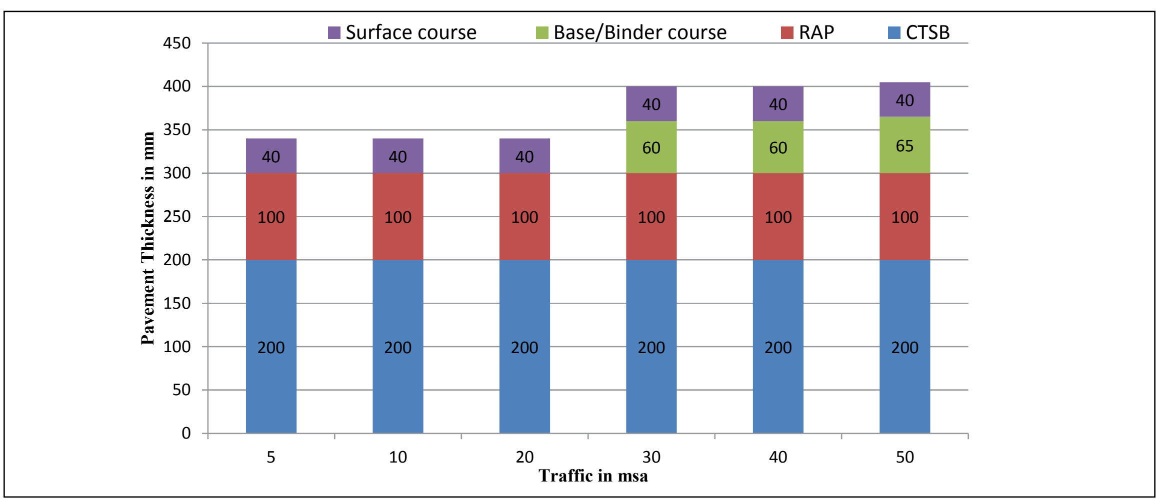

Fig. 12.30 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 10% (Plate-30)

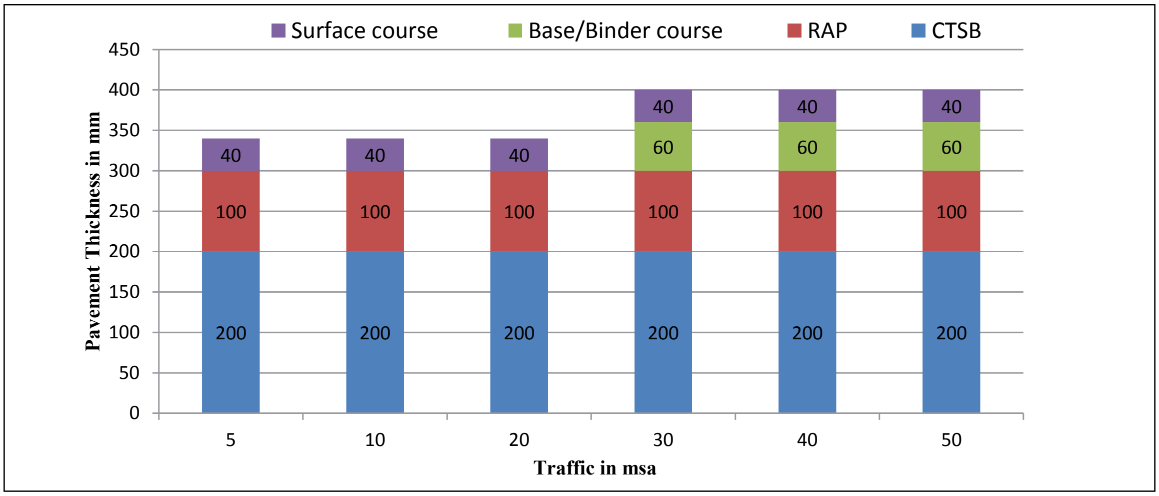

Fig. 12.31 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 12% (Plate-31)

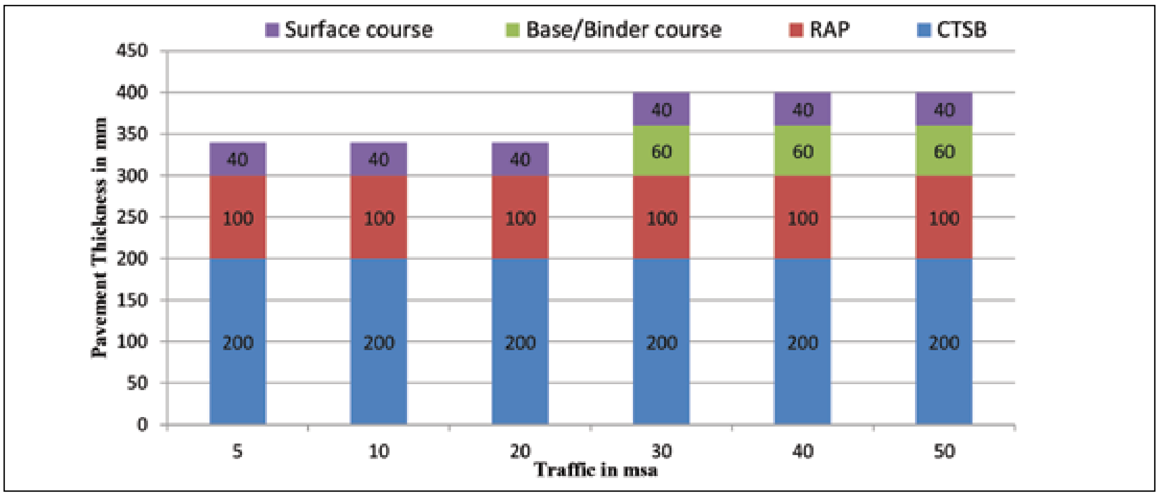

Fig. 12.32 Catalogue for Pavement with Bituminous Surface Course with CTSB and Emulsion/Foam Bitumen Stabilised RAP/Virgin Aggregate - Effective CBR 15% (Plate-32)

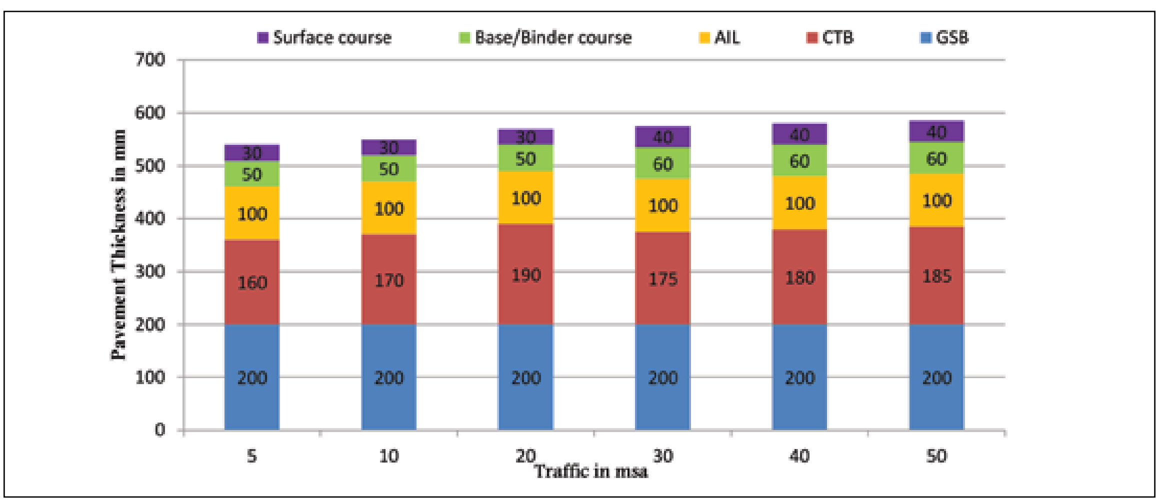

Fig. 12.33 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 5% (Plate-33)

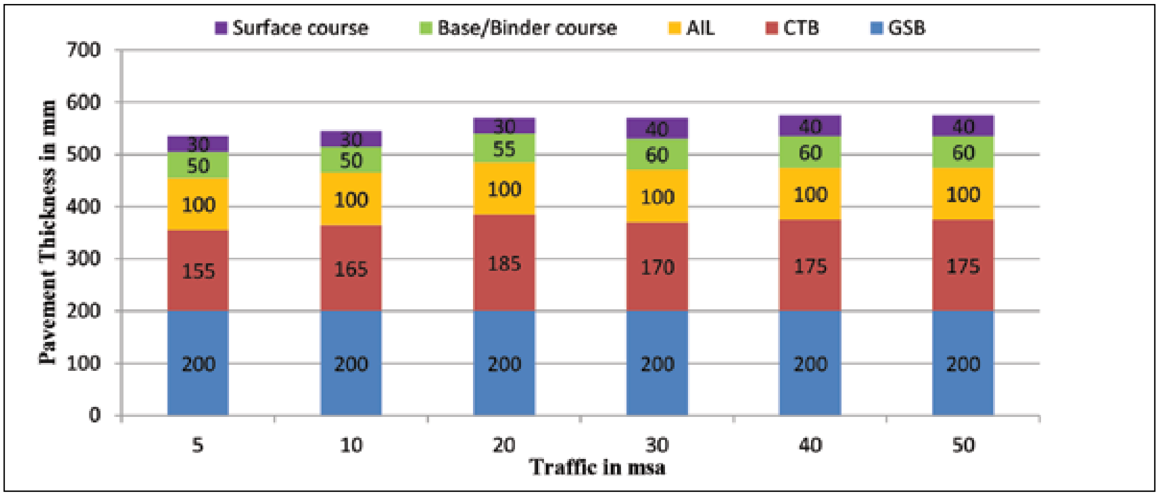

Fig. 12.34 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 6% (Plate-34)

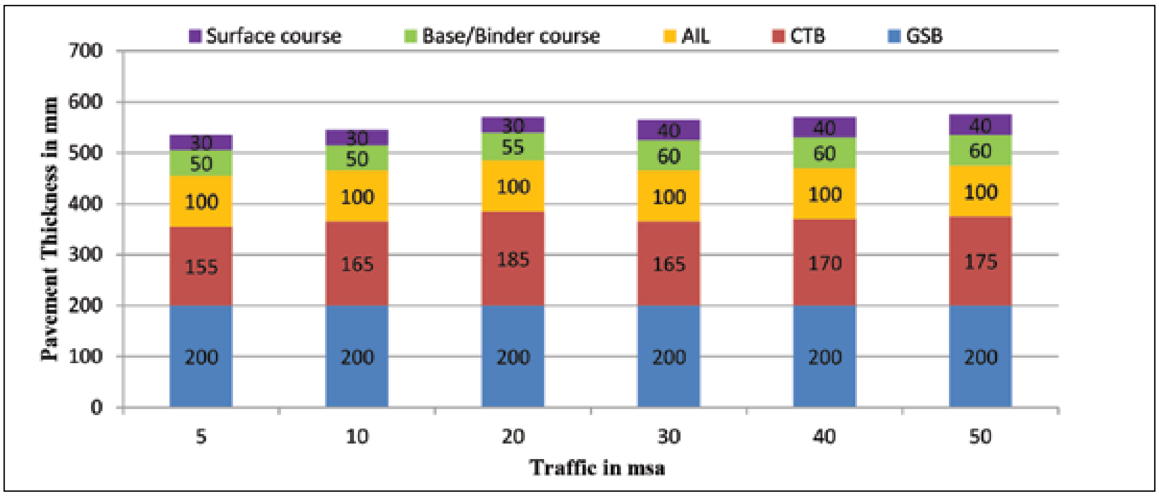

Fig. 12.35 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 7% (Plate-35)

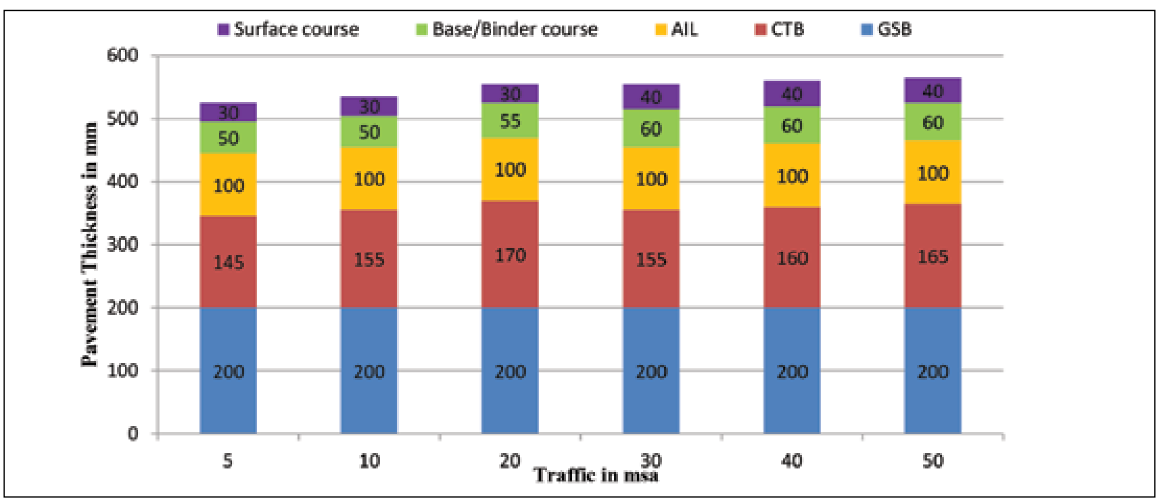

Fig. 12.36 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 8% (Plate-36)

