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Applicability Of MEPDG For India Based On Texas Pavement And Environmental Conditions

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DEDICATION

I would like dedicate my work to my father, mother and brother who supported me all the time.

APPLICABILITY OF MEPDG FOR INDIA BASED ON TEXAS PAVEMENT AND ENVIRONMENTAL CONDITIONS

by

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ABSTRACT

The Indian pavement design guide uses empirical models which were adopted and modified from US design guides. As with any empirical model, the prediction capabilities are limited to the conditions for which it was developed. In USA, Mechanistic Empirical Pavement Design Guide (MEPDG) has been recently developed to account for increase in truck traffic and minimize premature failure of pavements. Since the new design uses mechanistic approach, it is expected that the design guide can better predict performance of pavements. In addition, mechanistic approach allows use of design guide for different pavement and environmental conditions by calibrating model parameters for different conditions. Thus, the design guide can be used for Indian roads and was focus of this study.

The MEPDG requires inputs in four different modules: Traffic, materials, environment & pavement structure. These parameters may or may not be available for Indian conditions. To identify parameters that influence performance prediction significantly, the design guide was evaluated for Texas pavement and environmental conditions. After successful identification of parameters, the MEPDG analysis was performed and compared with the Indian design guide. The results of the analyses indicated that the expected performance can be achieved at lower Equivalent Single Axle Loads (ESALs) but not at higher ESALs.

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INTRODUCTION

1.1 PROBLEM STATEMENT

Road infrastructure is vital for the economic development of any country. Currently a significant amount of investment is being made in India for connecting major cities (National Highway Development Program (NHDP), as well as villages (through Pradhana Mantri Gram Sadak Yojana (PMGSY) (Ministry of Rural Department, 2001). Many of the road projects are currently undertaken under the Public Private Partnerships (Ministry of Finance, 2010). In most of the projects, the construction company guarantees the performance of the pavement to the government. However, the government will take over these highways at the end of the guarantee period. These mega road infrastructure projects consume a large quantity of natural resources. If these roads are not designed to carry the expected traffic load, the increased demand for maintenance of the road infrastructure will be a challenge for the government in the years to come.

Majority of the pavements constructed in India are flexible pavements with asphalt concrete as a surface layer. The current method of asphalt pavement design in India is empirical in nature. Typically, the empirical pavement design procedures are suitable for the conditions developed and may not be applicable for different conditions (environmental and/or pavement conditions). Thus, the predicted service life of pavements may not be reliable.

On the other hand, the developed countries (US, Canada, Europe, etc) have shifted to more analytical based pavement design and evaluation methods to reliably predict service life of pavements (WSDOT, 1995). One of the important input parameters in the analytical pavement design methods is the pavement distress models. Typically, these models consist of mechanistic approach (analytical) as well as empirical approach (calibration factors) to be suitable for various conditions. These models can better predict the deterioration of the pavement when subjected to local traffic and environmental conditions.

Although the mechanistic-empirical pavement design procedure have been in use in various forms for a long time, the recently developed Mechanistic-Empirical Pavement Design Software (MEPDS) and Guide (MEPDG) represent the results of the first comprehensive national-level effort for implementation of this procedure in the US (ARA, 2004). The MEPDG is comprehensive software that can be used for flexible as well as rigid pavement design. It uses most of the variables currently available for design of roadway structures, including the material properties of HMA, base, subbase, and subgrade layers. The software guide incorporates discussions on the causes and practical methods of prevention of commonly experienced distresses. Steps for each process, applicable equations, some default values, and background information regarding the results of each test are included in the guide.

Since Indian pavement conditions (soil type, environmental, traffic, etc.) vary significantly, it is essential to use the MEPDG rather than empirical relationships and was focus of this study.

1.2 RESEARCH OBJECTIVES AND SCOPE

The main purpose of this study was to identify suitability of the new design guide for Indian conditions. To achieve this goal, the following objectives were selected:

- develop an understanding of the MEPDG by predicting performance using Texas environmental and traffic conditions; and
- Evaluate feasibility of using MEPDG for Indian environmental and traffic conditions.

The recently developed software requires large number of inputs for analysis which may or may not be readily available. Therefore, it is necessary to identify essential inputs needed for predicting performance.

This study was performed by using pavement layer thicknesses commonly encountered within the state of Texas. The preliminary software runs were performed for El Paso (Western India) and Houston (Eastern India) weather conditions to identify suitable pavement structures. The suitable structures were then used for Miami (Southern India) weather conditions. Since use of modifiers within the asphalt concrete layers is quite common in India, the asphalt concrete mixes commonly used within the state of Texas were used. In the end, the asphalt concrete properties and pavement structures currently used in the national highways of India were evaluated.

1.3 ORGANIZATION

The problem statement, objective and approaches of this research are presented in this chapter. Chapter Two contains review of pavement design guides being used in India and

USA. In Chapter Three, the MEPDG modules and its inputs are described and a detail description of the approach for the evaluation of MEPDG is presented. In Chapter Four, the results of the evaluation and analysis of the data are discussed. Chapter Five presents results of case study for Indian roadway structure and material properties. The summary, conclusion and recommendations for the future research are included in Chapter Six.

Chapter 2

BACKGROUND

2.1 INTRODUCTION

The US government has invested significantly on its highway system since the fifties, and now has a very well developed system of interstate, state and local road network. Furthermore, developments in materials, design procedure and construction techniques have been absorbed successfully in the road industry leading to continuous improvements in durability, safety and reliability of roads. On the other hand, Indian government has recently started investing in national highway system; therefore, enough performance data is not available to develop mechanistic design procedure for Indian highways. In this chapter, a discussion on current design guide process followed in India is presented. In addition, a discussion on two major design guides developed in US is presented. In the end, the approach followed to achieve objectives of this study is discussed.

2.2. INDIAN GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS

In India, national, state, as well as rural roads have been designed and constructed as a flexible pavement (i.e., asphalt concrete as a surface layer). In 1970, Indian Road Congress (IRC) developed comprehensive guidelines for the design of flexible pavements based on California Bearing Ratio (CBR) method. These guidelines were revised in 1984 to accommodate increase in traffic and were based on equivalent single axle load (ESAL's) of up to 30 millions. These revised guidelines used semi-empirical approach and were based on past experience and

judgment of highway agencies. The rapid growth in India required national highway system to be designed for higher ESALs which required further modifications to the design guidelines. In 2001, IRC proposed new guidelines for design of flexible pavements (IRC: 37-2001) which is summarized in the following section.

2.2.1 GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS (IRC: 37-2001)

The 2001 flexible pavement design guide was developed to carry traffic of up to 150 million ESALs during the design life of the pavement which is 5 times higher than 1984 design guide. The design guide was developed for the design of new Expressways, National Highways, State Highways, Major District Roads and other Categories of roads predominantly carrying motorized vehicles (Indian Road Congress, 2001).

The flexible pavement is modeled as a three layer structure and a 50 mm (2 in.) or less wearing course. A typical pavement structure is shown in Figure 2.1. The surface or wearing course is made of bituminous concrete (BC), dense graded bituminous macadam (DBM) or binder course, granular base layer (GB), and granular subbase layer (GSB). The GSB is placed on top of the subgrade (existing soil surface). The design is based on vertical compressive strain at the top of the subgrade (C), horizontal tensile strain at the bottom of bituminous layer (A and B) and permanent deformation within bituminous or asphalt concrete layer. The permanent deformation in BC and DBM is controlled by selection of stronger aggregate skeleton and modified binders. While, the layer thicknesses for granular base and binder course are selected using the analytical (theory of elasticity) design approach. The selection of binder and aggregate skeleton is considered under different publication (IRC: SP53-2002). The thickness of different layers is considered using Fatigue and Rutting Criteria outlined in the following sections.

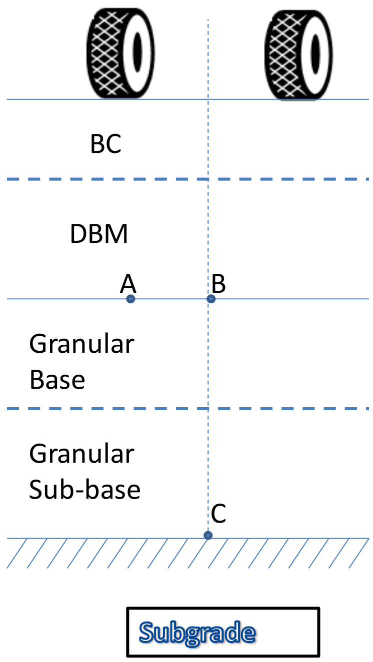


Figure 2.1 Typical Pavement Structure Used in Design Guide (IRC: 37-2001)

2.2.1.1 Fatigue Criteria

The analytical approach (elastic theory) suggests that the flexural fatigue failure will occur if the tensile strain at the bottom of the bituminous layer is beyond a certain limit. Based on research performed, a relationship between tensile strain and cumulative loading is provided:

$$N_F = 2.21 * 10^{-4}[1/\varepsilon_t]^{3.89}[1/E]^{0.854} \tag{2.1}$$

Where:

N_F = Number of Cumulative ESALs to produce 20 percent cracked surface area,

ε_t = Tensile Strain at the bottom of BC layer (micro strain)

E = Elastic Modulus of Bituminous Surfacing (MPa)

This fatigue equation was calibrated at 35° C for BC surfacing having 80/100 bitumen and the equation was generalized by introducing the term containing elastic modulus (E) of the bituminous layer such that the pavement can be designed for different temperature as shown in Table 2.1. The guideline also suggests using Poisson's ratio of bituminous layer to be 0.50 for pavement temperatures above 35 °C and 0.35 for pavement temperatures from 20 to 30 °C.

Table 2. 1 Elastic Modulus Values of Bituminous Materials for Different Temperature

Mix Type	Temperature, ° C				
	20	**25**	**30**	**35**	**40**
BC and DBM 80/100 Bitumen	2,300	1,966	1,455	975	797
BC and DBM 60/70 Bitumen	3,600	2,126	2,579	1,695	1,270
BC and DBM 30/40 Bitumen	6,000	4,928	3,809	2,944	2,276
BM 80/100 Bitumen	-	-	-	500	-
BM 60/70 Bitumen	-	-	-	700	-

2.2.1.2 Rutting Criteria

The analytical approach suggested that the vertical strain at the top of the subgrade will govern rutting failure of the pavement. Based on modeling and performance data, the following rutting equation was obtained by setting the allowable rut depth as 20 mm:

$$N_R = 4.1656 * 10^{-8}[1/\varepsilon_Z]^{4.3337} \tag{2.2}$$

Where

N_R = Number of cumulative ESALs to produce rutting of 20 mm

ε_Z = Vertical subgrade strain (micro strain)

Since the initial design guide was based on CBR, the following conversion factors have been proposed for obtaining modulus of elasticity.

- For subgrade layer, E(MPa) = 10*CBR for CBR $\leq$ 5 and 17.6*(CBR)$^{0.64}$ for CBR > 5
- For GB and GSB layers, $E_2 = E_3*0.2*h^{0.45}$ where E_2 is the composite modulus of elasticity of GSB and GB in MPa, E_3 is modulus of elasticity of subgrade in MPa, and h is the thickness of granular layers in mm.

2.2.1.3 Traffic

The design guide considers traffic in terms of the cumulative number of ESALs to be carried by the pavement during the design life. One ESAL is equivalent to 80 kN or 8,160 kg or 18 kips. The cumulative ESALs are estimated based on the following information:

- Initial traffic after construction in terms of commercial vehicle per day (CVPD). The CVPD can either be based on actual traffic count or on the basis of potential land use.
- The traffic growth rate during design life in percentage needs to be either estimated based on past trends or by assuming average annual growth rate of 7.5 percent.
- Design life of pavement is used as an input to identify CVPD carried by the pavement during life of the pavement before rehabilitation. The design guide proposes to design for 20 years of service for Expressways and Urban Roads and 15 years for the remainder roads.
- The design guide also proposes to use vehicle damage factor (VDF) to take into account various axle configuration, axle loading, terrain, type of road and from region to region. The VDF can either be estimated using actual traffic survey or by using VDF factors ranging from 0.5 to 4.5 depending on CVPD and terrain type.

- The design guide proposes to realistically estimate distribution of ESALs based on number of lanes planned for the roads. For example, single-lane roads should be designed based on actual ESALs while four lane roads should be designed based on 40% of actual ESALs.
- Based on these factors, the following equation is proposed for design of pavement:

$$N = \frac{365 * [(1+r)^n - 1] * A * D * F}{r} \tag{2.3}$$

Where,

N = the cumulative number of ESALs to be catered for in the design in terms of million ESALs.

A = Initial traffic in the year of completion of construction in terms of the number of CVPD

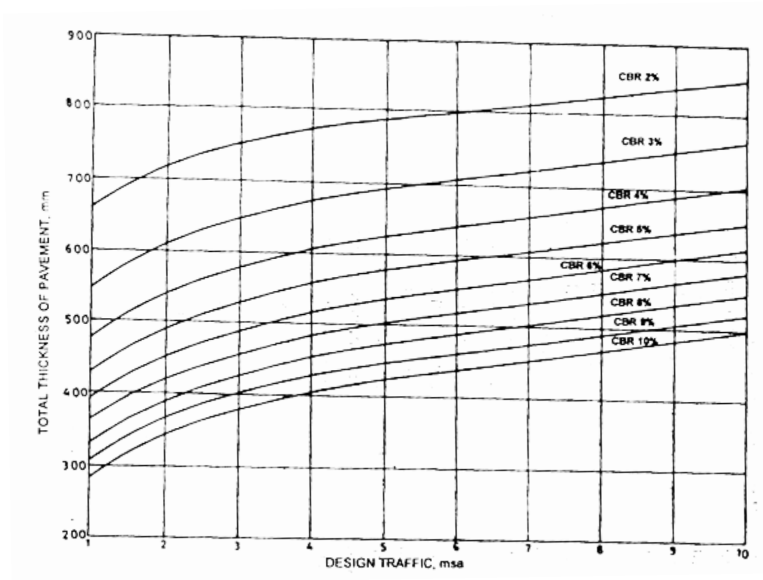
D = lane distribution factor,

F = vehicle damage factor

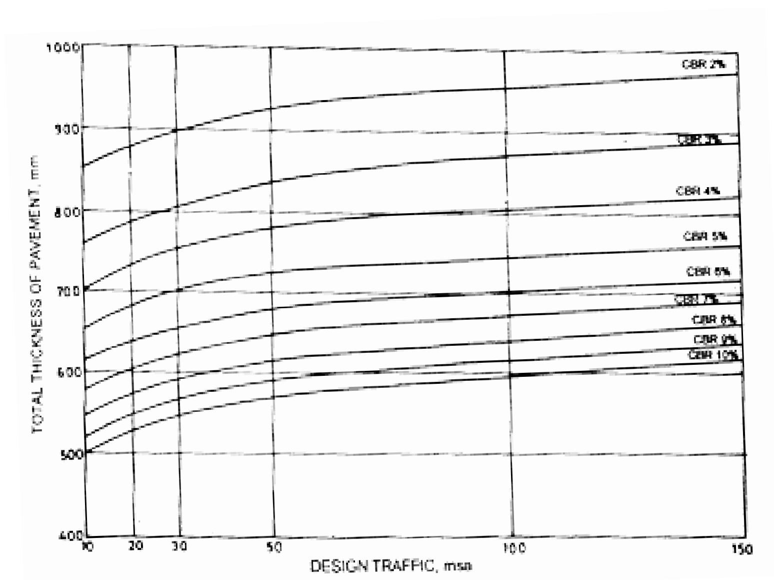
n = design life in years

r = annual growth rate of commercial vehicles.