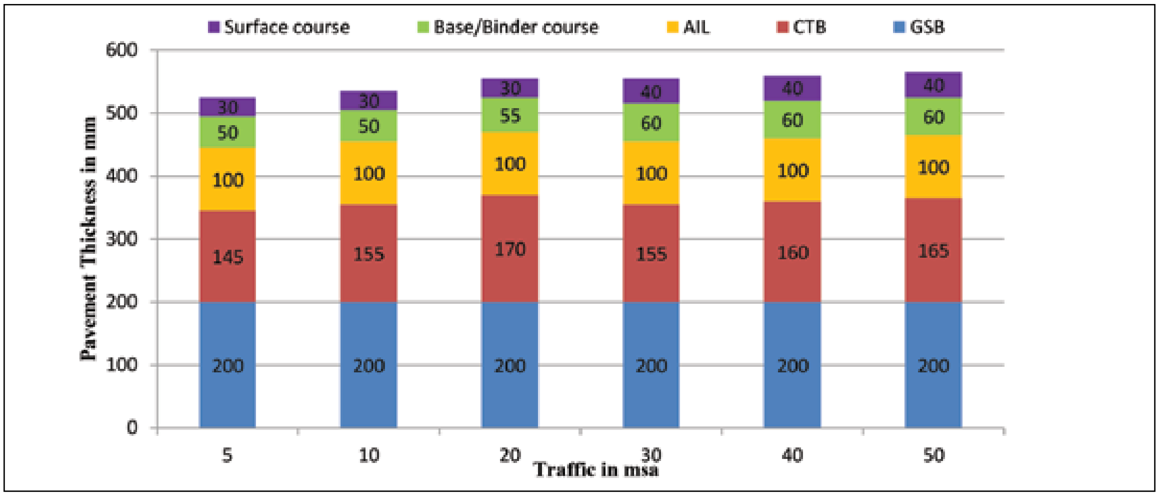

Fig. 12.37 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 9% (Plate-37)

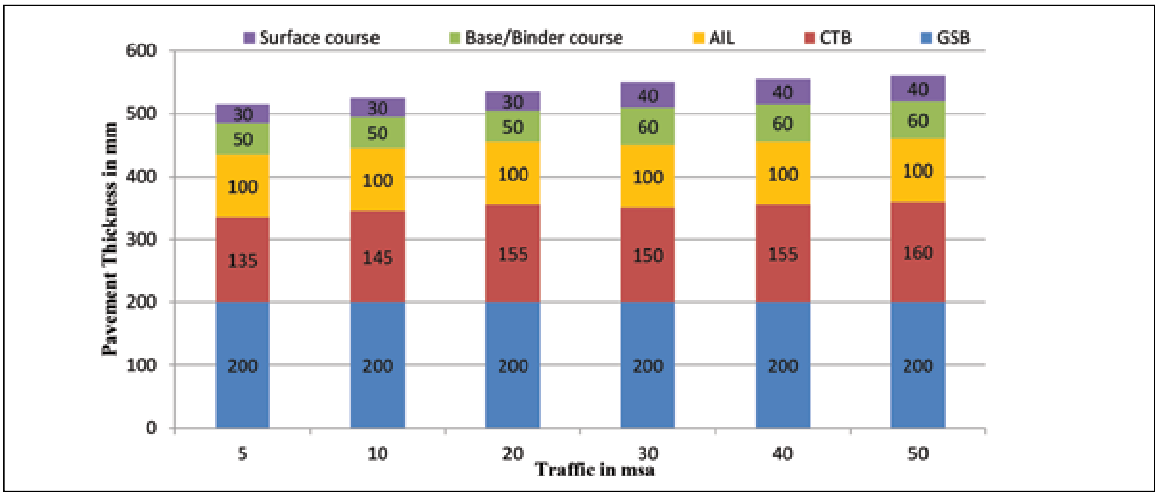

Fig. 12.38 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 10% (Plate-38)

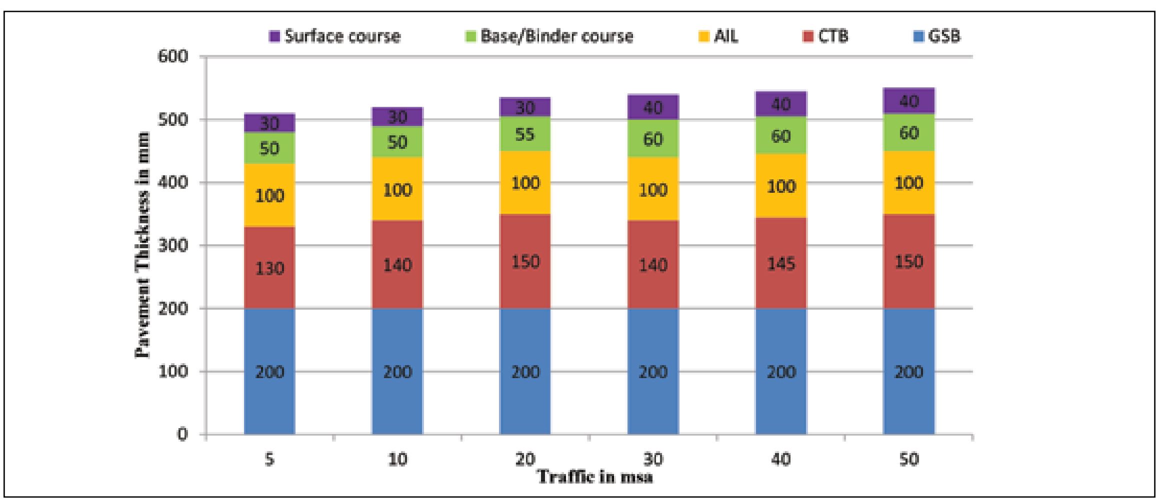

Fig. 12.39 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 12% (Plate-39)

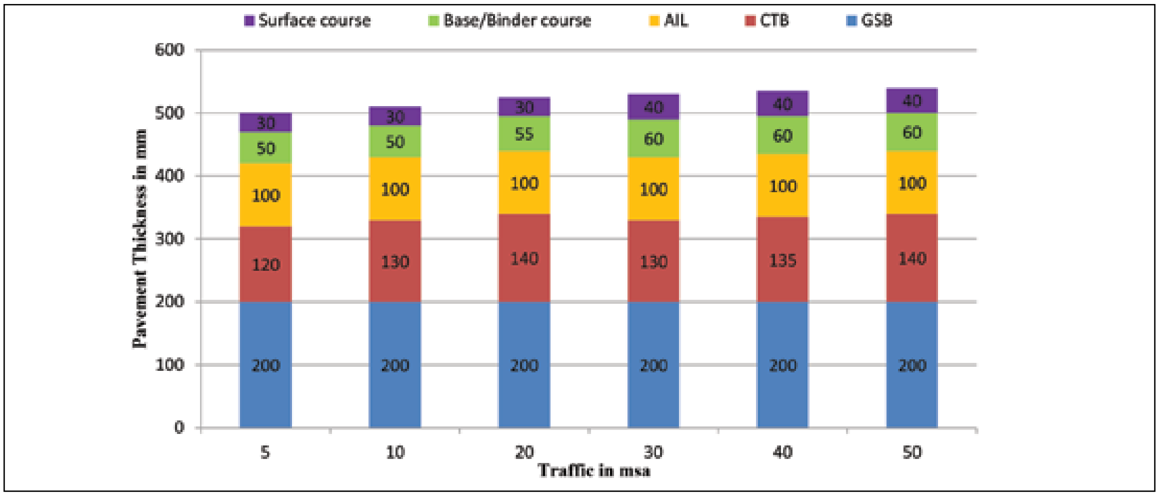

Fig. 12.40 Catalogue for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer - Effective CBR 15% (Plate-40)

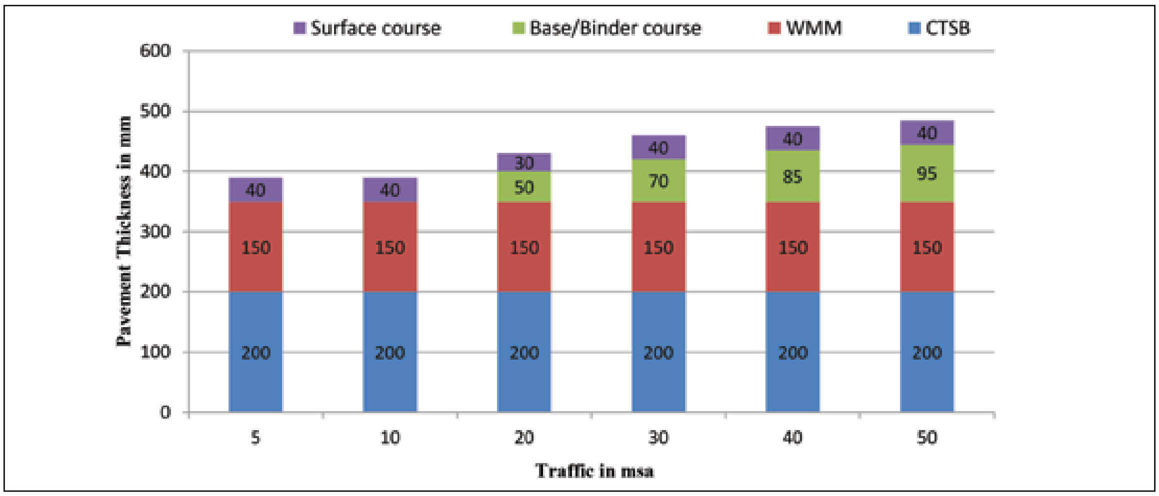

Fig. 12.41 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 5% (Plate-41)

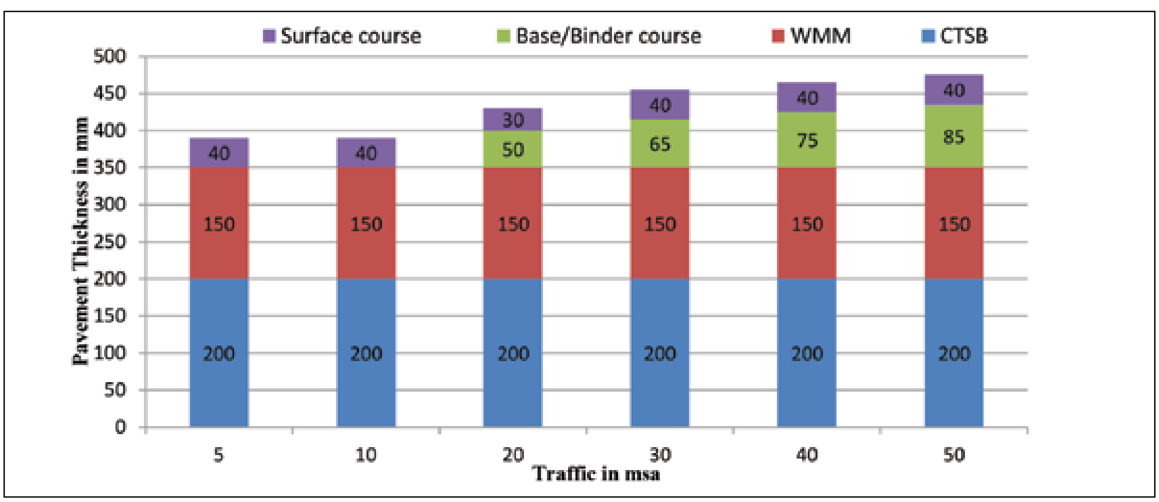

Fig. 12.42 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 6% (Plate-42)

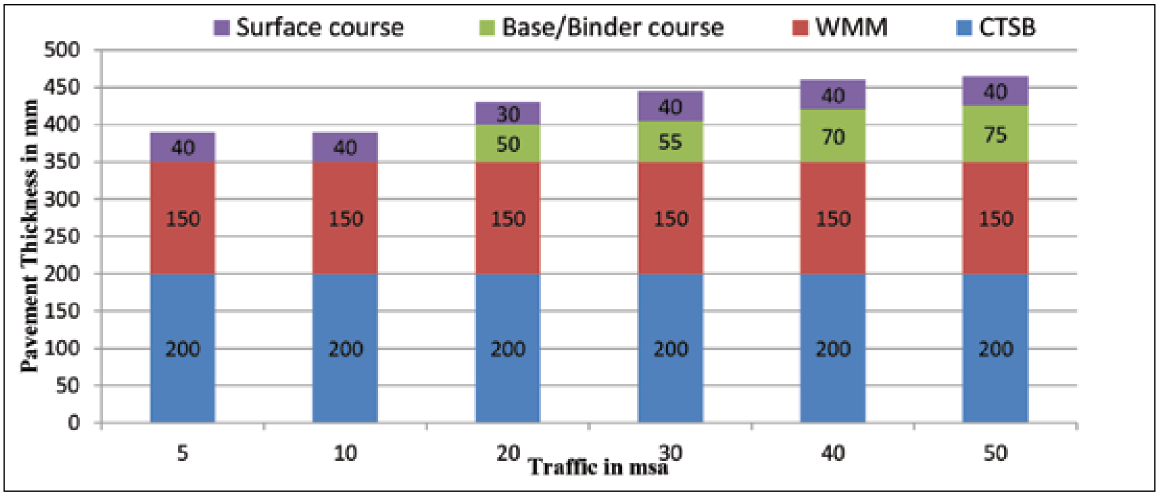

Fig. 12.43 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 7% (Plate-43)

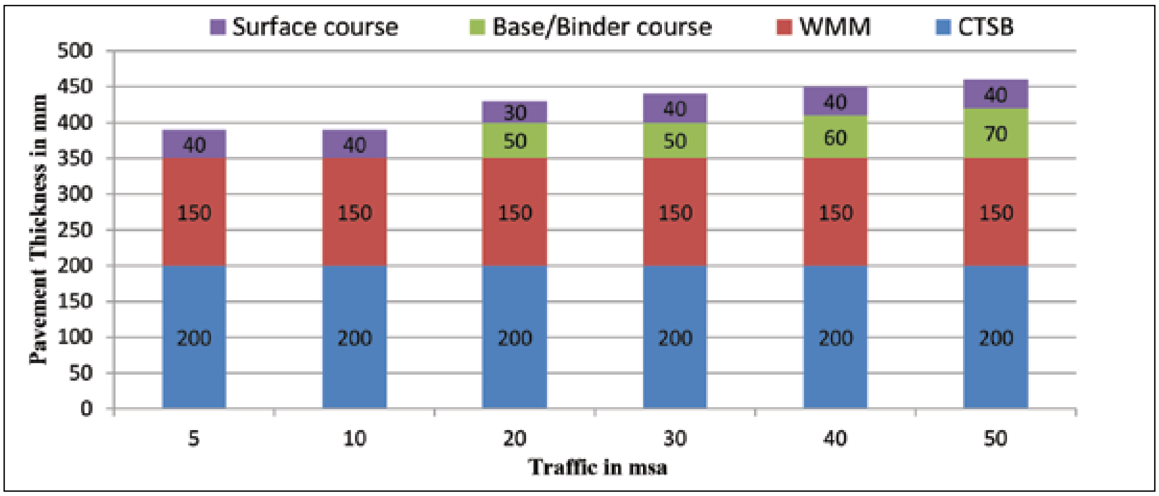

Fig. 12.44 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 8% (Plate-44)

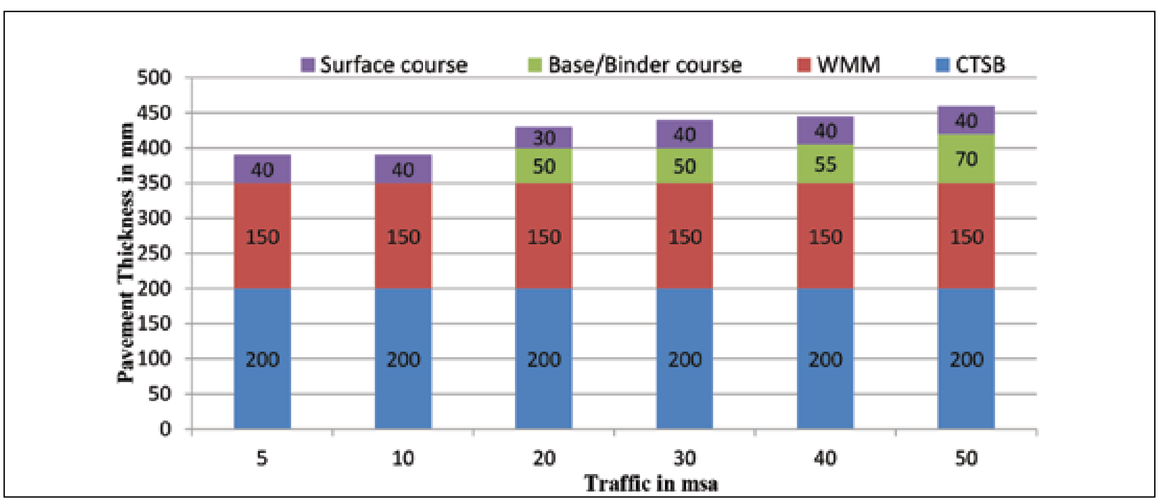

Fig. 12.45 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 9% (Plate-45)

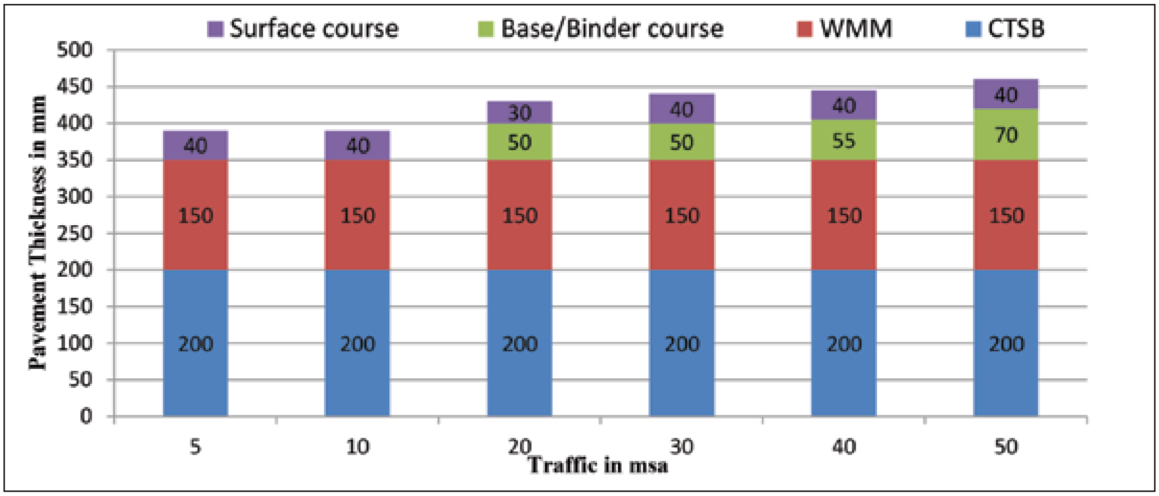

Fig. 12.46 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 10% (Plate-46)

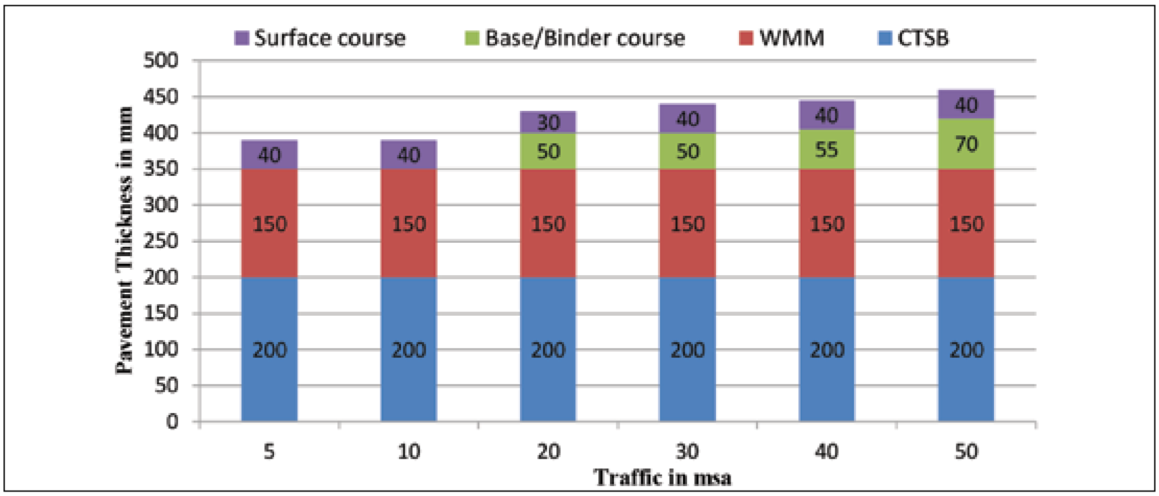

Fig. 12.47 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 12% (Plate-47)

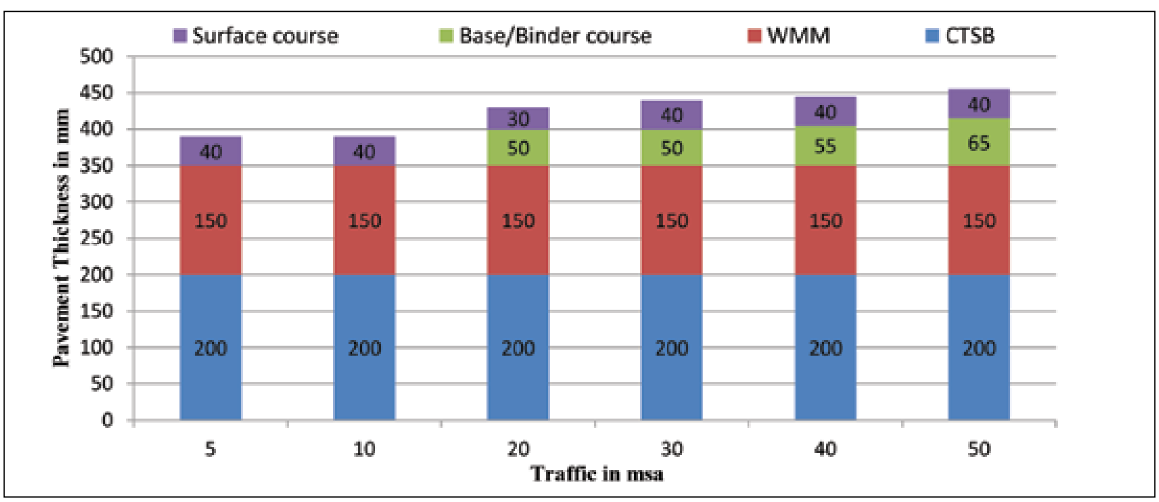

Fig. 12.48 Catalogue for Pavement with Bituminous Surface Course with CTSB and Granular Base Course - Effective CBR 15% (Plate-48)

13 DESIGN IN FROST AFFECTED AREAS

13.1 In areas susceptible to frost action, the design will have to be related to the actual depth of penetration and severity of the frost. At the subgrade level, fine grained clayey and silty soils are more susceptible to ice formation, but freezing conditions could also develop within the pavement structure if water has a chance of ingress from above.

13.2 One remedy against frost attack is to increase the depth of construction to correspond to the depth of frost penetration, but this may not always be economically practicable. As a general rule, it would not be advisable to provide total pavement thickness less than 450 mm even when the CBR value of the subgrade warrants a smaller thickness. In addition, the materials used for building up the crust should be frost resistant.

13.3 Another precaution against frost attack is that water should not be allowed to collect at the subgrade level which may happen on account of infiltration through the pavement surface or verges or due to capillary rise from a high water table. Whereas capillary rise can be prevented by subsoil drainage measures and cut-offs, infiltrating surface water can be checked only by providing a suitable surfacing course and a subsurface drainage system.

14 QUALITY CONTROL TESTS DURING CONSTRUCTION

The recommendations contained in Clause 903 of Specifications of the Ministry of Road Transport and Highways for Road and Bridge Works[23], IRC:120[33], IRC:SP:89[28], IRC:SP:59[26] about different tests along with their frequencies for different types of specifications to ensure quality in the construction are to be followed. In addition, the following tests are also required for addressing the specifications/aspects not covered in the documents referred to above.

S. No.	Item of Construction	Test	Frequency
1	Bituminous construction	Resilient modulus estimated from indirect tensile strength test on specimens prepared using field mix*	Three specimens for each 400 tonnes of mix subject to minimum 2 tests per day.
2	Cement treated/ stabilised base and sub-base	Unconfined compressive strength	Three specimens for each 400 tonnes of mix subject to minimum 2 tests per day.
3	Cement treated/ stabilised base and sub-base	Binder/cement content	Three specimens for each 400 tonnes of mix subject to minimum 2 tests per day.
4	Cement treated/ stabilised base and sub-base	Flexural strength/Indirect tensile strength test	Three specimens for each 400 tonnes of mix subject to minimum 2 tests per day.

S. No.	Item of Construction	Test	Frequency
5	Cement treated/ stabilised base and sub-base	Soundness test (BIS 4332 Part IV)	One specimen for each source and whenever there is change in the quality of aggregate
6	Cement treated/ stabilised base and sub-base	Density of compacted layer	One specimen of two tests per 500 sq m.
7	Emulsion/Foam bitumen	Indirect tensile strength test	Three specimens for each 400 tonnes of mix subject to minimum 2 tests per day.
8	Emulsion/Foam bitumen	Density of compacted layer	One specimen per 1000 sq m.

* In case, the M_R estimated from the indirect tensile strength is less than 90% of the design value, the M_R should be rechecked in accordance with ASTM 4123.

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APPENDIX-A

THE PRINCIPLES AND APPROACH FOLLOWED IN THESE GUIDELINES

A.1 An Overview

Highway pavements should be safe and serviceable. They should be capable of carrying the loads coming on it during their life period without unacceptable levels of failures. Unlike structures where the failure is usually followed by complete collapse, failure in pavements is not sudden but usually by gradual deterioration over time. At some stage in its life, when the deterioration renders it unserviceable to the users, the pavement is assumed as failed. Thus, safety criteria in pavement design are defined by serviceability thresholds (such as acceptable cracking and rutting), which, if breached, the design should be considered as unsafe and pavement unserviceable.

A.2 Cracking in Bituminous Layers

Cracking in pavement can occur in three primary modes: (a) bottom up cracking, (b) top down cracking, and (c) low temperature cracking.

A.2.1 *Bottom up cracking*

A.2.1.1 Cracks may initiate at the bottom of any bound layer due to fatigue phenomenon reducing the effective layer thickness causing the cracks to progress and move upwards with repeated application of traffic loads. When the whole layer cracks, the crack progresses into the upper layer and will eventually appear on the surface of the pavement as alligator cracks. Mixes should have adequate flexural tensile strength and should be sufficiently flexible at intermediate temperatures at which the traffic loads (except the very small proportion of traffic which is applied when the pavement has peak summer or peak winter temperatures) to resist fatigue cracking caused by repeated flexure under traffic loads. Stiffer mixes usually have larger flexural tensile strength compared to the softer ones. However, the higher stiffness is usually associated with more brittleness. The fatigue cracking in bituminous layers has been addressed in these guidelines using a performance model which gives limiting tensile strain value for a given design traffic level and for a selected mix.

A.2.1.2 The fatigue cracking susceptibility of the bituminous layer can be reduced by controlling the flexural tensile strains at the bottom of the bituminous layer. This can be done by (i) providing a strong support from the underlying layers which reduces the deflection in the bituminous layer (ii) using stiffer bituminous mix which reduces the tensile strain in the material and (iii) using a mix that is adequately elastic to recover from damage.