Based on cumulative millions ESALs and subgrade CBR values, design charts have been developed to identify total pavement thickness as shown in Figure 2.2. The design guide has also proposed individual layer thicknesses. For instance, Table 2.2 shows individual layer thicknesses for traffic up to 10 million ESALs.



a) 1 to 10 million ESALS



b) 10 to 150 million ESALS

Figure 2.2 Pavement Thickness Design Chart From IRC 37:2001

Table 2. 2 Recommended Design for Traffic Range of 1-10 million ESALs

CBR 2%					
Cumulative Traffic (million ESALs)	Total Pavement Thickness (mm)	Bituminous Surfacing		Granular Base (mm)	Granular Subbase (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	660	20		225	435
2	715	20	50	225	440
3	750	20	60	250	440
5	795	25	70	250	450
10	850	40	100	250	460

2.3. USA DESGIN GUIDES

Pavement design procedures have evolved through local and/or national practices, developed by agencies like AASHTO, the U.S. Corps of Engineers, the various State Highway Agencies (SHA), oil companies like shell oil and industry supported associations such as Asphalt Institute (AI). The variability in design practices can be attributed to the availability of materials and traffic in each state and depending on the agency's perspective. The typical life of asphalt concrete pavements is 12.5 years (it ranges from 7 years to 21 years nationally) according to the results of 1987 questionnaire on the typical "lives" for asphalt concrete (WSDOT, 1995). A NCHRP Synthesis report (WSDOT, 1995) contains a 1991 survey of 48 SHAs and six Canadian provinces which was used to overview state practice on

- Procedures to determine thickness: The procedure followed by most of the states is AASHTO Guide (the 1972 and 1986 versions are used about equally) which is empirical. Mechanistic-empirical design is used by four agencies and five agencies use the AI method (Manual series No. 1) which is also mechanistically based.

- Layer compositions and configurations: 20 years of design period is used by most of the agencies and the design period of 20 – 40 years is used in WSDOT depending on the highway classification and the ESALs level.
- Drainage treatments: The survey states that ten states and one Canadian province use the either unbound or treated open graded permeable materials into flexible pavement structures on regular basis and 12 more agencies doing the same on experimental basis.
- Traffic characterization: 53 agencies use 80 KN equivalent single axle loads (ESALs) to characterize the efforts of mixed traffic.

  Material Characterization: The most common tests being conducted by various SHAs on flexible pavement materials are shown in Table 2.3. The data indicates that the Marhsall method is commonly used for asphalt concrete material and CBR is commonly used for characterization of granular base (GB), granular subbase (GSB) and subgrade layers.
- Miscellaneous design features.

Table 2. 3 Material Characterization Tests for Materials used in Various Pavement Layers (WSDOT, 1995)

Material	Number of Agencies	Test/Process
Asphalt Concrete	40	Marshall Stability
	7	Hveem Stability
	4	Resilient Modulus
Unstabilized Aggregate Base/Sub base	14	CBR
	10	R-value
	3	Elastic Modulus
Sub grade	25	CBR
	13	R-value
	10	Elastic Modulus
Asphalt Treated Permeable Material	1	CBR
	3	R-value
	2	Elastic Modulus or Resilient Modulus

The survey stated that the SHAs plan to make change in the flexible pavement design procedure, practiced by the agencies and 22 among them are opting for the mechanistic empirical pavement design and 12 other states are planning to follow resilient modulus testing and 10 states plan to adopt the 1986 AASHTO Guide.

The most commonly identified premature failures or failures at the end of the design life by various SHAs are shown in Table 2.4. The results suggest fatigue failure is most prevalent followed by rutting in the pavements.

Table 2. 4 Failure Modes Identified by Various SHAs (WSDOT, 1995)

Mode	Number of SHAs*
Fatigue Cracking	20
Rutting	14
Cracking (non-specific)	12
Thermal Cracking	6
Stripping	5
Weathering	4
Raveling	3
Reflective Cracking	2
Other Modes	10

* Multiple modes noted by some SHAs.

The review of literature identified that AASHTO flexible design guide or a variation thereof has been most commonly used. In the following sections, a discussion on these design guides is presented.

2.3.1. AASHTO FLEXIBLE PAVEMENT DESIGN PRE-1986

Initial, AASHTO flexible pavement design guide was developed from the AASHTO road test conducted in the early 1960s at Ottawa, Illinois. The AASHTO test facilities consisted of six

2-lane test loops, where the loop 1 was used for figuring out the environmental effects and the remaining five tracks were used to figure out the performance measurements such as roughness, visual distress, deflections, strains and Pavement Serviceability Index (PSI) with different type of axle weights and distributions.

The asphalt concrete (AC) mixes used in the road test consisted of crushed limestone coarse aggregate, natural siliceous coarse sand, mineral filler which was limestone dust and penetration grade asphalt cement (85-100 pen). The asphalt concrete mixes were designed by Marshall Method (50 blows per face). The typical field asphalt content were around 5.4 and 4.4 percent by weight of total mix for the surface and binder courses, respectively and the field air voids averaged 7.7 percent for the six test loops. The base course material was crushed dolomitic limestone which had an averaged CBR value to be 107.7 percent based on laboratory tests and 10 percent passed the No. 200 sieve, which is frost susceptible. Typical in place mean dry densities were about 140-142 lb/ft^{3,} with mean moisture contents ranging from 5.6 to 6.1 percent.

The sub-base material was sand-gravel mixture with a CBR value ranging in between 28 and 51 percent and the mean percent passing the No. 200 sieve was 6.5 percent and the typical in place mean dry densities ranged from 139-141 lb/ft^3, with moisture contents ranging from 6.1-6.8 percent and all the pavements were built on a uniform embankment with the top 3 ft. constructed on A-6 soil. There were 322 flexible pavement test sections in the main experiment including replications and each flexible pavement section in the main experiment was 100 ft. long and with all sections being separated by a transition pavement of at least 15 ft. Table 2.5 shows the different structural thickness of each loop.

Table 2. 5 Structural Thicknesses followed in Testing for Each Loop (WSDOT, 1995)

Loop No.	AC Thickness (in.)	Base Thickness (in)	Subbase Thickness (in)
1	1.0 3.0 5.0	0.0 6.0	0.0 8.0 16.0
2	1.0 2.0 3.0	0.0 3.0 6.0	0.0 4.0
3	2.0 3.0 4.0	0.0 3.0 6.0	0 4 8
4	3.0 4.0 5.0	0 3 6	4 8 12
5	3.0 4.0 5.0	3.0 6.0 9.0	4.0 8.0 12.0
6.	4.0 5.0 6.0	3.0 6.0 9.0	8.0 12.0 16.0

The AASHTO Road test experiment was designed as full factorial experiment with design factors being surface thickness, base thickness and subbase thickness. In many sections, the PSI reached a value of 1.5 before completion of test. Therefore, these test sections were overlaid before remaining ESALs were applied. The following observations were made from the study:

- For low ESALs pavements, the depth of the pavement should be about 65 percent of the expected depth of the frost.

- For high ESALs pavements, the depth of the pavement should be about 90-100 percent of the expected depth of the frost.

Equations were developed to relate loss in serviceability, traffic, and pavement thickness from the AASHTO Road Test and the developed equations:

- may be extended other for subgrade soils by an abstract soil support scale,
- can be applied to mixed traffic by using equivalent 18-Kip single axle loads,
- may be applied to other regions through the use of regional factor,
- may be applied to the other surfaces, bases and sub bases by assigning appropriate layer coefficients, and
- accelerated testing (2-year period) can be extended to 20 years design life.

The evaluation results of the test were used to develop nomographs for identifying Structural Number for various terminal serviceability indexes. Briefly, the procedure is as follows:

- First, the design life of pavement is assumed and then it is used to determine cumulative axle load (ESAL) using Figure 2.4. The ESALs are adjusted to take into account directional and lane distribution similar to IRC: 37-2001. For ESAL estimation SN is assumed to be 3 or 4.
- The Structural Number (SN) is considered to be a function of terminal serviceability index (P_t), total number of ESALs (W_t), subgrade support (S), and regional factor (R). The nomograph shown in Figure 2.5 is used to identify SN.

Axle Load		Structural Number					
kips	kN	1	2	3	4	5	6
Single Axles, $p_t = 2.0$							
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
10	44.5	0.08	0.08	0.09	0.08	0.08	0.08
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.59	1.56	1.55	1.57	1.60
30	133.4	10.38	10.03	9.24	8.65	8.73	9.17
40	177.9	39.57	38.02	34.34	30.92	30.04	31.25
Single Axles, $p_t = 2.5$							
2	8.9	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
10	44.5	0.08	0.10	0.12	0.10	0.09	0.08
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.57	1.49	1.47	1.51	1.55
30	133.4	10.31	9.55	7.94	6.83	6.97	7.79
40	177.9	39.26	35.89	28.51	22.50	21.08	23.04
Tandem Axles, $p_t = 2.0$							
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
20	89.0	0.10	0.12	0.12	0.12	0.11	0.10
30	133.4	0.61	0.62	0.65	0.64	0.63	0.63
34	151.2	1.06	1.07	1.08	1.08	1.08	1.07
40	177.9	2.22	2.19	2.15	2.13	2.16	2.18
48	213.5	5.10	4.98	4.72	4.58	4.68	4.83
Tandem Axles, $p_t = 2.5$							
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
20	89.0	0.11	0.14	0.16	0.14	0.12	0.11
30	133.4	0.61	0.65	0.70	0.70	0.66	0.63
34	151.2	1.06	1.08	1.11	1.11	1.09	1.08
40	177.9	2.21	2.16	2.06	2.03	2.08	2.14
48	213.5	5.08	4.80	4.25	3.98	4.17	4.49

Figure 2.3 Sample of AASHTO Traffic Equivalent Factors for Flexible Pavements (WSDOT, 1995)

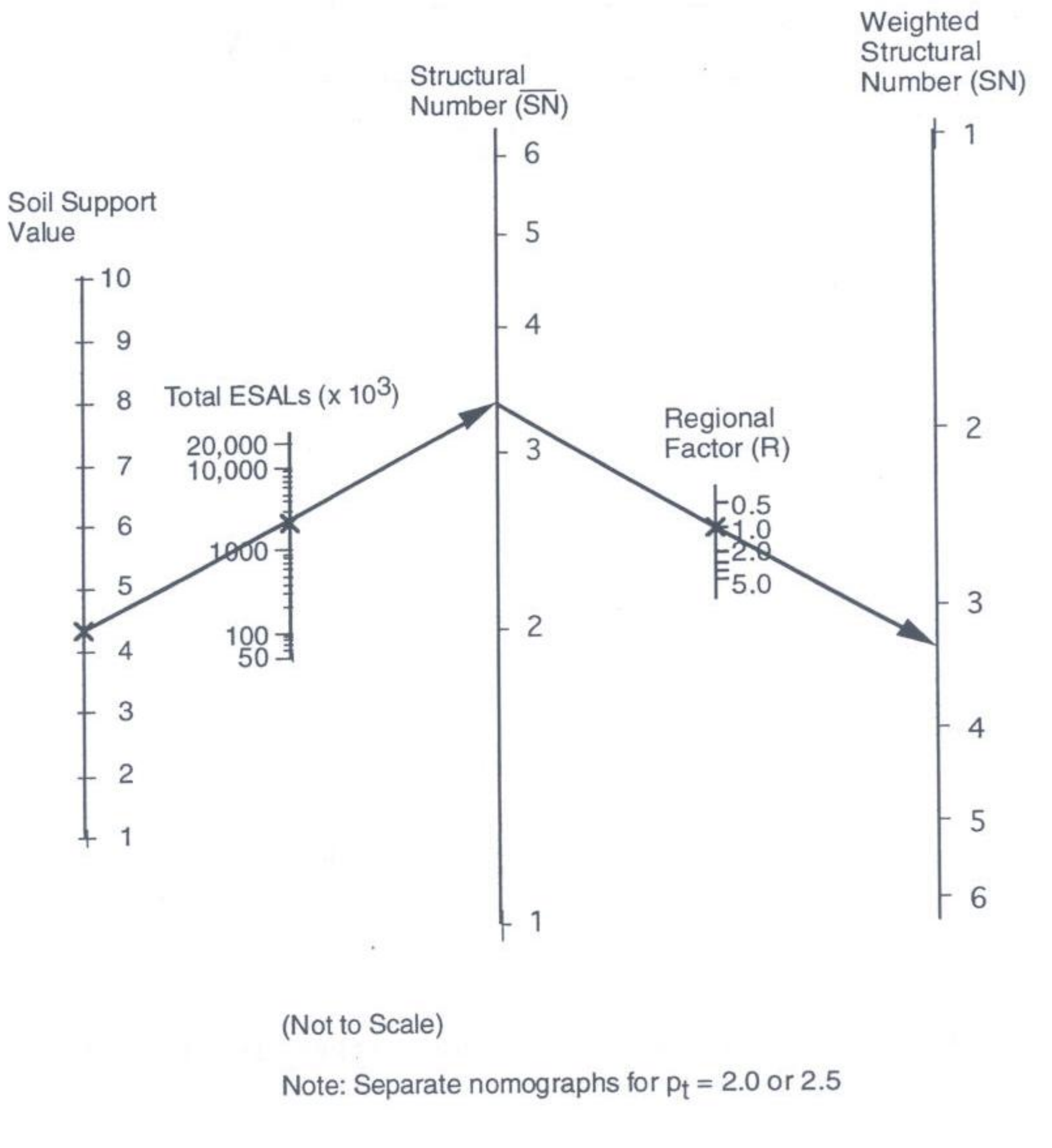
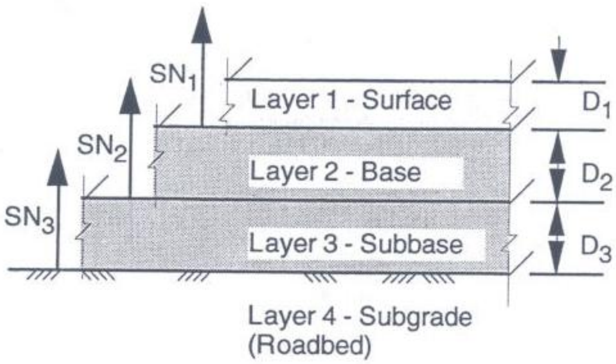


Figure 2.4 AASHTO Flexible Pavement Design Nomograph (WSDOT, 1995)

- The SN is then used to identify individual layer thickness based on the system presented in Figure 2.6. The SN= $A_1D_1+A_2D_2+A_3D_3$ where $D_{1,2,3}$ represents thickness of pavement layers in inches and $A_{1,2,3}$ represents structural layer coefficients.



$$D_1^* \geq \frac{SN_1}{a_1}$$

$$SN_1^* = a_1 D_1^* \geq SN_1$$

$$D_2^* \geq \frac{SN_2 - SN_1^*}{a_2}$$

$$SN_2^* = a_2 D_2^*$$

$$SN_1^* + SN_2^* \geq SN_2$$

$$D_3^* \geq \frac{SN_3 - (SN_1^* + SN_2^*)}{a_3}$$

* indicates value actually used which must be equal to or greater than the required value.

Figure 2.5 AASHTO Conceptual Flexible Pavement Layer Determination (WSDOT, 1995)

2.3.2. AASHTO FLEXIBLE PAVEMENT DESIGN 1986 & 1993

This section summarizes the information about the AASHTO Guide (1993) which is a substantially revised version of 1986 AASHTO Design Guide. The objectives and goals guidelines of the new AASHTO Design Guide (1986) was to incorporate new design tools and respond to industry concerns and characterization of strength of layers from tests, variability and reliability (emphasis on reliability), life cycle costs, drainage, reliability, MR, Sub-base erosion, Life cycle costs, Rehabilitation, Pavement management, Load equivalencies, Traffic data, Low volume roads and Mechanistic/empirical design. The 1986 and 1993 AASHTO Design Guides are divided into four parts: Pavement management, Design procedures for new construction, Rehabilitation of existing pavements and Mechanistic empirical pavement design. The performance concept of the AASHTO design guide is shown in Figure 2.7.

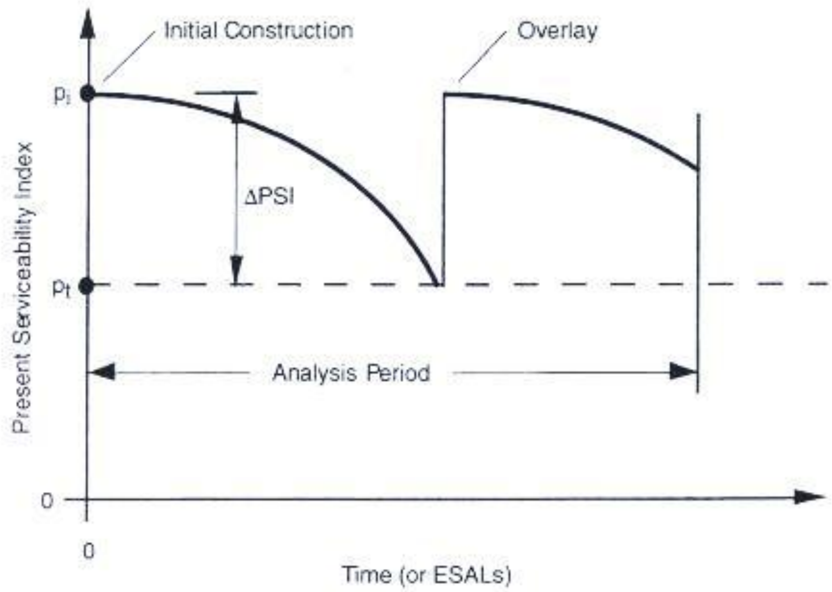


Figure 2.6 Illustration of Performance Concept (WSDOT, 1995)

The changes made to the performance design equations in the 1993 AAASHTO Design Guide are shown in the Figure 2.8. The traffic input for design of pavement is in the form of the number of 18,000 pounds Equivalent Single Axle Load (ESALs) over a specific period of time and state DOTs develop truck factors either for each major truck type or for an average truck to convert mixed truck traffic to ESALs. The Figure 2.9 gives the ESALs per truck for different state DOTs. There is a possibility of increase in the truck mix or axle load changes with time for estimating total ESALs. The Figures 2.10 and 2.11 explain the growth of truck volumes, vehicle miles travelled in 10 year interval in the state of Washington and change in AADT and ESALs/Day on a typical interstate highway. A sample of some of the currently used axle load equivalency factors for each type of axles and for terminal serviceability of 2.0, 2.5 and 3.0 are

shown in the Figure 2.12. The lane distribution factors, shown in Table 2.6, are used to calculate the ESALs per lane.

Original AASHTO Performance (Design) Equation

$$\log W_{18} = (9.36)(\log(SN+1)) - 0.20 + \frac{\log\left[\frac{4.2-p_t}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + \log\frac{1}{R} + 0.372(S-3.0)$$

Revised AASHTO Performance (Design) Equation

$$\log W_{18} = (z_R)(S_o) + (9.36)(\log(SN+1)) - 0.20 + \frac{\log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + (2.32)(\log M_R) - 8.07$$

Figure 2.7 AASHTO Flexible Pavement Performance Equations (WSDOT, 1995)

Ordered by Increasing ESALs		Ordered by State	
State	18,000 lb ESALs/Truck*	State	18,000 lb ESALs/Truck*
Connecticut	0.345	Alabama	0.928
New Hampshire	0.367	Arizona	1.154
Pennsylvania	0.517	Arkansas	0.712
Vermont	0.559	California	1.156
Florida	0.666	Colorado	0.893
Arkansas	0.712	Connecticut	0.345
Maryland	0.719	Florida	0.666
Virginia	0.725	Georgia	0.925
Michigan	0.738	Idaho	1.027
Indiana	0.756	Illinois	0.830
North Carolina	0.779	Indiana	0.756
Kansas	0.792	Iowa	1.023
Wisconsin	0.794	Kansas	0.792
Louisiana	0.804	Kentucky	0.885
Oklahoma	0.820	Louisiana	0.804
Texas	0.826	Maine	1.167
Illinois	0.830	Maryland	0.719
Minnesota	0.841	Michigan	0.738
Mississippi	0.841	Minnesota	0.841
New Mexico	0.882	Mississippi	0.841
Kentucky	0.885	Missouri	0.902
Colorado	0.893	Montana	1.219
Missouri	0.902	Nebraska	1.215
New York	0.912	Nevada	1.220
Ohio	0.924	New Hampshire	0.367
Georgia	0.925	New Jersey	1.175
Alabama	0.928	New Mexico	0.882
Tennessee	0.930	New York	0.912
West Virginia	0.941	North Carolina	0.779
Iowa	1.023	North Dakota	1.082
Idaho	1.027	Ohio	0.924
Washington	1.028	Oklahoma	0.820
Utah	1.066	Oregon	1.136
North Dakota	1.082	Pennsylvania	0.517
South Dakota	1.088	South Dakota	1.088
Oregon	1.136	Tennessee	0.930
Arizona	1.154	Texas	0.826
California	1.156	Utah	1.066
Maine	1.167	Vermont	0.559
New Jersey	1.175	Virginia	0.725
Nebraska	1.215	Washington	1.028
Montana	1.219	West Virginia	0.941
Nevada	1.220	Wisconsin	0.794
Wyoming	1.511	Wyoming	1.511

*For two axle-six tire single units and above

Figure 2.8 ESALs per Truck for State DOTs (WSDOT, 1995)

Vehicle Classification	Survey Year 1970	1980	1990
Passenger cars and buses, and light single unit trucks	86%	82%	78%
Heavy single unit trucks	3%	3%	4%
3 and 4 axle combinations	3%	1%	2%
5 axle or more combinations	8%	14%	16%
	100%	100%	100%

Figure 2.9 Vehicle Classification Survey (WSDOT, 1995)

Year	Vehicle Miles Traveled (billions)[1]
1965	15
1975	25
1985	35
1995 (projected)	52

Figure 2.10 Vehicle Miles Travelled (WSDOT, 1995)

Single Axles

Axle Load		Structural Number and p_t								
		SN = 1			SN = 3			SN = 6		
kips	kN	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$
2	8.9	0.0002	0.0004	0.0008	0.0002	0.0003	0.0006	0.0002	0.0002	0.0002
10	44.5	0.075	0.078	0.082	0.090	0.118	0.168	0.076	0.080	0.086
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.61	1.61	1.60	1.56	1.49	1.41	1.59	1.55	1.51
30	133.4	10.4	10.3	10.2	9.2	7.9	6.5	9.2	7.8	6.3
40	177.9	39.6	39.3	38.8	34.3	28.5	22.2	31.2	23.0	15.3

Tandem Axles

Axle Load		Structural Number and p_t								
		SN = 1			SN = 3			SN = 6		
kips	kN	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$
10	44.5	0.007	0.008	0.011	0.008	0.011	0.020	0.006	0.006	0.007
20	89.0	0.103	0.107	0.113	0.124	0.162	0.232	0.105	0.110	0.119
30	133.4	0.607	0.611	0.616	0.646	0.703	0.788	0.617	0.633	0.656
34	151.2	1.06	1.06	1.07	1.08	1.11	1.15	1.07	1.08	1.09
40	177.9	2.22	2.21	2.21	2.15	2.06	1.94	2.18	2.14	2.08
50	222.5	6.15	6.12	6.08	5.64	5.03	4.31	5.77	5.28	4.70
60	267.0	14.3	14.2	14.1	12.7	10.9	8.9	12.6	10.7	8.6
70	311.5	29.2	29.0	28.7	25.6	21.5	17.0	24.3	19.2	13.9

Triple Axles

Axle Load		Structural Number and p_t								
		SN = 1			SN = 3			SN = 6		
kips	kN	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$	$p_t = 2.0$	$p_t = 2.5$	$p_t = 3.0$
20	89.0	0.024	0.027	0.031	0.029	0.042	0.069	0.023	0.024	0.026
30	133.4	0.125	0.129	0.136	0.149	0.195	0.279	0.126	0.133	0.143
40	177.9	0.434	0.439	0.447	0.481	0.554	0.671	0.443	0.459	0.483
50	222.5	1.17	1.17	1.18	1.20	1.24	1.30	1.18	1.20	1.22
60	267.0	2.67	2.67	2.66	2.59	2.48	2.34	2.63	2.58	2.50
70	311.5	5.40	5.38	5.34	5.03	4.57	4.02	5.15	4.84	4.44
80	356.0	9.98	9.92	9.84	9.05	7.95	6.67	9.18	8.21	7.06
90	400.5	17.2	17.1	16.9	15.3	13.2	10.7	15.2	12.9	10.4

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Figure 2.11 Travel Equivalency Factors (WSDOT, 1995)

Table 2. 6 Lane Distribution Factors

Number of lanes in one direction	Percent ESALs in Design Lane
1	100
2	80-100
3	60-80
4 or more	50-75

Resilient modulus (M_R) is used to characterize the roadbed soil and other layer in the pavement. The reasons of adopting resilient modulus for characterization of the pavement includes: fundamental engineering property, standard test procedure (AASHTO T274), and resilient modulus test is not too complex. Special construction considerations for the roadbed soils are considered such as special treatment to soils like expansive, frost susceptible and organic soils in AASHTO Design Guide (1986 &1993). Back calculation method is used for calculating the resilient modulus by various deflection devices. This is a mechanistic evaluation and it is being used extensively in recent times. The change in the effects the moduli of the pavement and these changes are illustrated in the below figure and the effective resilient modulus is given by the equation

$$(M_R)_{eff} = (3005)(\bar{u}_f)^{-.431} \tag{2.4}$$

Where:

$(M_R)_{eff}$ = effective roadbed resilient modulus weighted by seasonal variation, and

$\bar{u}_f$ = average relative seasonal damage

Reliability of a pavement design is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period (AASHTO). The reasons for using reliability is to provide the design engineers the capability for incorporating variability in the design process, some degree of certainty into the design process, and providing a better basis for comparing flexible and rigid pavement designs. The flexible pavement design includes both asphalt concrete and surface treatment surfaces and design nomograph based on the original but slightly modifies AASHTO performance equation.

The 1993 AASHTO flexible pavement design procedure is explained in the step by step procedure as follows

To determine Structural Number (SN)

- Estimate the future traffic for the performance periods (ESALs)
- Select reliability according to the below figure
- Select or evaluate the overall standard deviation (S_o). It could be ranging from 0.4 to 0.5.
- Determine the effective roadbed resilient modulus
- Select the design serviceability loss, $\Delta PSI = p_0 - p_t$
    - Select layer coefficients.
        - asphalt layer coefficients, a_1 from the Figure 2.13
        - granular base layer coefficient, a_2 from the Figure 2.14 or from the equation, $\mathbf{a_2 = 0.249(\log E_{BS})-.0977}$
        - granular subbase layer, a_3 from the Figure 2.15 and from the equation, $\mathbf{a_3 = 0.227(\log E_{SB})-0.839}$