A.2.1.3 A strong subgrade is essential for giving firm support to the upper pavement layers. The elastic modulus of the subgrade (required as input for analysis using linear elastic layered theory) is recommended to be estimated from its CBR value using the empirical equations given in the guidelines. When there is significant difference in the mechanical properties of the material used in the prepared subgrade compared to the material used in the embankment, it is proposed to estimate an equivalent subgrade property (effective modulus) for use in design. These guidelines recommend the use of subgrades with a minimum effective CBR of 5% for roads with more than 450 commercial vehicles per day. These Guidelines also restricts the value of the effective modulus of the subgrade that can be used for design to a maximum value of 100 MPa.

A.2.1.4 Since determining the resilient (elastic) modulus of the granular material used in the sub-base and base layers requires sophisticated equipment and skill, it is proposed to estimate the modulus value of the granular layer from a widely used empirical equation for which the elastic modulus value of the supporting layer and the thickness of the granular layer are inputs.

A.2.1.5 In situations where granular materials are placed over cementitious materials, e.g. granular base over cementitious sub-base and granular crack relief layer over cemented bases, the layer M_r cannot be estimated directly using the same empirical equation as mentioned above. Though models are available in literature for estimating the modulus of granular layer based on the state of stress prevailing in the material, no such rigorous analysis is proposed in these guidelines for estimating the resilient modulus of the granular materials. A resilient modulus value of 450 MPa is proposed for the crack relief layer of WMM placed over CTB. In case of a granular base placed over cementitious sub-base (CTSB), the recommended values are 300 MPa and 350 MPa for natural gravel and crushed rock respectively.

A.2.1.6 The modulus values of cemented materials are usually estimated from their Unconfined Compressive Strength (UCS) values (if the flexural modulus cannot be determined directly). The values estimated from the laboratory measured UCS are not adopted directly for analysis and design for two reasons, (i) these are laboratory values of the UCS tested on 'un-cracked' material samples while in the field, the material, if used in the sub-base, will start cracking immediately on application of the construction traffic and progressively loses its strength during construction as well as 'in-service' stages till it reaches the fully cracked terminal condition when it will behave more like a granular material rather than a rigid layer, (ii) higher the UCS wider will be the cracks, which have the chance of reflection into the overlying layer leading to an undesirable situation for pavement performance. Therefore, lower UCS materials are targeted to use in pavement layers.

A.2.1.7 The design elastic modulus of cemented sub-base materials is capped at a relatively low value of 600 MPa because of the possibility of the layer getting cracked right from the start. As far as the cementitious bases are concerned, the design modulus is recommended as 5000 MPa. These recommended values of modulus values are subject to the condition that the laboratory values of the UCS are within the ranges specified in these guidelines.

A.2.1.8 Testing the durability of the cementitious base materials is compulsory. This is because the low UCS materials have small quantity of cement binder and there may be the likelihood of inadequate binding of the materials, especially if the proportion of fines in the materials is high. The loss of weight in the 'wetting- drying' or 'freeze-thaw' test as relevant will reveal whether the cemented material will be durable when subjected to moisture cycles and other adverse climatic conditions.

A.2.1.9 The cementitious bases have to be analysed for cumulative fatigue damage as recommended in these Guidelines. Fatigue performance models have been recommended in these guidelines for cemented bases. Flexural strength (modulus of rupture) of the CTB material is necessary for carrying out the cumulative fatigue damage analysis. The guidelines recommend that the flexural strength of the cement treated materials may be taken as 20 per cent of the UCS value.

A.2.1.10 Where Reclaimed Asphalt Pavement (RAP) material (with or without addition of virgin aggregate), stabilised with foamed bitumen or emulsion is used as base, a conservative value of 800 MPa is recommended as the modulus value of the material for design provided the RAP mixes are designed and tested as per the procedure recommended in these Guidelines.

A.2.1.11 The resilient moduli of bituminous mixes used in the surface and base/binder layers are important inputs affecting the distribution of stresses and strains in different layers of the pavement, rutting and top-down cracking resistance of the upper layers and bottom-up cracking of the lower bituminous layer. The resilient modulus of bituminous mix varies over a wide range depending upon the aggregate gradation selected and the grade of bitumen used besides other influencing conditions. The temperature for which the M_r value should be considered for design is 35°C and the recommended test procedure is as in ASTM D4123.

A.2.2 *Top down cracking*

A.2.2.1 At the instance when the tyres come in contact with the road surface, they expand laterally and push the pavement surface at their edges. At the next instance when the tyre moves over, the laterally pushed surface should be elastic enough to pull itself back. If it is not, the surface will crack at the wheel edges along the longitudinal direction and the crack will propagate downwards from the surface. Another reason for top down cracking is the age hardening of bitumen. With age and exposure to sun light and Ultra Violet rays, the volatiles in bitumen are lost and the binder becomes hard and brittle, which significantly increases the cracking susceptibility of the material.

A.2.2.2 The objective of design for controlling top down cracking should be to use mixes that can accommodate more bitumen to have thicker films which reduce the rate of ageing, to minimize the effect of ageing by using ageing resistant modified binders in the surfacing course, to improve the visco-elastic properties of the binder by using binders that have better elastic recovery. These Guidelines recommend Stone Matrix Asphalt (SMA) and Gap Graded Rubberised Bitumen (GGRB) and Bituminous Concrete (BC) with modified binders, for high traffic (more than 50 msa) roads. In other cases, stiff grade binders or modified binders are considered suitable for surface course mixes.

A.3 Rutting in Bituminous Pavements

A.3.1 Rutting in pavement occurs in two ways: (a) Due to deformation in subgrade and other unbound layers (granular sub-base and base and (b) due to rutting in bituminous layer. The guidelines provide limiting strain criteria for controlling rutting in subgrade. Even through no separate criteria are included in the guidelines for rutting in the granular layers, controlling the vertical compressive strain on top of subgrade indirectly results in the control of strains in the upper granular layers. Larger elastic strains in the subgrade and unbound granular layers (which are calculated by linear elastic layered theory) are generally expected to produce larger plastic strains. Thicker bituminous layers and stronger sub-bases/bases (such as CTSB and CTB) reduce the subgrade strains significantly.

A.3.2 Even if the subgrade or unbound granular layers do not undergo rutting, the bituminous layers may do. This happens in various situations such as when the bituminous layers are not initially properly compacted and undergo large secondary compaction during their service life, the binder used is of a softer grade, has less elasticity, high pavement temperatures and high wheel load stresses. It is necessary to use sufficiently stiffer mixes with binders that will have less plastic deformation at high temperatures and high stresses, especially in the upper layers. At lower depths, the stresses as well as the temperatures will be less compared to the surface layers and thus the lower bituminous layers are less susceptible to rutting.

A.4 Structural Analysis of Pavement

A.4.1 These Guidelines continue to follow the Mechanistic-Empirical approach for pavement analysis as in its previous two versions. The stresses and strains in the pavement layers are analysed by the software IITPAVE, which requires inputs from users in terms of number of layers, their thicknesses and elastic moduli. Standard loading of 80 kN acting over four wheels (two dual wheel sets on each side of the axle) at 0.56 MPa uniform contact pressure is considered for the analysis. For evaluating the CTB bases, a contact pressure of 0.80 MPa shall be considered.

A.4.2 The trial pavement composition and layer thicknesses are selected and the stresses and strains at the critical locations are computed by running the IITPAVE software. The permissible strains are obtained from the fatigue and rutting models, for a given design traffic (csa). If the computed strains are larger than those derived from the model (limiting strains), the trial composition and layer thicknesses are changed until the values come within the permissible limits.

A.5 Effect of Climate and Environment on Pavement Performance

A.5.1 The discussion so far has been on the response of the pavement to load repetitions and on the design of pavement to limit the cracking and rutting. Climate and environment are other factors, which can affect the performance of pavements.

A.5.2 Water entering the cracks from bottom (bottom up cracks), top (top down cracks) or sides (shoulders or medians) may strip the bitumen leading to loss of bond between aggregates and bitumen, and may reduce the strength of granular and subgrade layers.

A.5.3 Binders age when exposed to environment and lose volatiles, harden, become brittle and then crack. Thus, it is necessary to use an appropriate grade of bitumen, with modifications if required, in the surfacing layer to make it resistant to the oxidation of bitumen.

A.5.4 At high temperatures, the binder and hence the mix becomes softer and is susceptible to rutting. At low temperatures, the mix is likely to become brittle and is susceptible to cracking. Hence, binders that are less temperature susceptible and have adequate properties at high as well low service temperatures are expected to yield better performance.

A.5.5 The resilient modulus of bituminous layers is quite sensitive to temperature. At high temperatures, the modulus is low and at low temperatures it is high. The maximum pavement temperature is important for understanding which grade of binder would be the most suitable for designing bituminous mixes. One model, based on research carried elsewhere, has the potential to predict the average seven day maximum surface temperature of the pavement from the corresponding air temperatures.

$$T_{20\,mm} = [(T_{air} - 0.00618Lat^2 + 0.2289Lat + 42.2) * (0.9545)] - 17.78 \qquad \text{(A-1)}$$

Where, T_{20mm} = Pavement temperature (°C) at a depth of 20 mm

Lat = Latitude of the place

T_{air} = Average of maximum temperatures (°C) of seven days

Based on this equation and for the maximum temperatures prevailing in different parts of the country, it is reasonable to consider that in most parts of the country, the maximum pavement temperature may reach around 70°C.

A.5.6 The appropriate grade of binder for a given project site should ideally be the one that is suitable for the range of variation in the pavement temperature. Even though there appears to be some gap in the existing standards with regard to the suitability of binders in extreme temperature conditions that are likely to prevail in the country, it is recommended that the grade suitable for temperatures nearest to the specified maximum could be adopted. Very broadly, the stiffest grade of available bitumen should be used where the pavement temperature is expected to rise above 60°C and the softer grades under low temperature conditions.

A.6 Mix Design

A.6.1 Bituminous binder/base courses and the surfacing courses have different requirements. The base is subjected to flexural tension and, therefore, needs to have sufficiently large stiffness (modulus) to reduce strains. Larger stiffness is usually achieved with dense aggregate grading as in DBM. Fatigue cracking starts at the bottom of the base. The mix also has to be sufficiently flexible to be more compliant with the deflections to which the layer will be subjected to under traffic loads. The fatigue life of the layer/mix can be increased by increasing the per cent of bitumen in the mix, but the dense grading of DBM does not allow enough void space to accommodate more bitumen without reducing the air voids. More bitumen and less air voids in the DBM would increase the fatigue life. Top down cracking starts at the surfacing at the edges of the wheel because the inflation pressure of the tyres at the contact surface deforms it across the wheel path and the mix should be elastic enough to recover this deformation after the passage of the wheels. The binder in the surfacing layer should, therefore, have high elastic recovery. Surfacing is also exposed to atmosphere and thereby to ageing. Therefore, the surfacing material should be age resistant. The surfacing is also exposed to water damage by stripping or displacement of the bitumen film by water, and therefore, it should be resistant to water damage.

A.6.2 *Some considerations for design of bituminous mix for base layer*

A high resilient modulus of DBM (typical base/binder course mix) should be targeted in the design, which in comparison to mixes having low or moderate resilient modulus values, will result in smaller tensile strain and less plastic strain under the same set of loading and hence will result in smaller DBM layer thicknesses. Thus, a higher resilient modulus mix will be more appropriate to resist both cracking (unless the mix becomes too brittle) and rutting.

A high resilient modulus mix can be achieved by a strong granular skeleton of aggregates represented by their grading. DBM grading-I having higher maximum nominal size of aggregates will have a stronger aggregate structure compared to DBM Grading-II of MoRTH. The choice between the two aggregate gradings, however, is also dependent upon the layer thickness, which should not be less than 2.5 times the maximum nominal size of the aggregate.

The lower layer DBM mix has to be rich in bitumen and low in air voids. The lower layer DBM, subject to the thickness and nominal size limitations, should be in Grading-I. The larger size fractions of aggregates and lower surface area would enable more void space to accommodate additional quantity of bitumen and thicker coating of aggregate particles by bitumen. The likelihood of rutting of the layer is minimal for two reasons, first because the lower layer is subject to lower stresses as the intensity of the load decreases with depth, and secondly the maximum temperatures of the bottom bituminous layer will be significantly smaller compared to the maximum temperatures applicable for the surface layer. Also, the degree of secondary compaction in the bottom layer will be less due to the fact that the bottom layer will have more confining stresses than the upper layers.

A part of the quantity of bitumen used in the mix is lost in the aggregate pores where the aggregates have porosity even though it is within permissible limits. This will reduce the effective quantity of bitumen in the mix, which might make the mix deficient in bitumen. It is necessary that the binder quantity lost due to absorption by aggregates should be carefully estimated during the mix design process.

Ingress of water into the DBM layer from shoulder and median needs to be prevented by taking adequate measures so that stripping and loss of bond between aggregates and Bitumen does not happen.

A.6.3 *Some considerations for design of bituminous mixes for surfacing layer*

Surfacing layer should have good elastic recovery property, which means that the binder should have less plastic deformation at higher temperatures. The surfacing layer should have adequate binder to make it durable. Selecting surface mixes in which more binder can be used has three main advantages; first, it will provide better bonding of aggregate and binder; second, on exposure to atmosphere it will resist the effects of oxidation and ageing, (with the resulting reduction in the top-down cracking susceptibility) and third, resistance to moisture damage.

For accommodating more binder, the aggregates should have more void space to accommodate the additional binder. This is somewhat difficult if the grading is a dense one as used in BC. The grading has to be opened up while retaining the granular load bearing skeleton that gives strength. Stone Matrix Asphalt (SMA) and Gap-Graded Rubberised Bitumen (GGRB) are examples of such mixes. These Guidelines strongly recommend SMA and GGRB as surfacing layers and BC with modified binders on all important roads having design traffic more than 50 msa.

A.7 Tests and Design Documentation

A.7.1 Design has to be based on a number of tests conducted in accordance with the procedures indicated in the main document at the appropriate places. The tests on works and their frequencies are enumerated in Section 14. The designer has to plan all the required tests at different points of time such as when selecting material sources, at the time of delivery of materials, before using the material for preparation of specimens, at the time of testing of specimens; and at different places such as at the supplier's premises, in the laboratory, in the stock yards/storage tanks, in the mixing plants, in the field, etc.