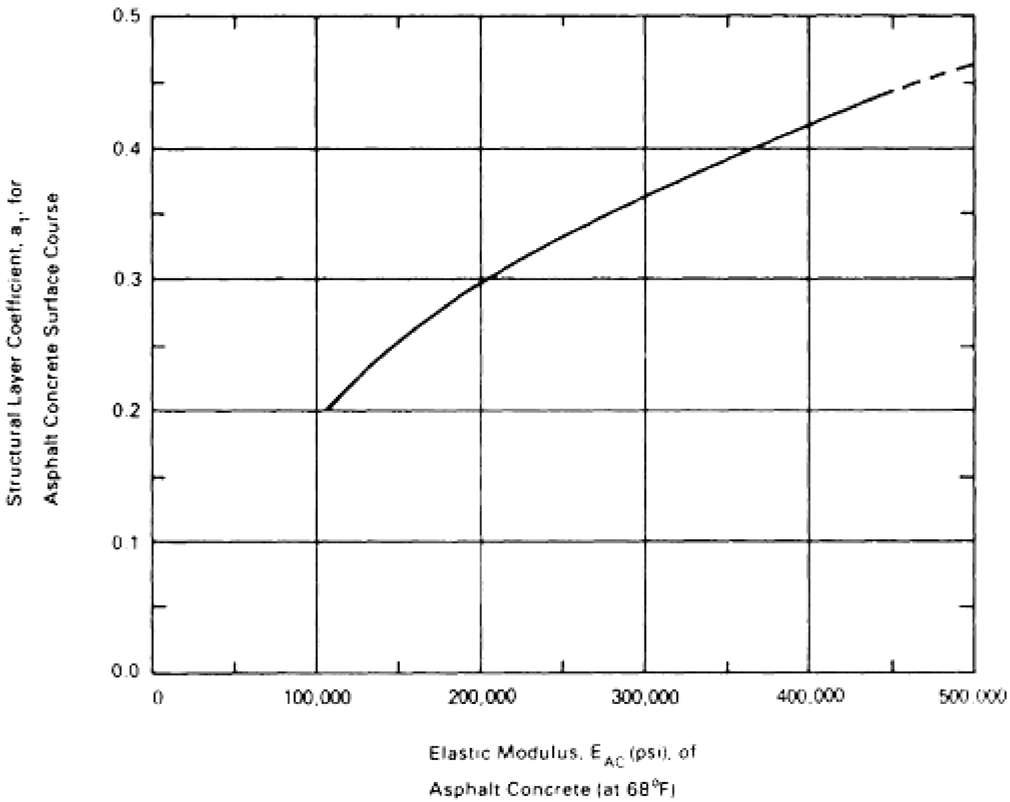
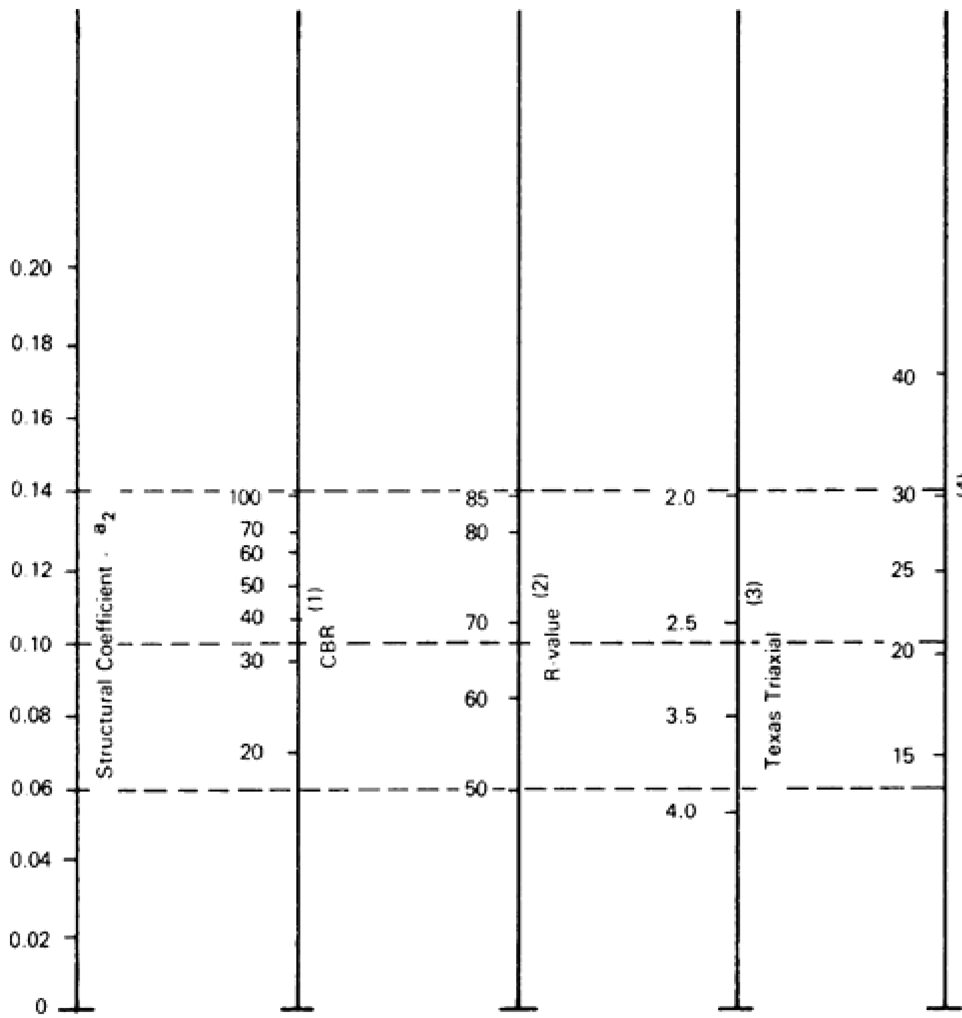
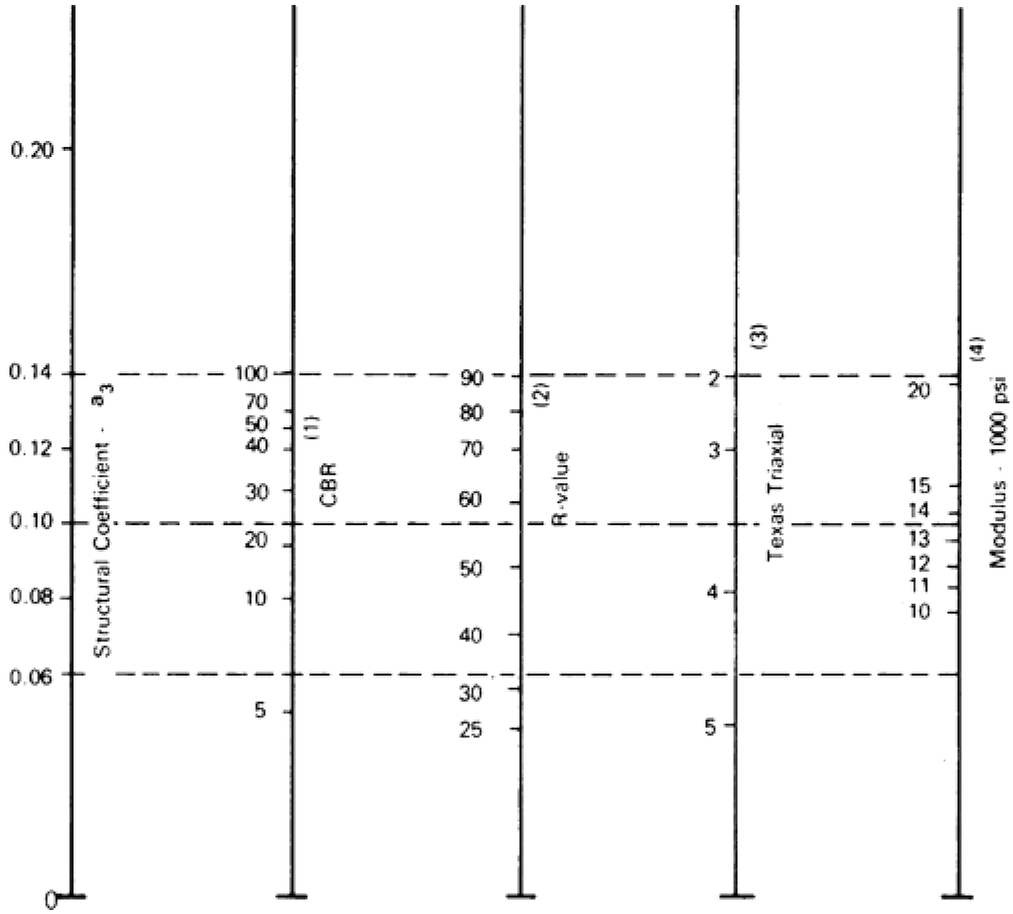


Figure 2.12 Asphalt Layer Coefficient, a_1 (WSDOT, 1995)



(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on NCHRP project *(3)*.

Figure 2.13 Base Layer Coefficient, a_2 (WSDOT, 1995)



(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

(4) Scale derived on NCHRP project *(3)*.

Figure 2.14 Granular Subbase Layer Coefficient, a_3 (WSDOT, 1995)

The drainage coefficient (m) from Table 2.7 for the unstabilized layer and the subbase layers are selected on the basis of "quality of drainage" to adjust the seasonal variation in those layers adjust the thickness of those layers.

Table 2. 7 Drainage Coefficient (WSDOT, 1995)

Base course moduli (psi)	Quality of Drainage	Recommended m- value
10,000	Very poor	0.4
20,000	Poor	0.7
30,000	Fair	1
40,000	Good	1.2
50,000	Excellent	1.4

The m-values are further adjusted depending on the saturation of the material used and modulus of the material and these values are being used for calculating layer thickness.

$$\mathbf{SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3} \qquad (2.5)$$

where a_1, a_2, a_3 = layer coefficients of surface base, subbase and m_2, m_3 are the drainage coefficients.

2.3.3 MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE

There are several limitations of the 1993 AASHTO design guides like under estimating the influence of environment, local materials, etc. In addition, there is tremendous growth in the traffic, changes in axle configurations, use of new and exotic material which reduced the reliability of AASHTO design procedure (ARA, 2004). To overcome limitations and to take into account new technology, AASHTO Joint Task Force on Pavements (JTFP) and National Cooperative Highway Research Program (NCHRP) proposed a research program to develop an Mechanistic Empirical Pavement Design Guide (MEPDG) using deformation models calibrated from the pavement performance data collected in Long Term Pavement Performance (LTPP) Program under the project name NCHRP1-37A.

The new MEPDG developed under the project NCHRP 1-37A has many benefits compared to the 1993 AASHTO design guide in achieving better pavement designs. The new MEPDG which is user friendly requires more than hundred inputs in four different modules: traffic, pavement structure, climate and materials. There are few major inputs like asphalt properties, Average Annual Daily Truck Traffic (AADTT) and resilient modulus which effects the pavement design and performance. The asphalt binders, pavement structures and resilient modulus which are commonly used in the state of Texas are used in the MEPDG at different climatic conditions.

The main input page of the program is shown in Figure 2.16 that includes general pavement design information, traffic, climatic information, pavement structure information, individual pavement layer information, and other input parameters affecting the life of pavement. The analysis identifies suitable pavement structures and individual layer properties that can satisfy the specific design life.

After all of the inputs have been entered, the software looks for the input errors and stops if there is an error. The entered material properties are used in models for generating material properties and if material properties are entered incorrectly then they cannot be used in the model. If the entered properties are correct, the program performs analysis which typically takes more than 20 minutes depending on computer used. A typical output generated is shown in Table 2.8. The output is compared with the desired target and reliability. If the predicted distress is more than the target then program shows that the pavement will fail within the design life.

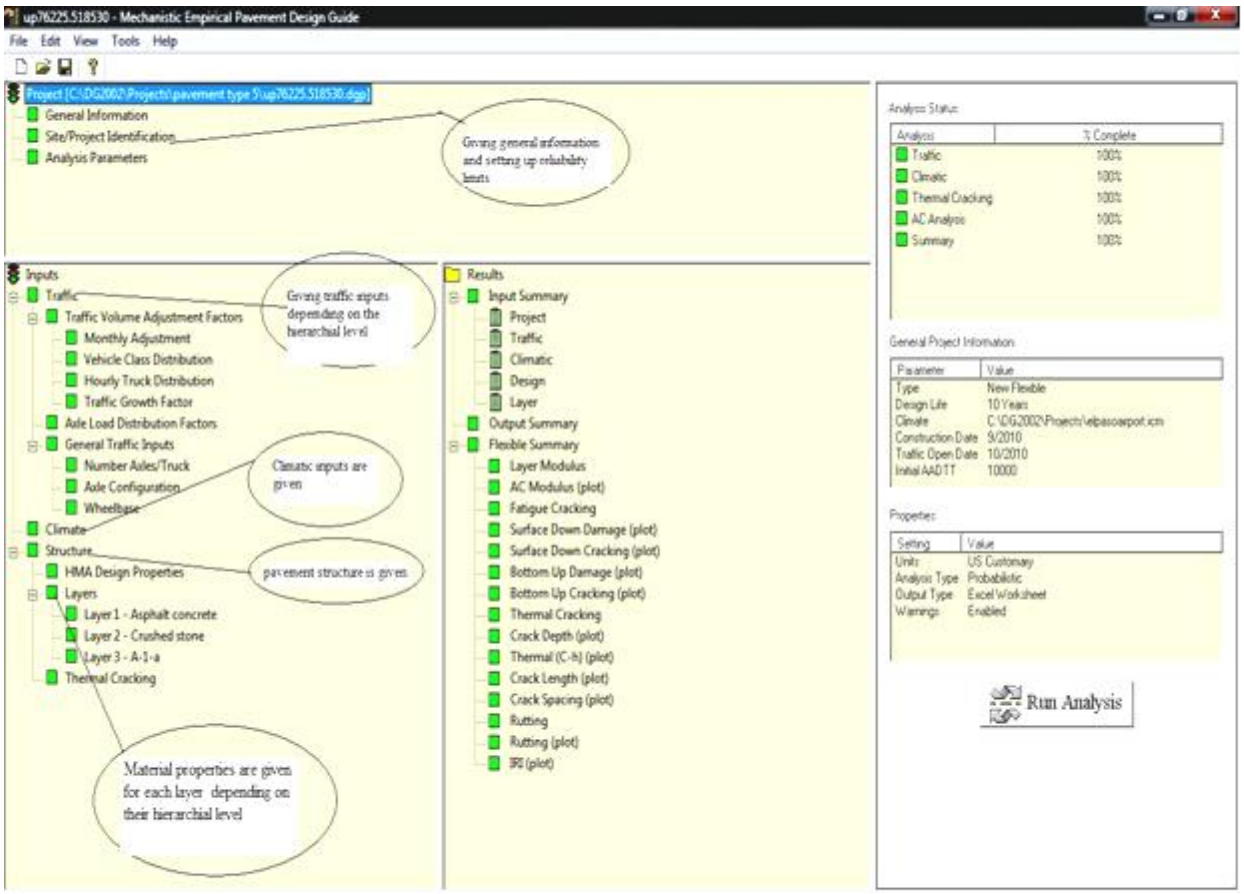


Figure 2.15 MEPDG Software Showing the Input Parameters

Table 2. 8 MEPDG Reliability Output

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mi)	172	90	98.6	99.59	Pass
AC Surface Down Cracking (Long. Cracking) (ft/mile)	2000	90	0.1	99.999	Pass
AC Bottom Up Cracking (Alligator Cracking) (%):	25	90	0	99.999	Pass
AC Thermal Fracture (Transverse Cracking) (ft/mi):	1000	90	1	99.999	Pass
Chemically Stabilized Layer (Fatigue Fracture)	25	90			N/A
Permanent Deformation (AC Only) (in):	*0.25*	*90*	*0.38*	*12.66*	*Fail*
Permanent Deformation (Total Pavement) (in):	*0.75*	*90*	*0.71*	*61.14*	*Fail*

2.4 RESEARCH APPROACH

Although software has been developed, the reliability of the software in terms predicting performance is not well known. The software requires more than 100 inputs which may not significantly impact the performance or may not be readily available. Therefore, better understanding of the software is required. The understanding and availability of inputs will govern experiment design of this study. To achieve the objectives of this study, the following approach is proposed:

1. Develop an understanding of MEPDG software and identify inputs required for the software.
2. Identify inputs that significantly influence pavement performance and are readily available.

3. Develop an experimental design to predict performance of pavements within the state of Texas.
4. In the end, compare MEPDG predicted performance with the IRC:37-2001.

Chapter 3

MEPDG

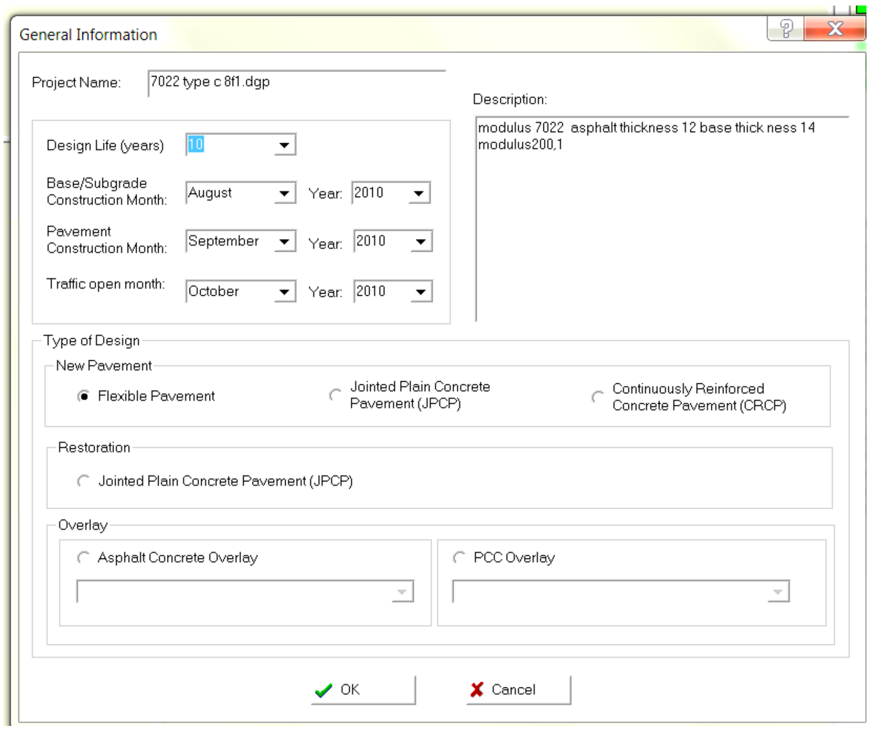
3.1 INTRODUCTION

The need for the development of MEPDG has been discussed earlier. The inputs in MEPDG could be divided in to four different modules namely: project information, traffic, climate, and pavement structure (including material properties of each layer) as shown in Figure 2.16. The inputs given in each module are used in distress models to identify accumulated damage. Each module is discussed in the following sections.

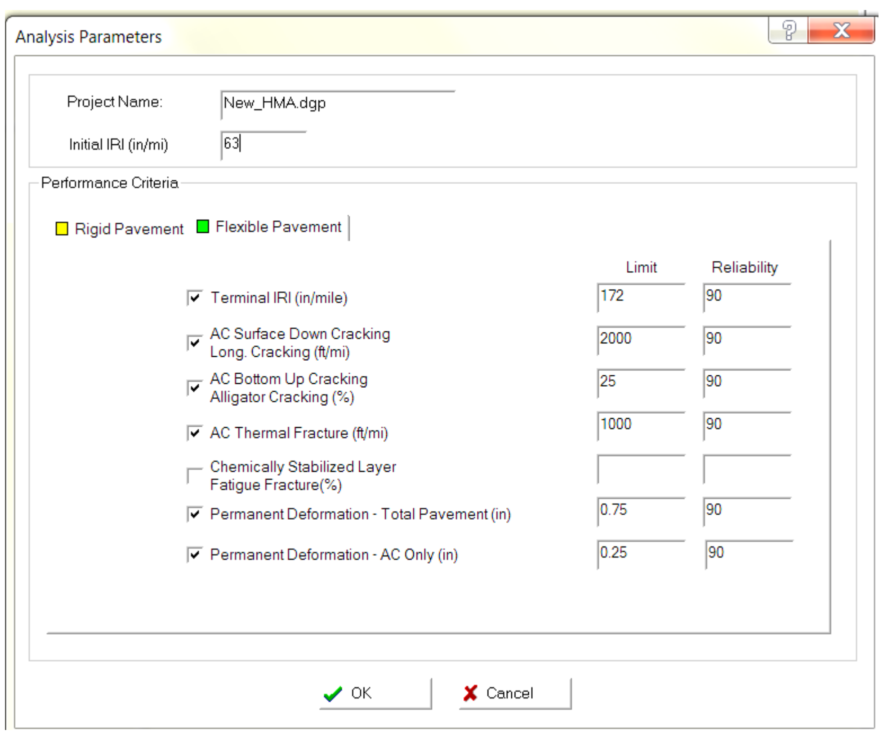
3.2 PROJECT INFORMATION

The project information is entered via three modules: 1) general information, 2) site/project identification, and 3) analysis parameters. The general information module requires information about the design life, type of pavement and whether the analysis is performed for new pavement or rehabilitation of existing pavement as shown in Figure 3.1a. The site/project identification module inputs are mainly for identification of the project like location, direction of traffic, etc.

The analysis parameters module requires information about the failure limit at specific reliability levels as shown in Figure 3.1b. For instance, the permanent deformation in asphalt concrete layer is expected to be less than 0.25 at the end of the service life (10 years) with 90% reliability. The output is displayed in red color if either the permanent deformation is more than 0.25 or the reliability is less than 90%. Similarly, the terminal International Roughness Index (IRI) is expected to be 63 in./mile at the end of the construction and is expected to be less than 172 in./mile at the end of design life. If predicted IRI is more than 172 in./mile then the pavement does not meet the expected design life. The analysis parameters can be changed by the user as per the local or state specifications.



a) General Information



b) Analysis Parameters

Figure 3.1 Project Information Input Required in MEPDG

3.3 TRAFFIC

The traffic input required in the design/analysis of pavement structures is significantly different from the ESAL approach used in the previous versions of design guides for traffic characterization. It uses the full axle load spectra of each axle type for both new and pavement and rehabilitation design procedures. The traffic input module is shown in Figure 3.2 and a brief discussion on relevant inputs is as follows:

- *Initial Two Way Average Annual Daily Truck Traffic (AADTT):* AADTT represents a weighted average between weekday and weekend truck traffic and it could be obtained from Weigh in Motion (WIM) data, Automated Vehicle Counters (AVC), or manual traffic counts. The AADTT value entered in the MEPDG software is after the roadway is opened to traffic or after the rehabilitation has been completed and the value entered should be based on both directions and for all lanes. If only one-way traffic is entered, the percent trucks in the design direction should be set to 100 percent.
- *Percent Truck in Design Lane*: The percent of truck in the design lane typically is determined by estimating the percent of truck traffic in the design lane relative to all truck traffic in one direction. However, the definition used in MEPDG is slightly different; it is defined by the primary truck class for the road way. The primary truck class represents the truck class with the majority of applications using the roadway. In other words, the percentage of trucks in the design lane is estimated for each truck class, and the predominant truck class is used to estimate this value. The percentage trucks in the design lane may be estimated from AVC data or manual vehicle count data.

- *Percent Trucks in Design Direction:* This value represents the percent of trucks in the design direction relative to all trucks using the roadway in both directions. This value may be estimated from AVC data or manual vehicle count data.
- *Operational Speed:* Truck speed has a definite effect on the predicted complex modulus (E*) of HMA and distresses. Lower speeds result in higher incremental damage values calculated by the MEPDG (more fatigue cracking and deeper ruts and faulting). The posted speed limit was used in all calibration efforts. As such, it is suggested that the posted truck speed limit is to be used for evaluating trail designs, unless the pavement is located in a special low – speed area such as a steep upgrade and bus stop.

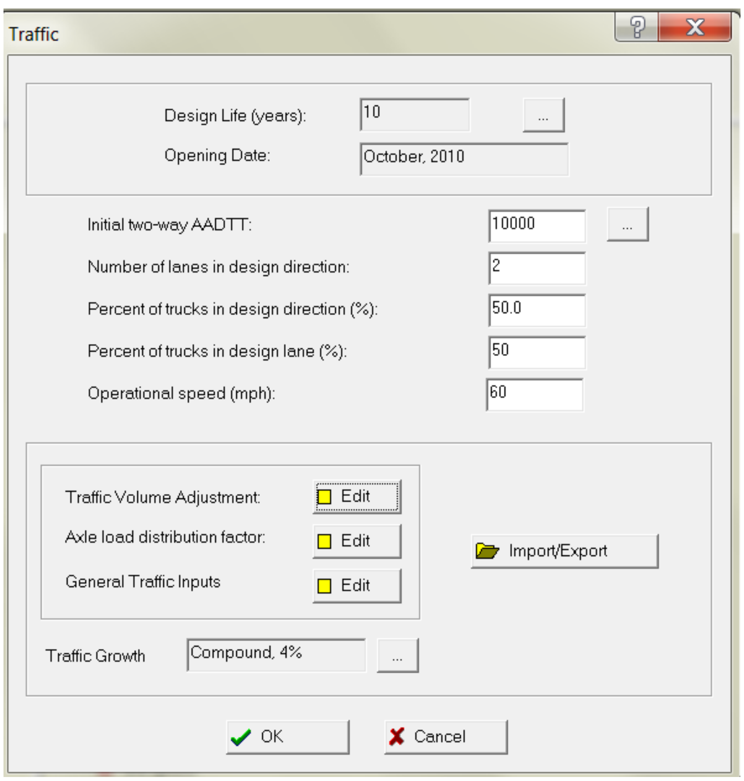
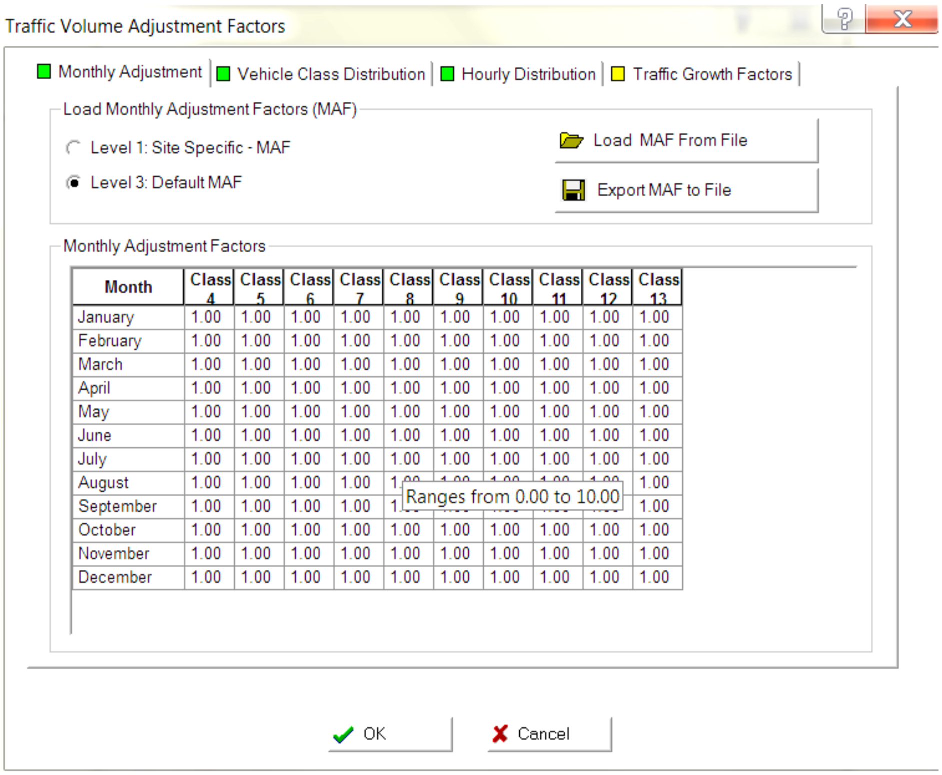


Figure 3.2 Initial Traffic Input Required in MEPDG

In addition to the above mentioned factors, the input module also requires traffic volume adjustment factors, axle load distribution factors, and general traffic inputs. These input modules are discussed as follows:

- *Traffic Volume Adjustment Factors*: The traffic data is adjusted on monthly basis depending on vehicle class distribution and traffic growth factor as follows:
  - *Monthly Distribution Factors:* The monthly distribution factors are used to distribute the truck traffic within each class throughout the year (Figure 3.3). Monthly distribution factors of 1.0 is entered as a default value, however, seasonal changes in truck traffic operations can be expected. These monthly distribution factors may be determined from WIM, AVC, or manual counts.
  - *Vehicle Class Distribution*: The traffic data needs to be entered in terms of the class of the vehicle which has been classified from Class 4 to Class 13 as shown in Figure 3.4. The data can be either entered for site specific conditions or default values can be used. The total AADTT by vehicle class should be equal to 100.
  - *Hourly Distribution*: The truck traffic can vary depending on the time of the day which will govern damage due to change in asphalt concrete stiffness. Again, the hourly distribution can be entered by user or default values can be used. A typical default hourly distribution is shown in Figure 3.5.
  - *Growth of Truck Traffic*: The growth of truck traffic is difficult to estimate accurately because there are many sites, social and economic factors that are difficult and impossible to predict over 20+ years. The MEPDG has the capability to use different growth rates for different truck classes (Figure 3.6),



Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.00
September	1.00	1.00	1.00	1.00	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Figure 3.3 Traffic Volume Adjustment Factors Used in MEPDG