A.7.2 After material sources are selected, the designer needs to make sure that the supply from the source will be available for the entire project, otherwise the design has to be changed with change in material source. Conformity of all the material ingredients to the relevant specifications and the procedures to these Guidelines need to be ensured.

A.7.3 A design documentation comprising the complete design including the drawings, sketches, plans, assumptions made, if any, time and location referenced test results that the design is based on has to be prepared and made available to the Project Authority for monitoring the performance of the designed pavement over time.

A.8 Performance Monitoring

These Guidelines strongly recommend that the Project Authorities monitor over time the performance of the designed pavement as laid in the field to validate the adopted design and to further refine the models and the procedures used in the design. This should be done by

i) Measuring a set of pavement performance parameters: surface irregularity, rutting, alligator cracking, top down cracking.

ii) Observing other kinds of distresses: age hardening, raveling, potholes, bleeding, etc.

iii) Investigating the distresses observed, if any; core samples of distressed portions.

iv) Gathering the air and pavement temperature data.

ANNEX-I

INSTALLATION AND USE OF IITPAVE SOFTWARE

I.1 Salient Features of IITPAVE

IITPAVE software has been developed for the analysis of linear elastic layered pavement system. The stresses, strains and deflections caused at different locations in a pavement by a uniformly distributed single load applied over a circular contact area at the surface of the pavement can be computed using this software. The effect of additional loads (which should also be uniformly distributed loads over circular contact areas) was considered using superposition principle. The single vertical load applied at the surface is described in terms of (a) contact pressure and radius of contact area OR (b) Wheel load and contact pressure OR (c) Wheel load and radius of contact area. For IITPAVE, wheel load and contact pressure are the load inputs. The pavement inputs required are the elastic properties (elastic/resilient moduli and Poisson's ratio values of all the pavement layers) and the thicknesses of all the layers (excluding subgrade). IITPAVE software, in its current version, can be used to analyze pavements with a maximum of ten layers including the subgrade. If the number of layers in the pavement is more than ten, different layers of similar nature (eg. granular, bituminous) can be combined and considered as one layer. Cylindrical co-ordinate system is followed in the program. Thus, the location of any element in the pavement is defined by (a) depth of the location of the element from the surface of the pavement and (b) the radial distance of the element measured from the vertical axis of symmetry (along the centre of the circular contact area of one wheel load).

I.2 Installation of IITPAVE

For installing the software, copy the **IRC_37_IITPAVE** folder supplied along with these guidelines into your system and install Java (if not already installed in your computer) by clicking on **jre-7u2-windows-i586.exe** file. Your system needs to be connected to the internet for doing this.

I.3 Using IITPAVE for Analysis of Flexible Pavements

The following steps may be followed for analyzing flexible pavements using IITPAVE

(a) Open **IRC_37_IITPAVE** folder.

(b) Double-click on **IITPAVE_EX.exe** file in the IRC_37_IITPAVE folder. IITPAVE start screen will appear as shown in **Fig. I.1.**

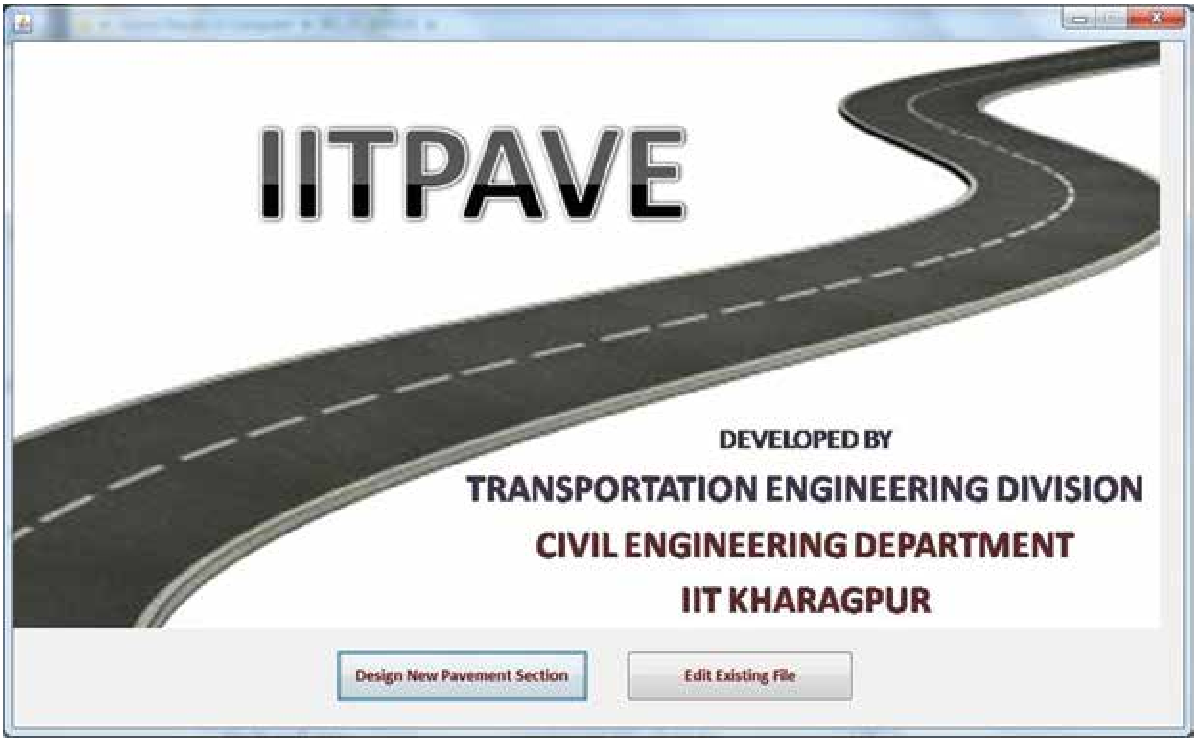

Fig. I.1 Screenshot of IITPAVE start screen

(c) Click on **Design New Pavement Section** to give inputs for the analysis of the selected pavement section.

(d) The inputs to be entered are:

(i) ***Number of pavement layers*** including subgrade (if all the bituminous layers are taken as one bituminous layer and all the granular layers are taken as one layer, then the number of layers is 3 (bituminous layer, granular layer and subgrade).

(ii) ***Resilient modulus/Elastic modulus*** values of all the layers in MPa

(iii) ***Poisson's ratio*** values of all the layers

(iv) ***Thicknesses*** (in mm) of all the layers except subgrade.

(e) **Single wheel load:** For the purpose of calculation of critical strains such as vertical compressive strain on top of subgrade, horizontal tensile strain at the bottom of bituminous layer and horizontal tensile strain at the bottom of cement treated layers, since the analysis is done for a standard axle of 80 kN, a single wheel load of 20000 (N) is given as input. For carrying out cumulative fatigue damage analysis of CTB layers, the tensile stress/strain at the bottom of the CTB layer has to be calculated for different axle loads. For this, the IITPAVE will be run with different single wheel loads corresponding to the axle load considered, For example, if tensile stress due to a single axle load (with dual wheels) of 100 kN is to be calculated, a single wheel load of 25,000 (N) is given as input. For estimating the effective subgrade strength as per para 6.4 of the guidelines, select a single wheel load of 40,000 (N).

(f) **Tyre (contact) pressure:** For calculation of the vertical compressive strain on top of the subgrade and the horizontal tensile strain at the bottom of bituminous layer, a contact pressure of 0.56 MPa is considered. For analyzing the tensile strain or tensile stress at the bottom of the CTB base for carrying out fatigue damage analysis of CTB bases using equations 3.5 to 3.7, the contact pressure suggested is 0.80 MPa. The bituminous layer bottom-up fatigue cracking and subgrade rutting performance models have been developed/calibrated with the strains calculated with standard axle (80 kN) loading and a contact pressure of 0.56 MPa and hence, these inputs should not be changed.

(g) The **number of locations** in the pavement at which stress/strain/deflection has to be computed. This input can be entered through a drop down menu.

(h) For the locations selected for the analysis, the values of **depth** (mm) from the pavement surface and the **radial distance** (mm) from the centre of the wheel load contact area are to be given.

(i) IITPAVE Software provides the option to carryout analysis for a **single wheel load or for a dual wheel load set** (two wheels at a centre to centre spacing of 310 mm) by selecting 1 or 2 respectively from the drop down menu next to “Wheel Set”. For design of pavements, select “Dual Wheel set” option. For estimating the effective subgrade strength as per the procedure given in para 6.4, select single wheel.

Fig. I.2 shows an abridged screenshot of the input page of IITPAVE

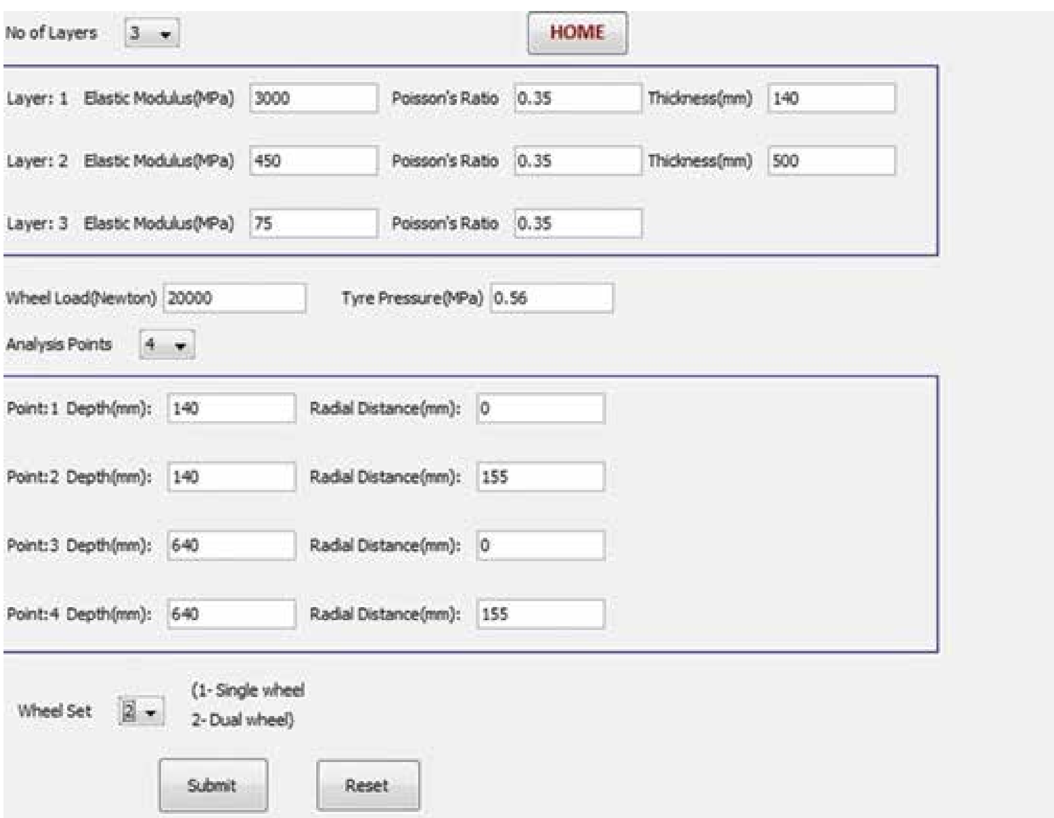

Fig. I.2 Abridged Screenshot of Input Page of IITPAVE

(j) Inputs can also be given through an input file. The name of the input file can be selected by clicking on '**Edit Existing File**' option which appears on the IITPAVE Start Screen.

(k) After all the inputs are entered, submit them by Clicking on "**Submit**". To change the data submitted, use "**Reset**" option

(l) After successfully submitting the inputs use the "**RUN**" options which will appear next to "**Reset**" after the inputs are submitted

(m) **Fig. I.3** shows the screenshot of the output page showing the output for the input data appearing in the screenshot of the input page given under **Fig. I.2.**