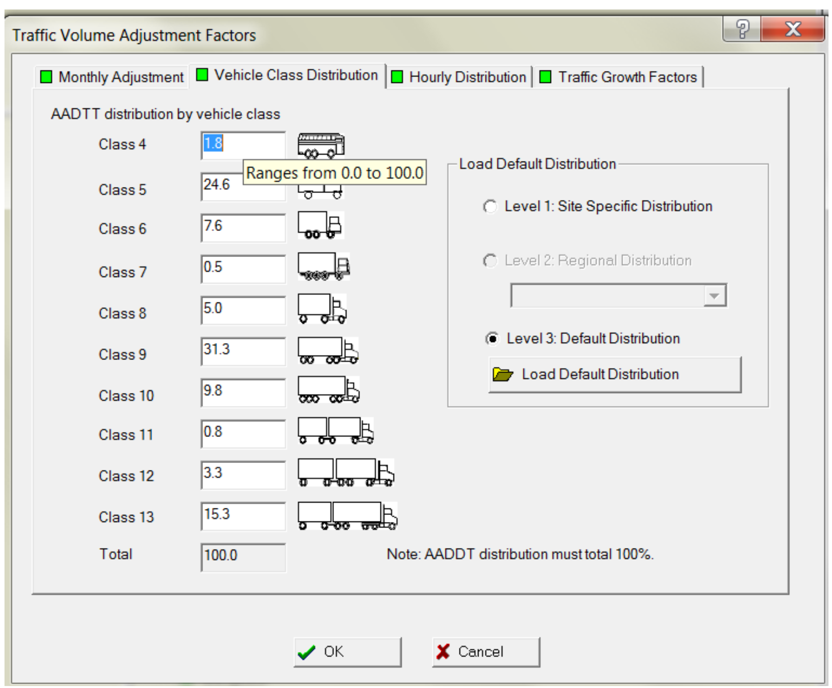


Figure 3.4 Default Vehicle Class Distribution Used in MEPDG

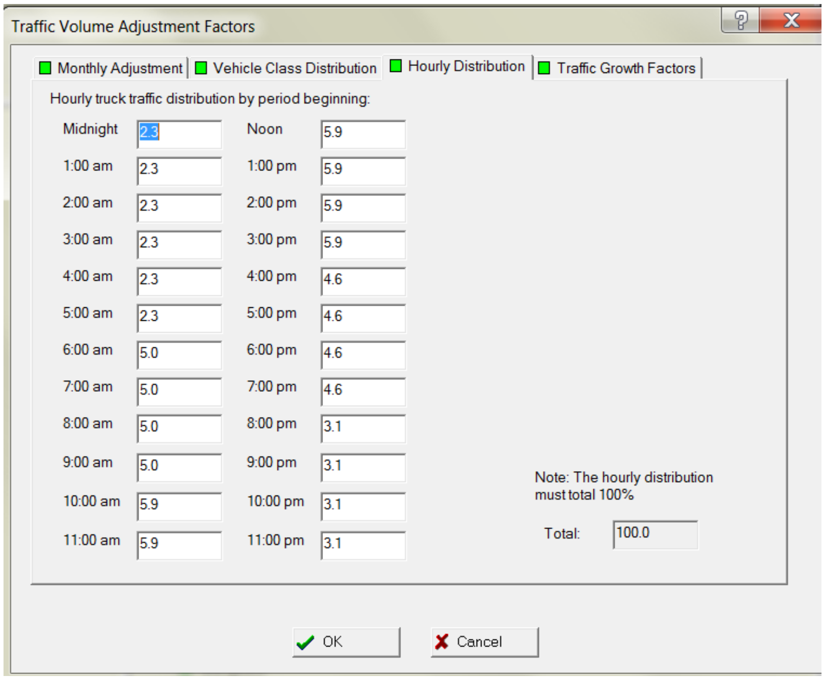


Figure 3.5 Default Vehicle Hourly Distribution Used in MEPDG

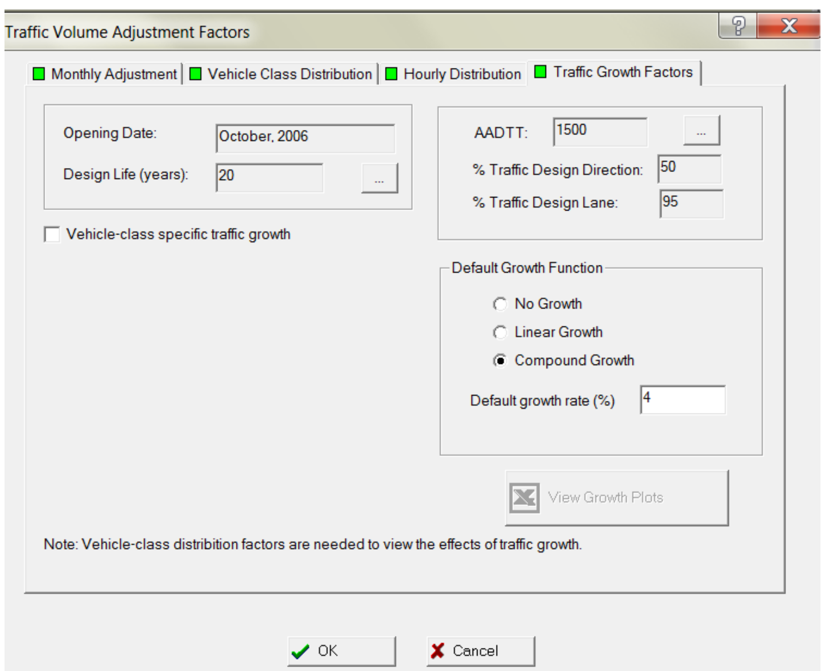


Figure 3.6 Default Traffic Factors Used in MEPDG

but assumes that the growth rate is independent over time; in other words the rate of increase remains same throughout the analysis period. Growth rates depending on truck class have a significant effect of predicted pavement performance and may be determined with as much information as possible about commodities being transported within and through the project location.

- *Axle- Load (single, tandem, tridem, and quads) Distribution Factor:* The axle-load distribution (Figure 3.7) represents a massive amount of data and the data processing should be completed external to the MEPDG software. There are multiple software tools or packages available for processing the axle load distributions data, including the NCHRP Project 1-39 software. These software tools have varying capabilities and functionality, and users may want to evaluate the options so as to select the tool most suitable to their agency needs. The average normalized truck- volume distribution is needed when limited WIM data are available to determine the total axle-load distribution for a project. The normalized distribution represents the percentage of each truck class within truck traffic distribution. This normalized distribution is determined from an analysis of AVC data and represent data collected over multiple years. The default normalized truck volume distributions are shown in Figure 3.7.
- *General Traffic Inputs*: The software also requires general truck information (Figure 3.8) like axle load configuration, wheelbase, etc. The following inputs are needed:
    - *Tire Pressure*: The software assumes a constant tire pressure of 120 psi for all loading conditions. It is recommended that this value be used, unless hot inflation pressures are known from previous studies or a special loading condition is simulated.

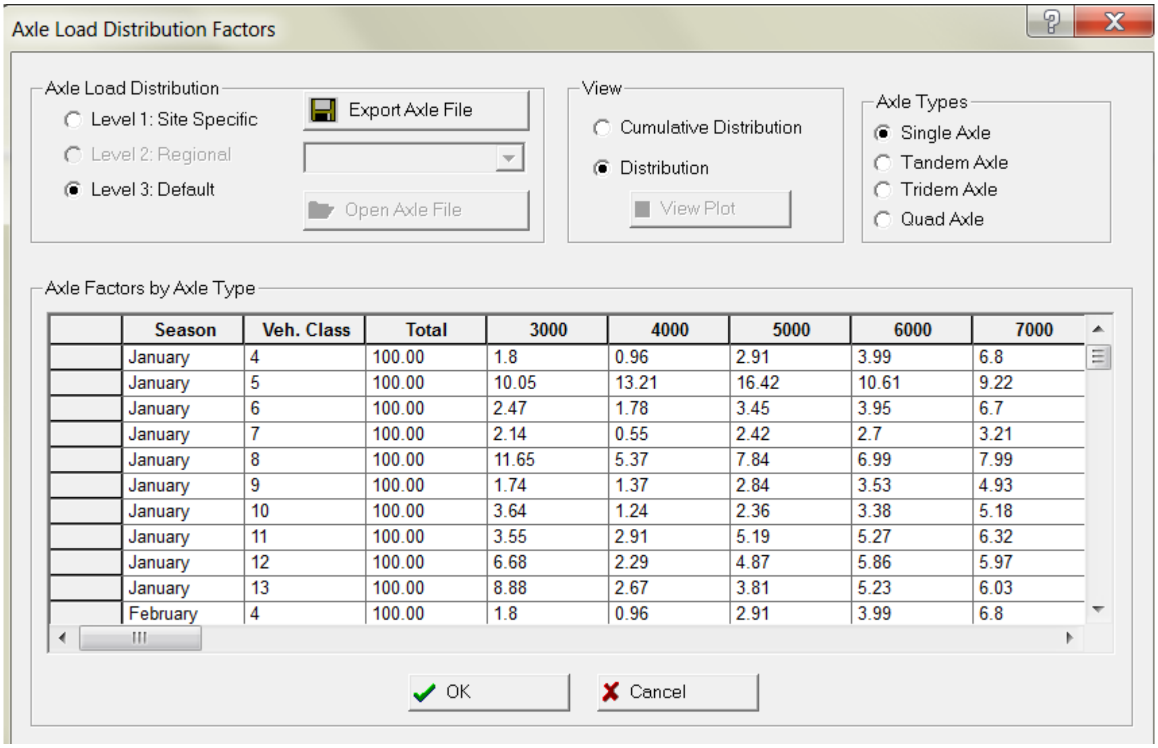


Figure 3.7 Axle Load Distribution Factors Used in MEPDG

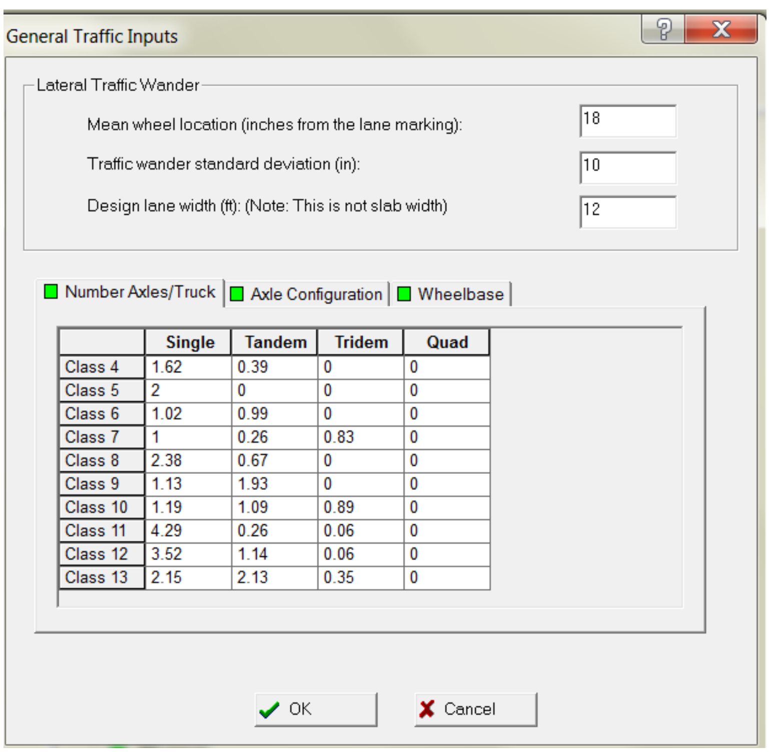


Figure 3.8 General Traffic Inputs Module

- *Axle Load Configuration:* The spacing of the axles is recorded in the WIM database. These values have been found to be relatively constant for the standard truck classes. The values have been found to be relatively constant for standard truck classes. The values used in all calibration efforts are listed below and suggested for use, unless the predominant truck class has a different configuration.
  - Tandem axle spacing; 51.6 in.
  - Tridem axle spacing; 49.2in.
  - Quad axle spacing; 49.2 in.
- *Dual Tire Spacing:* The MEPDG software assumes that all standard truck axles included in the WIM data contain dual tires. The dual tire spacing should represent the majority of trucks using the roadway and taken from trucking industry standards. The default value of 12 in. was selected based on the spacing of the tires used by most trucks. It is recommended that this default value be used unless the predominant type of truck has special loading conditions. The use of super single tires or single tires may be simulated in the MEPDG software by using the special loading conditions or simply increasing the dual tire spacing to a value where the influence is from one of the dual tire is insignificant to the other. This distance between the dual tires for this to occur is 60 in. for most cases.
- *Lateral Wander of Axle Loads:* The software assumes a constant wander for all trucks. A value of 10 in. was used for all calibration efforts, independent of the lane width. In some urban areas, narrower lane widths have been built

because of right of way, etc. For narrow lane widths (less than 10 ft.), it is recommended that a lower lateral wander value be used; a value of 8 in. is suggested unless the user has measured this value. Similarly, for wide lanes (more than 12 ft.) it is recommended that a higher lateral wander value be used; a value of 12 in. is suggested unless the user has measured this value.

3.4 CLIMATE

The impacts of the environmental effects were not considered in the earlier design procedures which influences the pavement performance. In MEPDG, environmental effects are given importance and the information required in the climatic module is depth of water table, latitude, longitude and elevation of the site (Figure 3.9). A user could virtually develop a weather station from the latitude longitude and depth of water table. There are around 800 weather stations in the North America collecting the weather information such as hourly temperature, precipitation, wind speed, relative humidity and cloud cover, which are used to predict the temperature and moisture content in each of the pavement layers and also provide required information for the predicting smoothness of the pavement.

3.4 PAVEMENT STRUCTURE AND MATERIALS INPUT

This section gives the information of the inputs in pavement structure and material modules of MEPDG. The MEPDG pavement structure module can analyze and pavement designs with up to 20 layers. Each layer type and thickness is added by using the insert option in the module as shown in Figure 3.10. The edit and delete tabs in the input page allows modification of each layer data. The asphalt concrete prediction models and data required for each layer is discussed in the following sections.