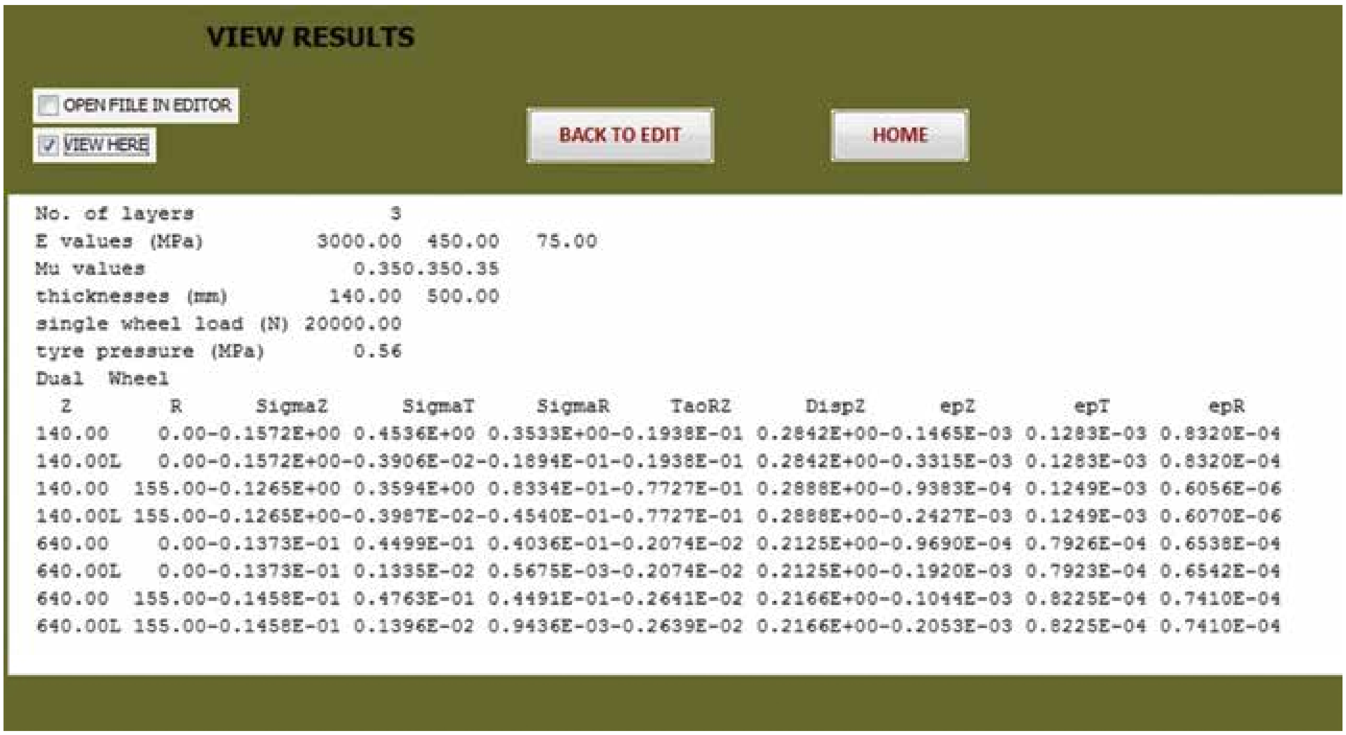

Fig. I.3 Abridged screenshot of the output page

The output screen displays options for the mode of output to be viewed either through "Open file editor" or "view here". Once either of the options is chosen the output page reports all the input data and gives the computed values of identified stresses, strains and deflections for the locations (represented by the depth (Z) of the location measured from pavement surface, and the radial distance (R) of the location measured from the centre of the circular contact area of the load) selected. The mechanistic parameters reported in the output page are: vertical stress (SigmaZ), tangential stress (SigmaT), radial stress (SigmaR), shear stress (TaoRZ), vertical deflection (DispZ), vertical strain (epz), horizontal tangential strain (epT), and horizontal radial strain (epR)

For locations on the interface of two layers, the analysis will be done twice: (a) assuming the elastic properties (elastic modulus and Poisson's ratio) of the layer above the interface and then (b) with the elastic properties of the layer below. The second set of results, for the layer below the interface, are identified in the output by the suffix "L" appearing after the depth (Z) value.

For the results of pavement analysis presented in the screen shot of the output page, the critical mechanistic parameter, horizontal tensile strain (ε_t), will be the largest of the tangential and radial strains at the bottom of the bituminous layer (layer above the interface between bituminous layer and granular layer) computed at two radial distances of ‘0’ and ‘155’. Thus, horizontal tensile strain (ε_t) will be taken as 0.0001283 ($0.1283*10^{-3}$) which is the maximum out of the four strain values (tangential and radial at ‘0’ and ‘155’ mm radial distances), i.e., 0.0001283, 0.0001249, 0.00008320 and 0.00006056 (shown in rectangular boxes). Note that the values have been taken from the upper line of the two sets of results reported for the interface between the bituminous layer and granular layer (at a depth of 140 mm). Similarly, for this pavement, vertical compressive strain (ε_v) will be taken from the results corresponding to the lower line (with “L”) of the two sets of results available for the interface between granular layer and subgrade. Thus, the vertical compressive strain (ε_v) value of 0.0002053 ($0.2053*10^{-3}$) which is the larger of the two strain values obtained for the interface between the subgrade and the granular layer (at radial distances of ‘0’ and ‘155’ mm), i.e., 0.0002053 and 0.000193 (shown in rectangular boxes).

Positive stresses and strains are “tensile” whereas Negative stresses and strains are “Compressive”. Only the absolute values without the (+) or (-) sign will be used in the performance models given by equations 3.1 to 3.6.

ANNEX-II

WORKED OUT EXAMPLES FOR PAVEMENT DESIGN

II.1 Estimation of Effective Subgrade Modulus/CBR

Problem: If the CBR of the soil used in the upper 500 mm of embankment is 8% and the CBR of the borrow soil used for preparing the 500 mm thick compacted subgrade above embankment is 20%, what is the effective subgrade Modulus/CBR for design of flexible pavement?

Solution:

Elastic modulus of the prepared (upper 500 mm) embankment soil = $17.6*(8)^{0.64}$ = 66.6 MPa

Elastic modulus of the select borrow material = $17.6*(20)^{0.64}$ = 119.7 MPa

Consider a two-layer elastic system consisting of 500 mm of select borrow soil of modulus 119.7 MPa and the semi-infinite embankment soil of modulus 66.6 MPa as shown in **Fig. II.1**.

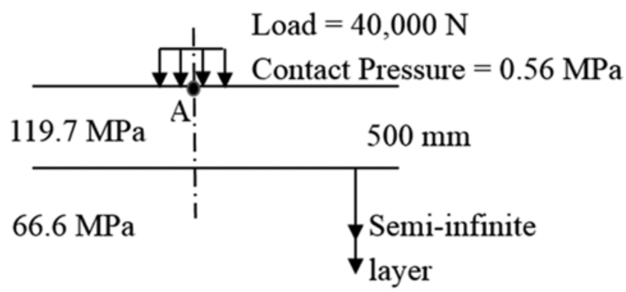

Fig. II.1 Two-Layer Pavement System with Subgrade and Embankment

Consider the Poisson's ratio value of both the layers to be 0.35. Apply a single load of 40,000 N at a contact pressure of 0.56 MPa. Radius of circular contact area for this load and contact pressure = 150.8 mm. Calculate surface deflection at the centre of the load (Point A in **Fig. II.1**) using IIPAVE (no of layers = 2; elastic moduli of 119.7 MPa and 66.6 MPa; Poisson's ratio of 0.35 for both the layers; thickness of 500 mm for upper layer; single wheel load of 40000 N, analysis points = 1; Depth = 0 mm; Radial distance = 0 mm. For this input data, surface deflection = 1.41 mm from IITPAVE.

For an equivalent single layer system, the modulus value of the single layer which will produce the same surface deflection of 1.41 mm for the same load and for a Poisson's ratio of 0.35

$$= [2(1-\mu^2)p\alpha]/\delta$$

$$= [2(1-0.35^2)*0.56*150.8]/1.41=105.10 \text{ MPa}$$

As per these design guidelines, the effective modulus value will be limited to 100 MPa for design purpose. The corresponding CBR (using equation 6.3) is 15.82 % for 105.1 MPa. For a restricted modulus value of 100 MPa, the corresponding effective CBR can be reported as 15.1 %. The equivalent single layer subgrade which gave the same surface deflection as that given by the two-layer system is shown in **Fig. II.2**.

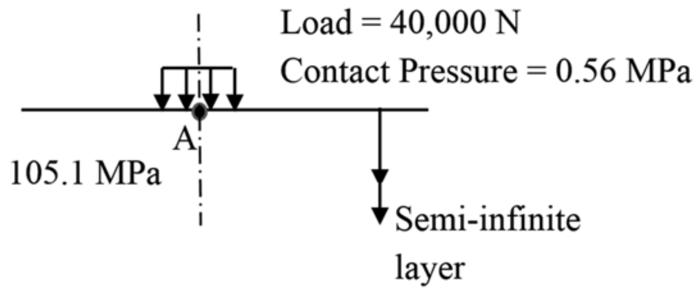

Fig. II.2 Equivalent (effective) Subgrade System

II.2 Design Example to check the Adequacy of Granular Sub-base Thickness

Problem: Determine the thickness of the GSB layer required over a foundation having an effective CBR of 5 % to carry construction material required for construction of WMM layer over GSB.

Solution:

- Effective Subgrade CBR = 5 %. Hence, resilient modulus of subgrade may be taken as 50 MPa (10X5 = 50 MPa)
- Select a trial thickness of 150 mm for the granular sub-base
- Assume that 200 repetitions of dumpers will be required for laying 250 mm thick WMM layer over a single lane of length 2.0 km stretch
- Assume that the load on the rear tandem axle of the dumper is 240 kN and that on the front axle is 80 kN.
- Consider the 240 kN tandem axle as two 120 kN single axles
- Thus, the VDF of a dumper = $2x(120/80)^4 + (80/65)^4 = 12.41$
- Total standard axle load repetitions = 200 x 12.41 = 2483. Design construction traffic to be considered is the traffic thus estimated or 10,000 standard axles, whichever is more
- Allowable vertical subgrade strain for 10000 repetitions of SA = 2433×10^{-6} (=0.002433)
- Computed (using IITPAVE) vertical subgrade strain for a standard axle for the two-layer system (subgrade and granular sub-base) with Subgrade modulus = 50 MPa, granular sub-base modulus = $0.2*((150)^{0.45})*50$ = 95 MPa and Poisson's ratio of 0.35 for both layers, is $4324 \times 10^{-6} > 2433 \times 10^{-6}$
- Hence, the thickness of 150 mm GSB is deficient. Increase the thickness to 250 mm
- Mr of the GSB = 119 MPa (using Equation 7.1)
- The computed vertical subgrade strain= 2179×10^{-6} (shown in the box in **Fig. II.3**) < 2433×10^{-6} hence safe (**Fig. II.3** shows the IITPAVE output for this analysis).

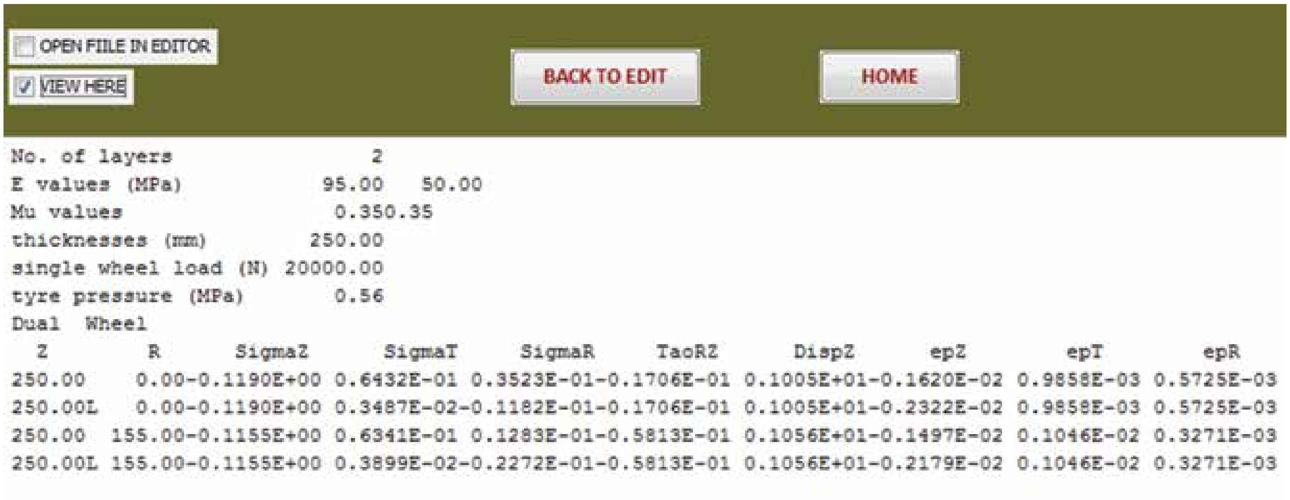

Fig. II.3 Screenshot of IITPAVE for example II.2

II.3 Design of Bituminous Pavement with Granular Base and Sub-base

Problem: Design a bituminous pavement with granular base and sub-base layers using the following input data

(i) Four lane divided carriageway

(ii) Initial traffic in the year of completion of construction = 5000 cvpd (two-way)

(iii) Traffic growth rate per annum = 6.0 per cent

(iv) Design life period = 20 years

(v) Vehicle damage factor = 5.2 (taken to be the same for both directions)

(vi) Effective CBR of subgrade estimated as per the procedure given in example II.1 = 7 %

(vii) Marshall mix design carried out on the bituminous mix to be used in the bottom bituminous layer (DBM) for an air void content of 3 % resulted in an effective bitumen content (by volume) of 11.5 %

Solution:

(i) Lateral Distribution factor = 0.75 (for each direction)

(ii) Initial directional traffic = 2500 CVPD (assuming 50 per cent in each direction)

(iii) Vehicle Damage Factor (VDF) = 5.2

(iv) Cumulative number of standard axles to be catered for in the design

$$N = \frac{2500 \times 365 \times ((1 + 0.06)^{20} - 1)}{0.06} \times 0.75 \times 5.2 = 131 \text{ msa}$$

(v) Effective CBR of subgrade = 7 %

(vi) Effective resilient modulus of Subgrade = $17.6(7.0)^{0.64}$ = 62 MPa (less than 100 MPa, the upper limit)

(vii) Since the design traffic is more than 50 msa, provide a SMA/GGRB or BC with modified bitumen surface course and DBM binder/base layer with VG40 with viscosity more than 3600 Poise (at 60°C)

(viii) Select a trial section with 190 mm total bituminous layer (provide 40 mm thick surface layer, 70 mm thick DBM-II, 80 mm thick bottom rich DBM-I); 250 mm thick granular base (WMM) and 230 mm thick granular sub-base (GSB). Total thickness of granular layer = 480 mm

(ix) Resilient modulus of the granular layer = $0.2 \times (480)^{0.45} \times 62$ = 200 MPa

(x) Use 90 % reliability performance models for subgrade rutting and bituminous layer cracking (design traffic > 20 msa)

(xi) Allowable vertical compressive strain on subgrade for a design traffic of 131 msa and for 90 % reliability (using equation 3.2) = 0.000301 (0.301×10^{-03})

(xii) Allowable horizontal tensile strain at the bottom of bituminous layer for a design traffic of 131 msa, 90 % reliability, air void content of 3 % and effective binder volume of 11.5 %, and a resilient modulus of 3000 MPa for bottom rich bottom DBM layer (DBM-I) (using Equation 3.4) = 0.000150 (0.150×10^{-03})

(xiii) Analyse the pavement using IITPAVE with the following inputs (elastic moduli: 3000 MPa, 200 MPa, 62 MPa, Poisson's ratio values of 0.35 for all the three layers, layer thicknesses of 190 mm and 480 mm). Computed Horizontal tensile strain = 0.000146 < 0.000150. Hence OK

(xiv) Computed vertical compressive strain = 0.000243 < allowable strain of 0.000301. Hence OK

(xv) A screenshot of the IITPAVE output generated for this problem is given as **II.4**.


```
No. of layers              3
E values (MPa)       3000.00  200.00   62.00
Mu values               0.350.350.35
thicknesses (mm)      190.00  480.00
single wheel load (N) 20000.00
tyre pressure (MPa)      0.56
Dual  Wheel
  Z         R      SigmaZ      SigmaT      SigmaR      TaoRZ       DispZ       epZ         epT         epR
190.00    0.00-0.7173E-01 0.5538E+00 0.4471E+00-0.1145E-01 0.3540E+00-0.1407E-03 0.1408E-03 0.9279E-04
190.00L   0.00-0.7173E-01 0.8741E-03-0.6243E-02-0.1145E-01 0.3540E+00-0.3493E-03 0.1408E-03 0.9278E-04
190.00  155.00-0.6920E-01 0.5315E+00 0.3381E+00-0.3037E-01 0.3635E+00-0.1245E-03 0.1458E-03 0.5875E-04
190.00L 155.00-0.6919E-01 0.6577E-03-0.1224E-01-0.3037E-01 0.3635E+00-0.3257E-03 0.1458E-03 0.5875E-04
670.00    0.00-0.1373E-01 0.2021E-01 0.1828E-01-0.1887E-02 0.2637E+00-0.1360E-03 0.9308E-04 0.8008E-04
670.00L   0.00-0.1373E-01 0.1156E-02 0.5746E-03-0.1886E-02 0.2637E+00-0.2312E-03 0.9291E-04 0.8025E-04
670.00  155.00-0.1442E-01 0.2115E-01 0.2006E-01-0.2276E-02 0.2684E+00-0.1442E-03 0.9590E-04 0.8852E-04
670.00L 155.00-0.1442E-01 0.1198E-02 0.8594E-03-0.2276E-02 0.2684E+00-0.2443E-03 0.9590E-04 0.8852E-04
```

Fig. II.4 Screen shot of IITPAVE out for example II.3

II. 4 Illustration of computation of Cumulative Fatigue Damage in Cement Treated Base (CTB) Layer

Problem: Compute the cumulative fatigue damage in CTB layer for the following combination of pavement constructed over a subgrade having 7 % effective CBR.

(i) 250 mm cement treated granular sub-base (CTSB)

(ii) 120 mm cement treated base (CTB)

(iii) 100 mm granular crack relief layer

(iv) 100 mm bituminous layer

(v) Flexural strength of cemented treated base = 1.4 MPa

Use the following axle load spectrum data which shows the expected (during the design period) repetitions of different categories of axle (single, tandem and tridem) with different axle load ranges.

Also check the CTB layer for construction traffic

Single Axle Loads		Tandem Axle Loads		Tridem Axle Loads	
Axle Load Class (kN)	**Expected Repetitions**	**Axle Load Class (kN)**	**Expected Repetitions**	**Axle Load Class (kN)**	**Expected Repetitions**
185-195	70000	390-410	200000	585-615	35000
175-185	90000	370-390	230000	555-585	40000
165-175	92000	350-370	240000	525-555	40000
155-165	300000	330-350	235000	495-525	45000
145-155	280000	310-330	225000	465-495	43000
135-145	650000	290-310	475000	435-465	110000
125-135	600000	270-290	450000	405-435	100000
115-125	1340000	250-270	1435000	375-405	330000
105-115	1300000	230-250	1250000	345-375	300000
95-105	1500000	210-230	1185000	315-345	275000
85-95	1350000	190-210	1000000	285-315	260000
<85	3700000	170-190	800000	255-285	180000
		<170	3200000	<255	720000

Solution:

(i) Elastic Moduli values: Bituminous layer = 3000 MPa; granular crack relief layer = 450 MPa; CTB layer = 5000 MPa; CTSB = 600 MPa; subgrade = 62 MPa (effective CBR of 7 %)

(ii) Poisson's ratio values: Bituminous, crack relief layer and subgrade = 0.35; CTSB and CTB = 0.25

Note: This example has been given only to illustrate the procedure for computation of Cumulative fatigue damage in Cement treated base. For complete design, the pavement has to be checked for limiting strains for subgrade rutting, bituminous layer fatigue cracking and CTB cracking as per the methodology given in example II.2. The bottom tensile strain CTB layer also shall be calculated for a standard axle load of 80 kN and tyre contact pressure of 0.80 MPa.

(a) Cumulative fatigue damage analysis for Single Axles

Modulus of Rupture (flexural strength) of the cementitious base = 1.4 MPa;

Stress Ratio = Tensile stress at the bottom of the CTB due to the applied load/Modulus of Rupture

Fatigue life N_f is estimated using equation 3.6 of the guidelines.

Fatigue life consumed = expected repetitions of a particular axle load/fatigue life corresponding to that axle

Single Axle Load (kN)	Expected Single Axle Repetitions (n_i)	Tensile Stress at the bottom of CTB σ_t (MPa)	Stress Ratio (σ_t/ M_{Rup})	Fatigue Life (N_f)	Fatigue life Consumed (n_i/N_f)
190	70000	0.70	0.50	5.26E+05	0.13
180	90000	0.67	0.48	9.57E+05	0.09
170	92000	0.63	0.45	2.12E+06	0.04
160	300000	0.60	0.43	3.86E+06	0.08
150	280000	0.56	0.40	8.58E+06	0.03
140	650000	0.53	0.38	1.56E+07	0.04
130	600000	0.49	0.35	3.46E+07	0.02
120	1340000	0.46	0.33	6.30E+07	0.02
110	1300000	0.42	0.30	1.40E+08	0.01
100	1500000	0.39	0.28	2.54E+08	0.01
90	1350000	0.35	0.25	5.64E+08	0.00
85	3700000	0.33	0.22	8.41E+08	0.00
Cumulative Fatigue Damage in CTB due to Single Axles =					**0.48**

The tensile stress at the bottom of bituminous layer is computed using IITPAVE using the following inputs: 5 layers, Elastic moduli of 3000, 450, 5000, 600 and 62 MPa; Poisson's ratio values of 0.35, 0.35, 0.25, 0.25 and 0.35; layer thicknesses of 100, 100, 120 and 250 mm; Wheel load of 47500 (N) for the first axle load group (single axle with four wheels) of 190 kN (wheel load = 190000/4); Tyre (contact) pressure = 0.80 MPa; Analysis points = 2; Coordinates for the first point (depth of 320 mm at the interface between CTB layer and CTSB at the bottom of CTB) and radial distance of '0' mm (at the centre of one wheel load); Coordinates for the second point (depth of 320 mm and radial distance of '155' mm (on the axis of symmetry of the dual wheel set (the two wheels are at a centre to centre spacing of 310 mm from each other) at the centre of one wheel load); and wheel load set of 2 (dual wheel). The largest tensile stress at the bottom of CTB layer obtained from the radial and tangential stresses calculated at two radial distances '0' and '155' mm is taken for estimating the cumulative fatigue damage. For example, for a single axle of 190 kN, for the radial distance of '0' mm, tangential stress and radial stresses are 0.6436 MPa and 0.4986 MPa respectively. The corresponding stresses for 155 mm radial distance are 0.6995 MPa and 0.5561 MPa respectively. Hence, a value of 0.6995 MPa (0.70 MPa) has been considered as shown in the table above.

(b) Cumulative fatigue damage analysis for Tandem Axles

Tandem Axle Load (kN)	Expected Single Axle Repetitions (n_i) =Tandem axles X 2	Tensile Stress at the bottom of CTB σ_t (MPa)	Stress Ratio (σ_t/ M_{Rup})	Fatigue Life (N_f)	Fatigue life Consumed (n_i/N_f)
400	400000	0.73	0.53	2.28E+05	1.76
380	460000	0.70	0.50	5.26E+05	0.87
360	480000	0.67	0.48	9.57E+05	0.50
340	470000	0.63	0.45	2.12E+06	0.22
320	450000	0.60	0.43	3.86E+06	0.12
300	950000	0.56	0.40	8.58E+06	0.11
280	900000	0.53	0.38	1.56E+07	0.06
260	2870000	0.49	0.35	3.46E+07	0.08
240	2500000	0.46	0.33	6.30E+07	0.04
220	2370000	0.42	0.30	1.40E+08	0.02
200	2000000	0.39	0.28	2.54E+08	0.01
180	1600000	0.35	0.25	5.64E+08	0.00
170	6400000	0.31	0.22	1.25E+09	0.01
Cumulative Fatigue Damage in CTB due to Tandem Axles =					**3.79**

(c) Cumulative fatigue damage analysis for Tridem Axles

Tridem Axle Load (kN)	Expected Single axle Repetitions (n_i) = Tridem axles X 3	Tensile Stress at the bottom of CTB σ_t (MPa)	Stress Ratio (σ_t/ M_{Rup})	Fatigue Life (N_f)	Fatigue life Consumed (n_i/N_f)
600	105000	0.73	0.52	2.28E+05	0.46
570	120000	0.70	0.50	5.26E+05	0.23
540	120000	0.67	0.48	9.57E+05	0.13
510	135000	0.63	0.45	2.12E+06	0.06
480	129000	0.60	0.43	3.86E+06	0.03
450	330000	0.56	0.40	8.58E+06	0.04
420	300000	0.53	0.38	1.56E+07	0.02
390	990000	0.49	0.35	3.46E+07	0.03
360	900000	0.46	0.33	6.30E+07	0.01
330	825000	0.42	0.30	1.40E+08	0.01
300	780000	0.39	0.28	2.54E+08	0.00
270	540000	0.35	0.25	5.64E+08	0.00
255	2160000	0.31	0.22	1.25E+09	0.00
Cumulative Fatigue Damage in CTB due to Tridem Axles =					**1.02**

It can be seen that the total fatigue damage due to single, tandem and tridem axles is 0.48 + 3.79 + 1.02 = 5.29 > 1.0. Hence, the pavement is unsafe and cemented layer will crack prematurely. It can also be noticed that the Tandem axle weighing 400 kN causes maximum fatigue damage followed by Tandem axle of 380 kN. This highlights the importance of controlling overloading. It can be verified that the CFD will be less than 1.0 if the thickness of the CTB layer is increased from 120 to 160 mm.

Checking the CTB for construction traffic

(i) Assume that the gross weight of a three-axle dumper (rear tandem and front single) is 320 kN

(ii) Assume that the load on the rear tandem axle is 240 kN and that on the front steering axle is 80 kN.

(iii) Assume the 7-day flexural strength of CTB material to be 1.0 MPa (about 70 % of the flexural strength of 1.4 MPa)

(iv) Assume that there will be 70 dumper trips for laying 2.0 km of 100 mm thick one lane wide granular crack relief layer.

(v) Allowable flexural stress for 70 X 2 = 140 repetitions of a single axle of 120 kN (240/2) from Equation 3.6 = 0.795 MPa.

(vi) It can be verified that a CTB layer thickness of 160 mm (for the other pavement data given in this example) will be required. Analysing this as a Three-layer system with CTB (160 mm), CTSB (250 mm) and subgrade, the maximum tensile stress at the bottom of CTB layer due to 120 kN single axle works out to 0.767 MPa

II.5 Design of Bituminous Pavement with Reclaimed Asphalt Pavement (RAP) material treated with Foamed Bitumen/Bitumen Emulsion and Cemented Sub-base

Problem: Design a bituminous pavement with RAP material stabilised using foam bitumen as base material and a CTSB for the subgrade and traffic data given example II.2.

Solution:

(i) Design traffic = 131 msa

(ii) Consider a trial pavement combination of 100 mm thick bituminous layer, foam bitumen stabilised RAP base of 180 mm thickness and CTSB of 250 mm thickness

(iii) Elastic moduli: 3000 MPa for bituminous layer, 800 MPa for stabilised Rap layer, 600 MPa for CTSB, 62 MPa for subgrade (from example II.2)

(iv) Poisson's ratio for all the layers except CTSB = 0.35, For CTSB it is 0.25.

(v) Allowable subgrade vertical compressive strain (for 90 % reliability, 131 msa traffic) = 0.000301

(vi) Allowable horizontal tensile strain (90 % reliability, 3000 MPa mix modulus, mix design parameters of 3 % air void content and 11.5 % effective bitumen volume) = 0.000150

(vii) Computed vertical subgrade strain = 0.000148 < 0.000301. Hence, OK

(viii) Computed horizontal tensile strain in bituminous layer = 0.0001042 < 0.000150. Hence, OK

II.6 Worked out Design Example: Long-life Pavement

For design traffic of 300 msa or more, a long-life pavement, also termed as perpetual pavement, is recommended. If the tensile strain caused by the traffic in the bituminous layer is less than 70 microstrains (80 με at 35°C) as per tests conducted in laboratories at 20°C in US, the endurance limit of the material, the bituminous layer never cracks (Asphalt Institute, MS-4, 7th edition 2007). Similarly, if the vertical subgrade strain is less than 200 microstrain, rutting in subgrade will be negligible. For a pavement temperature of 35ε, the endurance limit is about 80με Design of such a pavement is illustrated here.

Problem: Design a long-life pavement for the following data for a 7% effective subgrade CBR

Solution 1: Consider a conventional pavement option with granular base and sub-base with 200 mm GSB and 250 mm granular base (WMM). Consider a trial bituminous layer of 310 mm thickness

Allowable subgrade strain	= 200 με.
Allowable tensile strain in the bituminous layer	= 80 με (for 35°C).
Modulus of the Bituminous layer	= 3000 MPa (VG40 Bitumen).
Elastic Modulus of subgrade	= 62 MPa (corresponding to CBR of 7 %).
Granular layer modulus	= $0.2(450)^{0.45}*62$ = 194 MPa

The computed strain in the bituminous layer is 80 με = 80 με and the vertical subgrade strain is obtained as 0.000153 < 200 με. (The analysis is done by IITPAVE software).