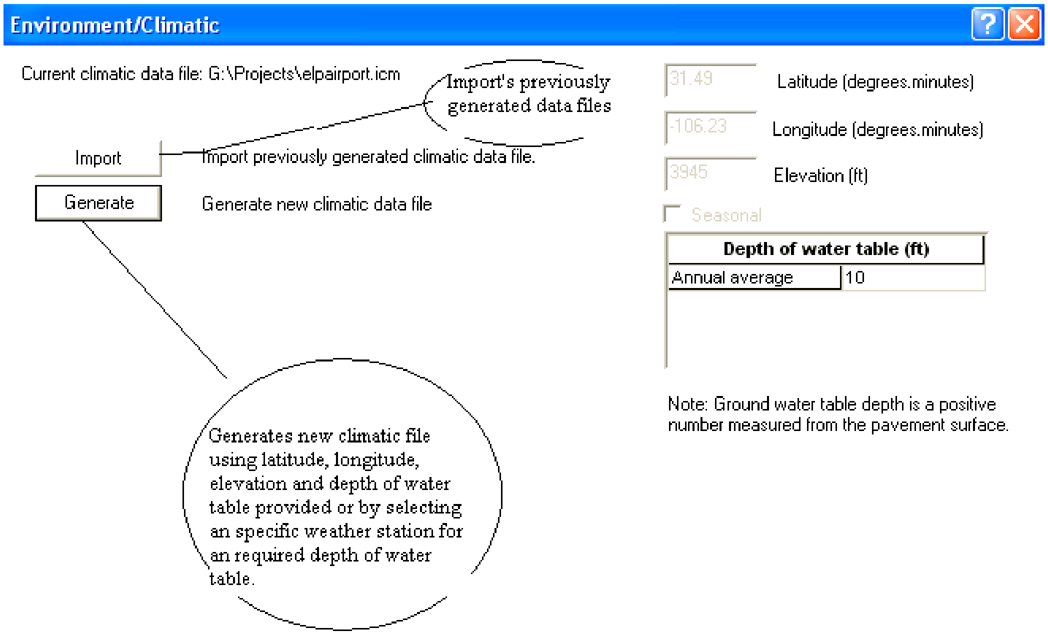


Figure 3.9 Climatic Input Module

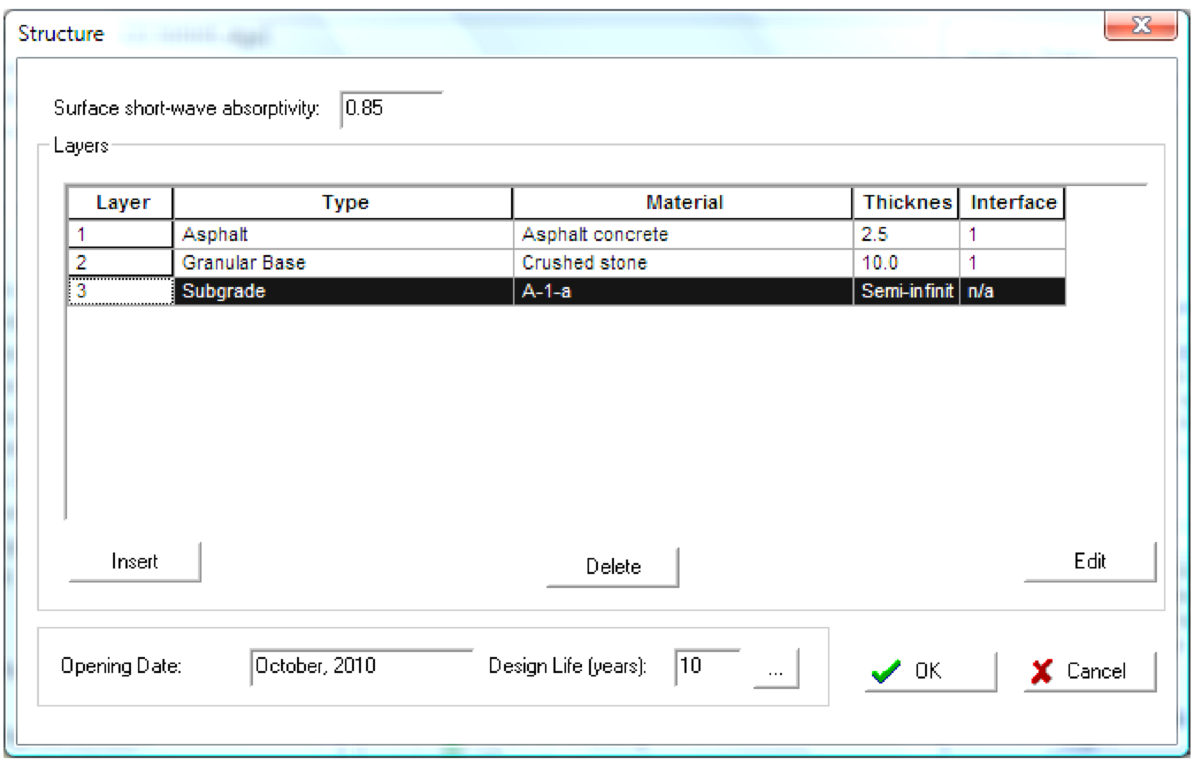


Figure 3.10 Pavement Structure Input Module

3.4.1 HMA DESIGN PROPERTIES

The software predicts properties of asphalt concrete at different temperatures using models shown in Figure 3.11. The pavement design could be evaluated with viscosity based model or G* based model which is revision of viscosity based model. The input option of G* based model has less inputs for the asphalt layer and it is uncalibrated. Therefore, the viscosity based models were used in NCHRP Project 1-37A.

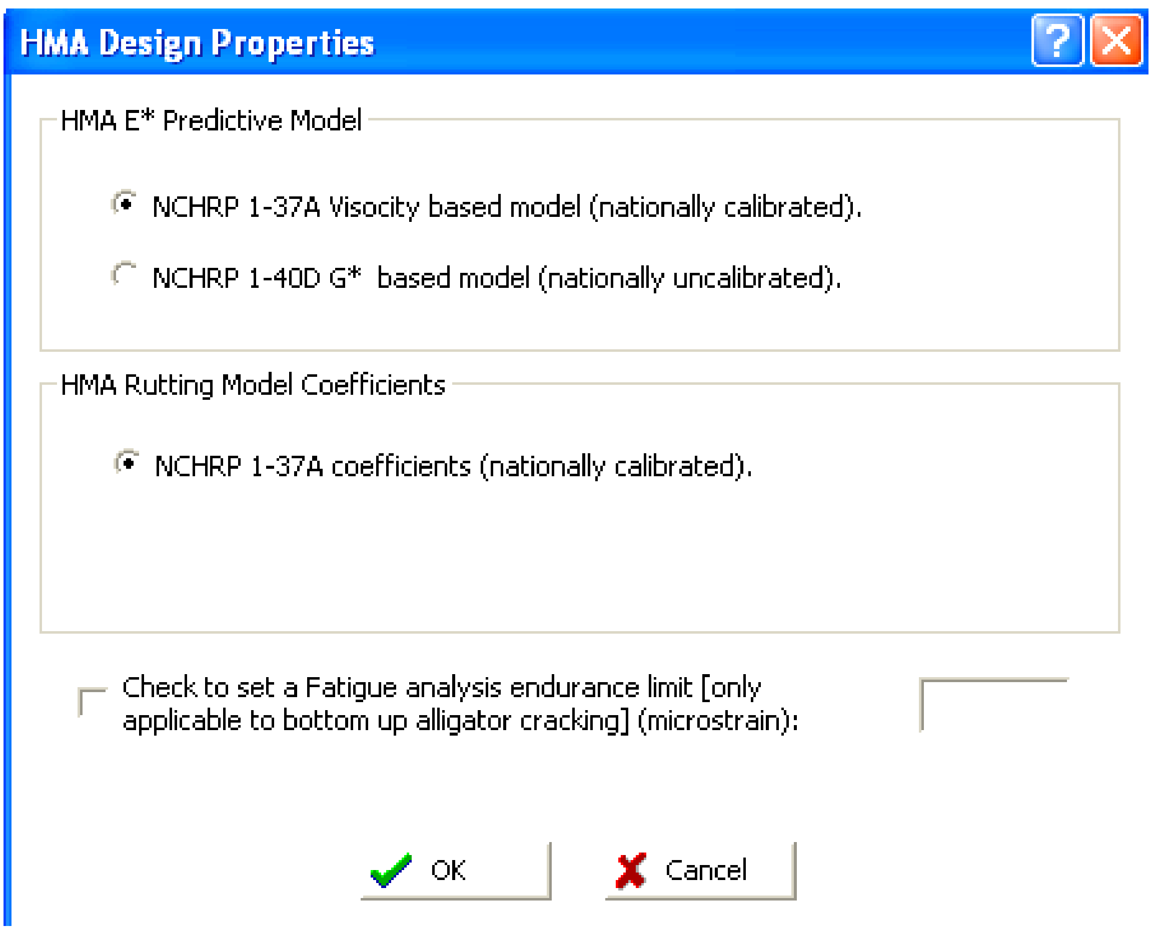


Figure 3.11 HMA Design Properties

3.4.2 ASPHALT CONCRETE LAYER

The input requirement for asphalt concrete layer has been significantly changed from previous design guides. It takes into consideration visco-elastic behavior of asphalt concrete and uses dynamic modulus (|E*|) of asphalt concrete to estimate change in stiffness and strength of asphalt concrete with changes in environmental conditions. Since new tests for characterization of asphalt concrete have been developed, it is not necessary that all of the needed information about asphalt concrete will be available. Therefore, three levels of input are suggested based on the available material information. Each of the levels is discussed as follows:

- Each level of input can be subdivided in three additional inputs: 1) Asphalt Mix, 2) Asphalt Binder, and 3) Asphalt General. The Asphalt General input module is similar for all three levels of design. The Asphalt Mix input module is similar for Level 2 and Level 3 while different for Level 1. The Asphalt Binder input module is similar for Level 1 and Level 2 but is different for Level 3.
- *Level 1*: This level is used for either heavily trafficked roads or where advanced asphalt concrete mix design information including dynamic modulus measurements are available. The Asphalt Mix input module (Figure 3.12) requires preparation of specimens and testing, as per AASHTO 315 procedure (Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR), to measure dynamic modulus of the specimens at different temperatures. The tests can be performed at a minimum of three temperatures. If tests are performed at five temperatures, a total of 15 specimens are at least needed for characterizing of asphalt concrete which requires significant amount of laboratory testing which may not be feasible for smaller projects.

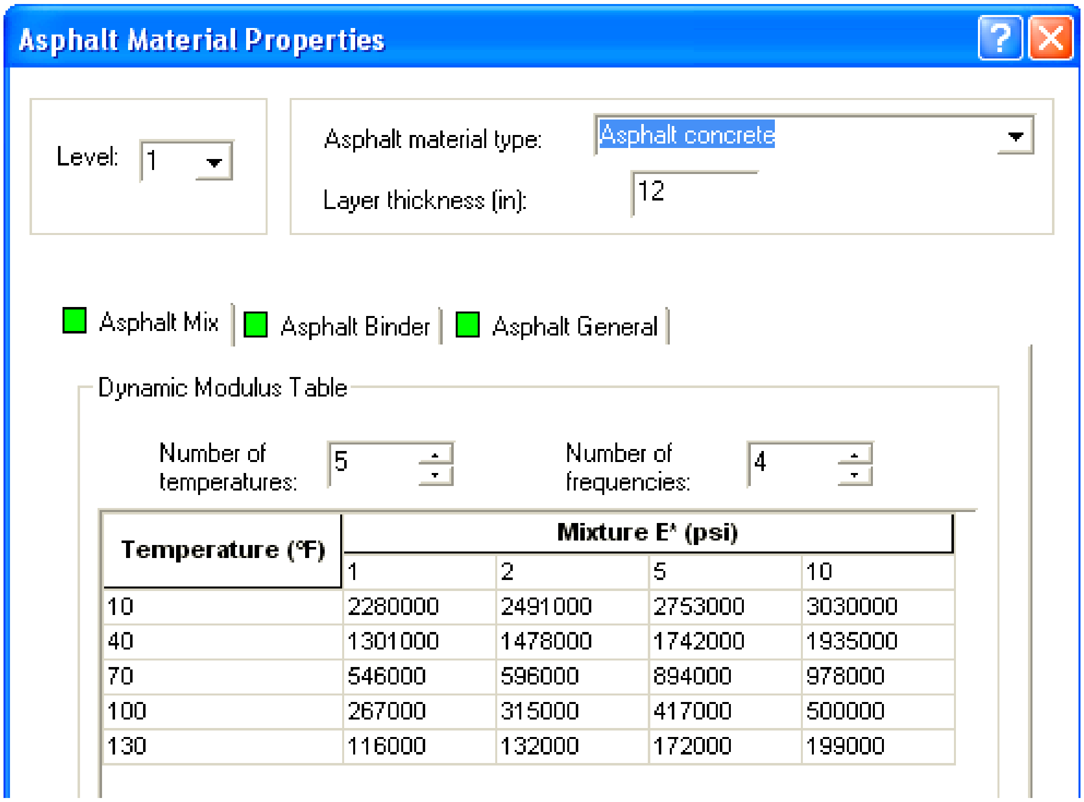


Figure 3.12 Level 1 Dynamic Modulus Input Module

The Asphalt Binder input module either requires Superpave Binder test results (G* and sinδ) as shown in Figure 3.13 or conventional binder test results like viscosity, penetration, etc. as shown in Figure 3.14 In recent years, Superpave grading system has been commonly used; therefore, this input data is easily available in USA. However, this data may not be available for smaller projects in India.

The Asphalt General input module (Figure 3.15) requires additional information related to the mix design used for the pavement. It requires thermal conductivity and heat capacity of the asphalt concrete. Therefore, it is left as a default value provided by the software. The software uses a reference temperature of 70 °F, to account for temperature

and rate of loading effects, for all analysis levels to compute dynamic modulus. It can be changed to other temperatures but is typically conducted at 70 °F. Similarly, Poisson's ratio of the asphalt concrete is kept constant at 0.35. The volumetric properties of the mixes changes from one mix type to another one and is needed for analysis.