Solution 2: Consider a CTSB layer of 300 mm thickness, WMM layer of 150 mm and a bituminous layer thickness of 250 mm

Elastic moduli of layers are: 3000 MPa for bituminous layer, 350 MPa for granular layer (crushed aggregate) over (CTSB), 600 MPa for CTSB and 62 MPa for subgrade

The computed strain in the bituminous layer is 80 με = 80 με and the vertical subgrade strain is obtained as 0.000157 < 200 με.

Solution 3: Use of high Modulus binder

Consider a thickness of 190 mm for the high modulus mix

Modulus of Bituminous layer = 5500 MPa, WMM = 150 mm of modulus 350 MPa (crushed rock), CTSB of 300 mm thickness of modulus 600 MPa

The computed strain in the bituminous layer is 78 με < 80 με and the vertical subgrade strain is obtained as 0.000172 < 200 με.

Comparison with a Conventional design for 150 msa

Consider the same CBR = 7 %

WMM = 250 mm. GSB=250 mm

Consider a Bituminous layer of 200 mm with VG40 and modulus 3000 MPa, Granular layer modulus = 203 MPa

Computed tensile strain in Bituminous layer = 136 µε < 145 µε

Vertical subgrade strain is 224 µε < 292 µε

Table below shows the comparison of long-life pavement designs with conventional pavement design

Solution 1 Long life pavement	Solution 2 Long life pavement	Solution 3 Long life pavement	Conventional For a design life of 150 msa
Bituminous layer = 310 mm with VG40	Bituminous layer = 250 mm with VG40	Bituminous layer = 190 mm with hard bitumen	Bituminous layer = 200 mm with VG40
WMM = 250 mm	WMM = 150 mm	WMM = 150 mm	WMM = 250 mm
GSB = 200 mm	CTSB = 300 mm	CTSB = 300 mm	GSB = 250 mm

A long life pavement with WMM and GSB needs 310 mm of bituminous layer with VG40 binder while that with Cement Treated sub-base needs 250 mm of bituminous layer. 190 mm thickness is needed for the high modulus mix while 200 mm of bituminous layer is needed for 150 msa which may be attained in 15 to 20 years of life or even earlier. It can be seen that solution 3 can be most economical. Practitioners of pavement design should examine various options for pavement design discussed above for the efficient use of materials for a long life pavement. Low strength cementitious GSB permits use of marginal aggregates such as natural river gravels.

II.7 Stage Construction

In this type of construction, thickness of the WMM and GSB is provided for the full design period and the thickness of the bituminous layer is then determined for a shorter design period. Consider a design period of ten years for the example -1

Design traffic = 46.9 msa for ten years (131 msa for 20 years). The pavement shall not be allowed to be damaged to an extent of 60% only so that overlay will be effective in extending the life till the design period. Cumulative Fatigue Damage = 0.60, hence the pavement shall be designed for 46.9/0.60=78 msa If only 46.9 msa is selected for the pavement design, the pavement may suffer structural damage in the form of full depth cracking and periodical maintenance such as patching, crack sealing and micro-surfacing will not prevent bottom up crack because of short design period. Rehabilitation may be needed after ten years followed by application of wearing course. This is avoided if the design is done for the traffic mentioned above and FWD tests done after ten years is to be used for determination of overlay thickness.

II.8 Design options for Diversions

Pavement for temporary diversion

During the construction of culverts, widening of roads and construction of other structures, temporary diversion of highways is needed to allow smooth and safe movement of traffic. Poor quality of pavement of diversion leads to serious accidents. IRC:SP:84 recommends a design traffic of 10 msa for diversion road for four-lane highways. For other categories of single lane or two lane roads, design traffic may be computed from the traffic count for the likely duration of the diversion and minimum design traffic of 2 msa may be adopted for pavement design. Cement treated soil or aggregates with thin bituminous surfacing such as BC/SDBC using the subgrade strain criterion can be used for pavement design. Surface cracks should be sealed with hot bitumen as and when they arise. Heavily loaded commercial vehicles, construction vehicles carrying aggregates and dumpers carrying hot bituminous mixes and dry lean concrete mix, cause maximum distresses to any diversion. Another pavement composition can be cement treated aggregates (minimum 7-day UCS of 1.5 to 3 MPa), a 30 mm sand layer, precast concrete blocks 100 mm thick (IRC:SP:63). Joints between precast concrete blocks should be densely filled with sand and SS-2 bitumen emulsion with 50:50 dilution with water should be poured in the joints to prevent erosion by water. It will also prevent sucking away of jointing sand due to pneumatic wheels. Concrete blocks with bedding sand of 30 mm may be considered to have a modulus of 1000 MPa and the cement treated sub-base may be assigned a modulus of 600 MPa for pavement design. Concrete blocks can be reused for other works after the diversion is no more in use.

ANNEX-III

EXAMPLE CALCULATIONS FOR SELECTED PAVEMENT COMPOSITIONS

This section presents examples of the calculations and design outputs of IITPAVE in a tabular form. The examples are presented for 10% effective subgrade CBR case for the all the six pavement composition types and for six levels of design traffic for which thickness catalogues are given in section 12 of the guidelines.

The following design inputs are considered for the pavement analysis. It may be noted that these calculations are presented only to illustrate the design procedure. The designers are encouraged to select appropriate inputs depending on the type of highway, design traffic and other considerations.

1. Reliability: 80% for 5, 10, 20 msa and 90% for 30, 40, 50 msa
2. Bitumen type: VG30 for 5, 10, 20 msa and VG40 for 30, 40, 50 msa
3. Resilient modulus of bituminous layer: 2000 MPa for VG30 mix and 3000 MPa for VG40 mix
4. Elastic modulus of CTB: 5000 MPa
5. Elastic modulus of CTSB: 600 MPa
6. Elastic modulus of RAP/aggregate stabilised with emulsion or foam bitumen: 800 MPa
7. Elastic modulus of granular layer (WMM) over CTSB: 350 MPa (crushed rock)
8. Effective Elastic modulus of subgrade (using Equation 6.2): 77 MPa
9. Poisson's ratio: 0.25 for CTB and CTSB and 0.35 for all other layers
10. The mix volumetric parameters assumed for estimation of the fatigue life are given in the tables
11. The values of RF factor used in Equation 3.5 are taken as 2 for design traffic less than 10 msa and as 1 for design traffic of 10 msa or more.

Some of the thicknesses (especially those of bituminous layers) given in the thickness templates in Section 12 of the guidelines and in the following tables have been selected based on the minimum thickness requirement of bituminous layer for pavements with CTB base. If only one bituminous layer is provided, the minimum thickness of the bituminous layer has been provided as 40 mm.

The *** appearing under the bituminous layer fatigue life column indicates that the computed horizontal strain at the bottom of the bituminous layer is 'compressive' and thus fatigue performance need not be checked.

Table III.1 Calculations for Pavement with Bituminous Surface Course with Granular Base and Sub-base (Effective Subgrade CBR = 10%)

Design Traffic (msa)	Total Bituminous Layer Thickness (mm)	Total Granular Layer Thickness (mm)	Granular layer Modulus (MPa)	Mix Parameters			Computed Strains		Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa
				V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Subgrade Compressive Strain		
5	80	400	228	4.5	10.5	1.12	3.28E-04	4.80E-04	10	46
10	80	450	240	4.5	10.5	1.12	3.15E-04	4.07E-04	11	97
20	110	450	240	4.5	10.5	1.12	2.65E-04	3.54E-04	22	185
30	125	450	240	3.5	11.5	2.35	2.01E-04	3.09E-04	31	116
40	135	450	240	3.5	11.5	2.35	1.87E-04	2.93E-04	41	147
50	145	450	240	3.5	11.5	2.35	1.75E-04	2.78E-04	53	187

Table III.2 Calculations for Pavement with Bituminous Surface Course with CTSB, CTB and Granular Crack Relief Layer (Effective Subgrade CBR = 10%)

Design Traffic msa	Layers Thickness in (mm)				Mix Parameters			Computed Strains			Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa	Fatigue Criteria Max Allowable Traffic in msa (CTB)
	Total Bituminous Layer	AIL	CTB	CTSB	V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Tensile Strain (CTB)	Compressive Strain			
5	40	100	100	200	3.5	11.5	2.35	1.18E-04	8.19E-05	3.33E-04	1096	242	18
10	40	100	100	200	3.5	11.5	2.35	1.18E-04	8.19E-05	3.33E-04	1096	242	18
20	50	100	110	200	3.5	11.5	2.35	1.47E-04	7.59E-05	3.03E-04	471	372	23
30	100	100	100	200	3.5	11.5	2.35	1.32E-04	5.92E-05	2.52E-04	160	293	438
40	100	100	100	200	3.5	11.5	2.35	1.32E-04	5.92E-05	2.52E-04	160	293	438
50	100	100	100	200	3.5	11.5	2.35	1.32E-04	5.92E-05	2.52E-04	160	293	438

Table III.3 Calculations for Bituminous Surface Course with CTSB, CTB and SAMI (Effective Subgrade CBR = 10%)

Design Traffic (msa)	Layer Thickness in (mm)			Mix Parameters			Computed Strains			Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa	Fatigue Criteria Max Allowable Traffic in msa (CTB)
	Total Bituminous Layer	CTB	CTSB	V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Tensile Strain (CTB)	Subgrade Compressive Strain			
5	40	160	200	3.5	11.5	2.35	6.59E-05 (Comp)	8.34E-05	2.96E-04	***	413	14
10	50	160	200	3.5	11.5	2.35	6.21E-05 (Comp)	8.01E-05	2.84E-04	***	497	24
20	80	150	200	3.5	11.5	2.35	5.16E-05 (Comp)	7.44E-05	2.65E-04	***	680	57
30	100	130	200	3.5	11.5	2.35	3.36E-05 (Comp)	7.26E-05	2.51E-04	***	298	38
40	100	135	200	3.5	11.5	2.35	3.41E-05 (Comp)	7.09E-05	2.44E-04	***	336	51
50	100	135	200	3.5	11.5	2.35	3.41E-05 (Comp)	7.09E-05	2.44E-04	***	336	51

Comp: the horizontal strain is compressive

Table III.4 Calculations for Pavement with Bituminous Surface Course with CTSB and Emulsion/ Foam Bitumen Stabilised RAP/Virgin Aggregate (Effective Subgrade CBR 10%)

Design Traffic (msa)	Layer Thickness in (mm)			Mix Parameters			Computed Strains		Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa
	Total Bituminous Layer	RAP	CTSB	V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Subgrade Compressive Strain		
5	40	100	200	4.5	10.5	1.12	3.48E-05	5.25E-04	60039	31
10	40	100	200	4.5	10.5	1.12	3.48E-05	5.25E-04	60039	31
20	40	100	200	4.5	10.5	1.12	3.48E-05	5.25E-04	60039	31
30	100	100	200	3.5	11.5	2.35	1.11E-04	3.74E-04	313	49
40	100	100	200	3.5	11.5	2.35	1.11E-04	3.74E-04	313	49
50	105	100	200	3.5	11.5	2.35	1.10E-04	3.64E-04	325	55

Table III.5 Calculations for Pavement with Bituminous Surface Course with GSB, CTB and Granular Crack Relief Layer (Effective Subgrade CBR = 10%)

Design Traffic (msa)	Layer Thickness in (mm)				Granular layer Modulus (MPa)	Mix Parameters			Computed Strains			Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa	Fatigue Criteria Max Allowable Traffic in msa (CTB)
	Total Bituminous Layer	AIL	CTB	GSB		V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Tensile Strain (CTB)	Subgrade Compressive Strain			
5	80	100	135	200	167	3.5	11.5	2.35	1.57E-04	9.08E-05	2.57E-04	362	783	5
10	80	100	145	200	167	3.5	11.5	2.35	1.57E-04	8.604E-05	2.43E-04	360	1022	10
20	85	100	155	200	167	3.5	11.5	2.35	1.55E-04	7.54E-05	2.25E-04	375	1464	25
30	100	100	150	200	167	3.5	11.5	2.35	1.29E-04	7.28E-05	2.07E-04	176	715	37
40	100	100	155	200	167	3.5	11.5	2.35	1.29E-04	7.11E-05	2.01E-04	178	806	49
50	100	100	160	200	167	3.5	11.5	2.35	1.28E-04	6.95E-05	1.96E-04	179	910	65

Table III.6 Calculations for Pavement with Bituminous Surface Course with CTSB and WMM Layer (Effective Subgrade CBR = 10%)

Design Traffic (msa)	Layer Thickness in (mm)			Mix Parameters			Computed Strains		Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa
	Total Bituminous Layer	WMM	CTSB	V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Subgrade Compressive Strain		
5	40	150	200	4.5	10.5	1.12	1.83E-04	4.99E-04	94	39
10	40	150	200	4.5	10.5	1.12	1.83E-04	4.99E-04	94	39

Design Traffic (msa)	Layer Thickness in (mm)			Mix Parameters			Computed Strains		Fatigue Criteria Max Allowable Traffic in msa	Rutting Criteria Max Allowable Traffic in msa
	Total Bituminous Layer	WMM	CTSB	V_a (%)	V_{be} (%)	C	Tensile Strain (BT)	Subgrade Compressive Strain		
20	80	150	200	4.5	10.5	1.12	2.27E-04	4.18E-04	41	86
30	90	150	200	3.5	11.5	2.35	1.94E-04	3.82E-04	36	44
40	95	150	200	3.5	11.5	2.35	1.89E-04	3.72E-04	40	50
50	110	150	200	3.5	11.5	2.35	1.73E-04	3.45E-04	56	70

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/Manual, etc. from the date specified therein)