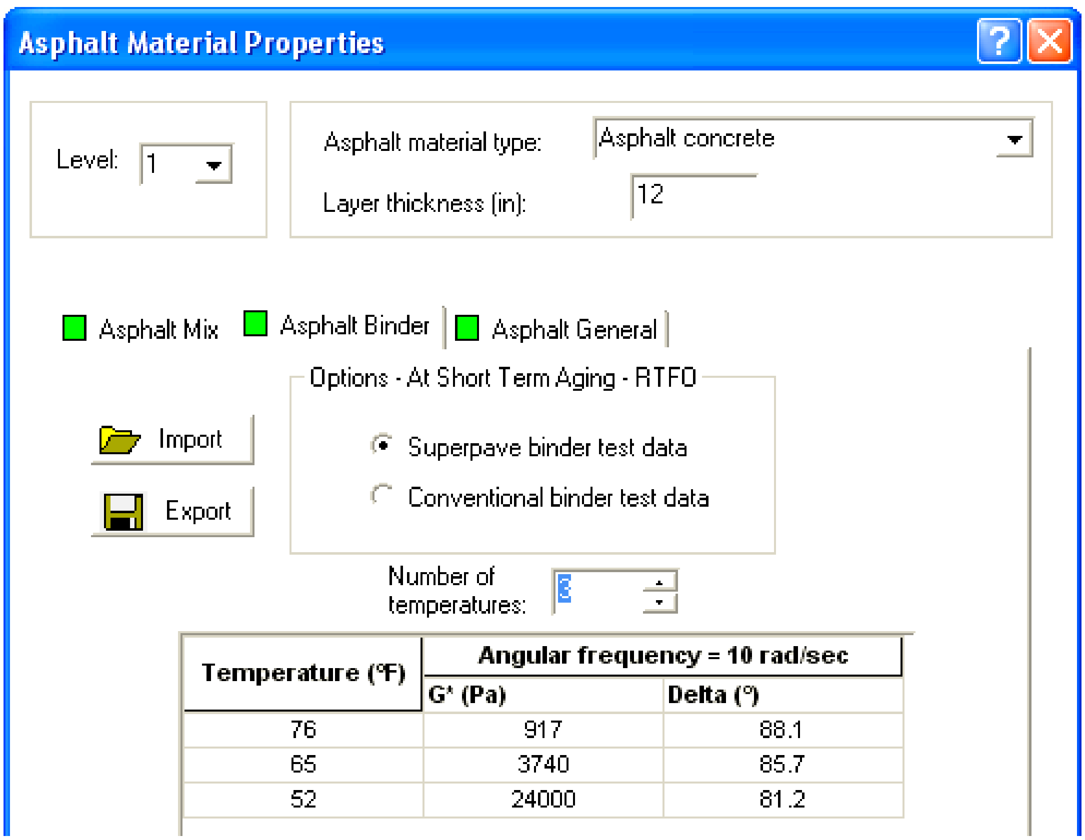


Figure 3.13 Level 1 Superpave Input Module

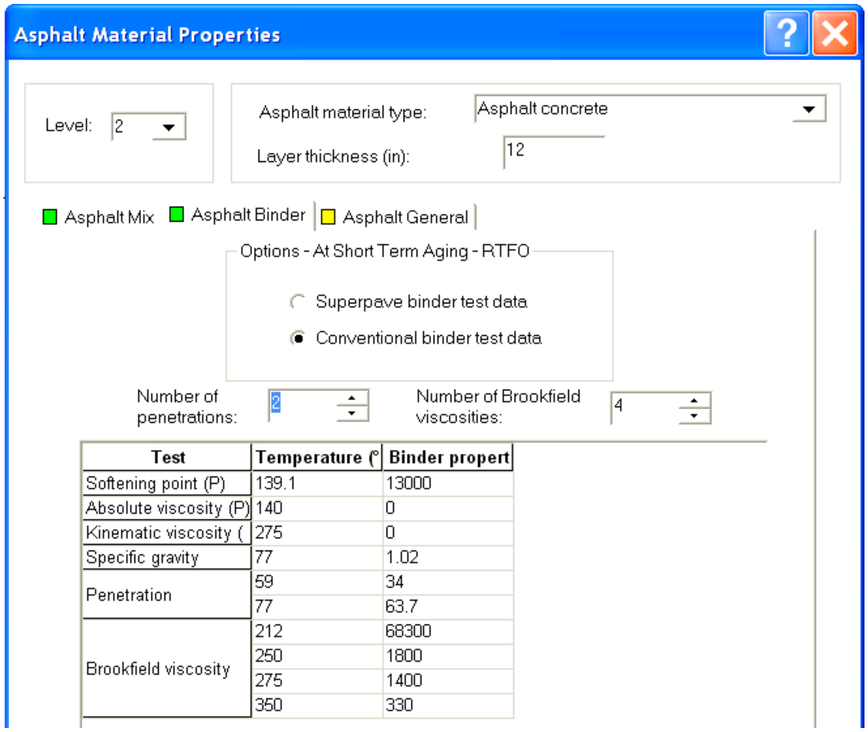


Figure 3.14 Level 1 Conventional Binder Input Module

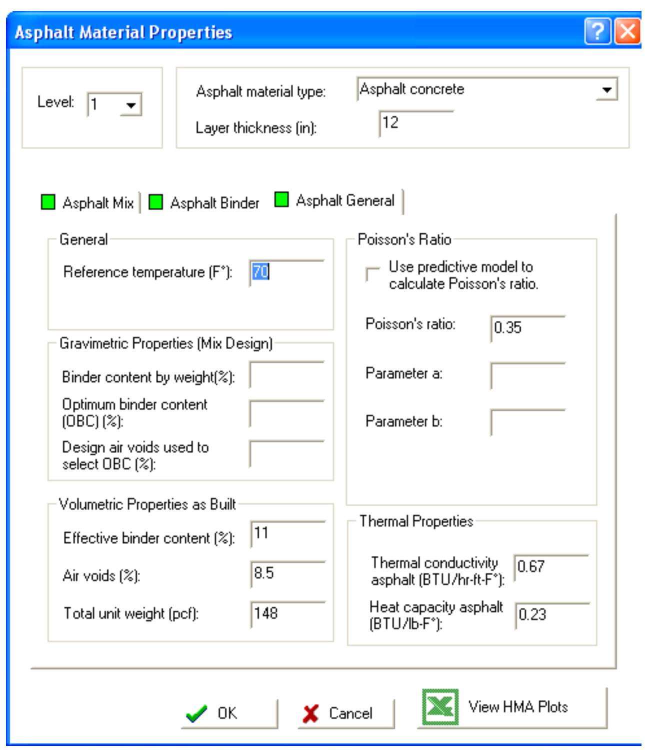


Figure 3.15 General Binder Input Module

- *Level 2:* The level 2 input data for the Asphalt Binder and Asphalt General input module are the same. However, asphalt mix information requirement is different as shown in Figure 3.16. Since it is anticipated that dynamic modulus information may not be available for mixes, the software generates dynamic modulus information from the binder type and gradation of aggregates used in the mix as shown in Figure 3.16. This information is available for all mix types within USA as well as in India.

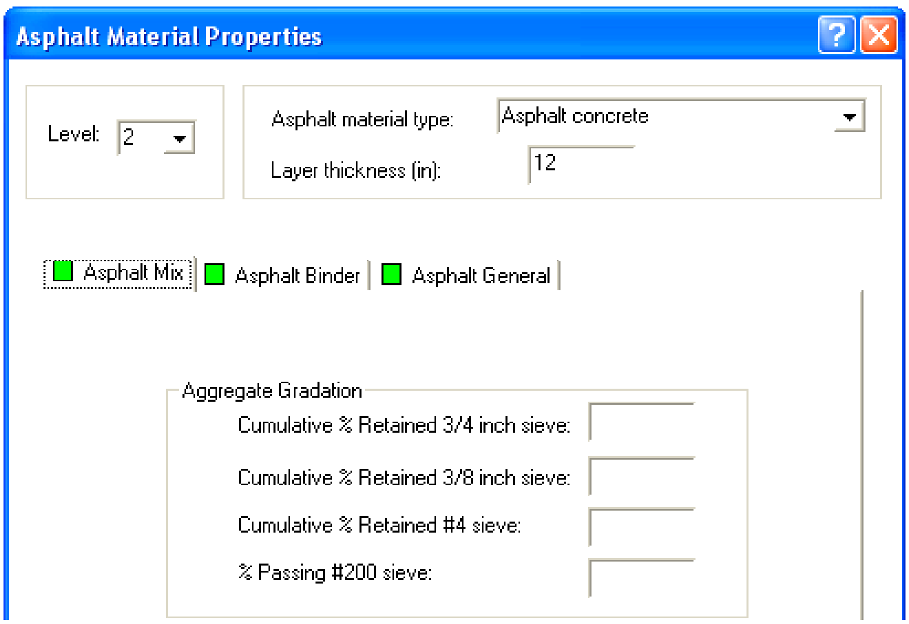


Figure 3.16 Level 2 Asphalt Concrete Mix Input Module

- *Level 3*: The level 3 input is similar to the Level 2 with only exception of asphalt binder. The software uses binder grading information either Superpave (Figure 3.17) or conventional viscosity grade (Figure 3.18) or penetration grade (Figure 3.19) to estimate dynamic modulus of asphalt concrete. It uses existing models for different grade types and can be used for lower level projects for which the information may not be available.

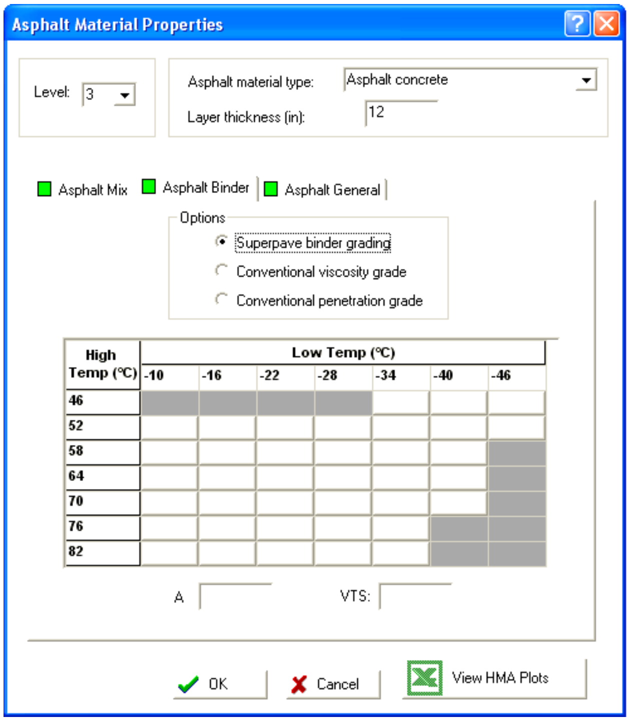


Figure 3.17 Level 3 Superpave Binder Input Module

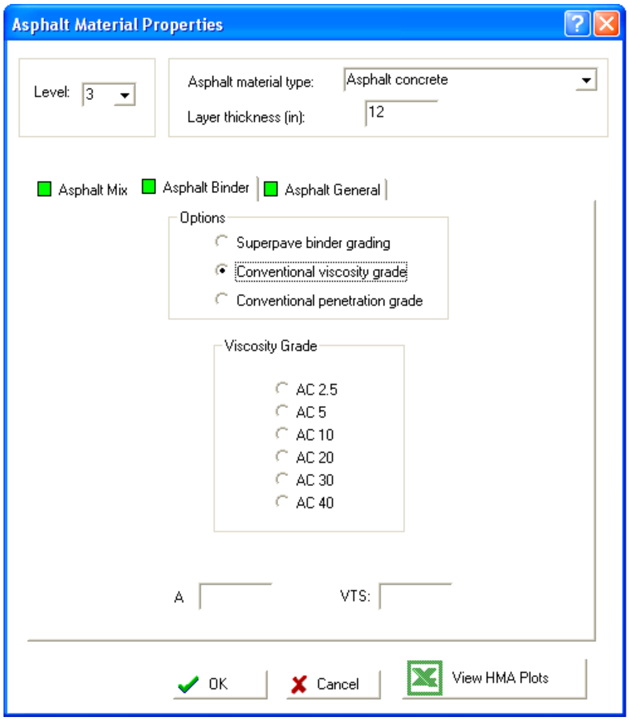


Figure 3.18 Level 3 Viscosity Grade Binder Input Module

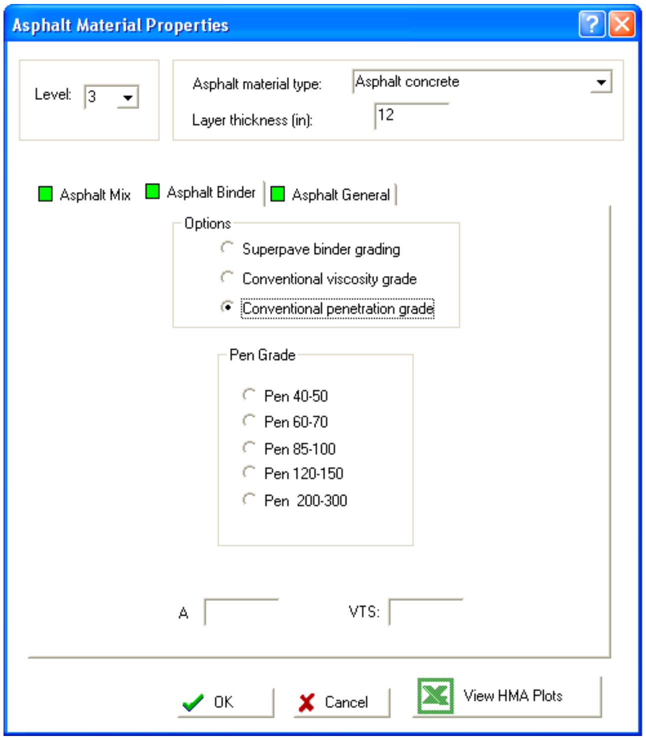
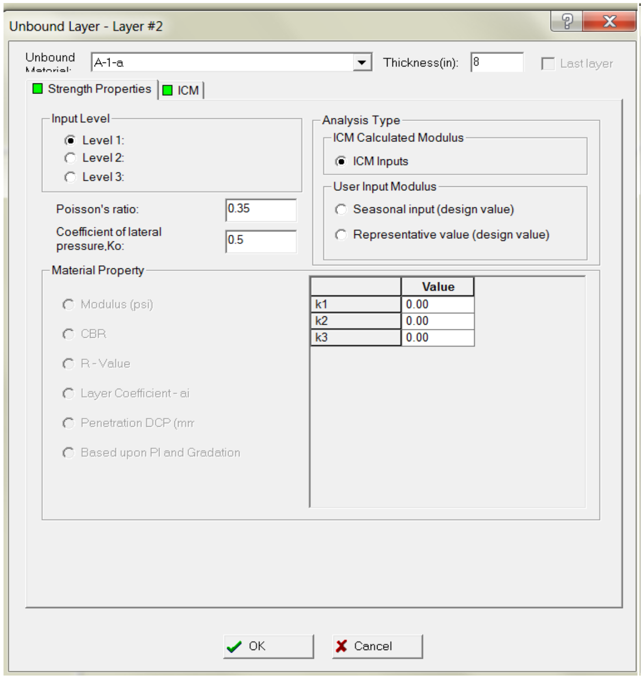


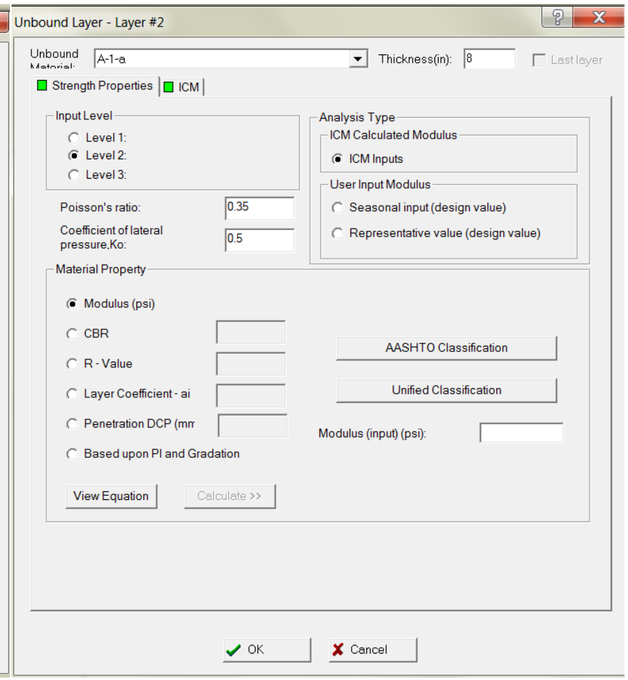
Figure 3.19 Level 3 Penetration Grade Binder Input Module

3.4.2 BASE, SUBBASE, AND, SUBGRADE LAYER

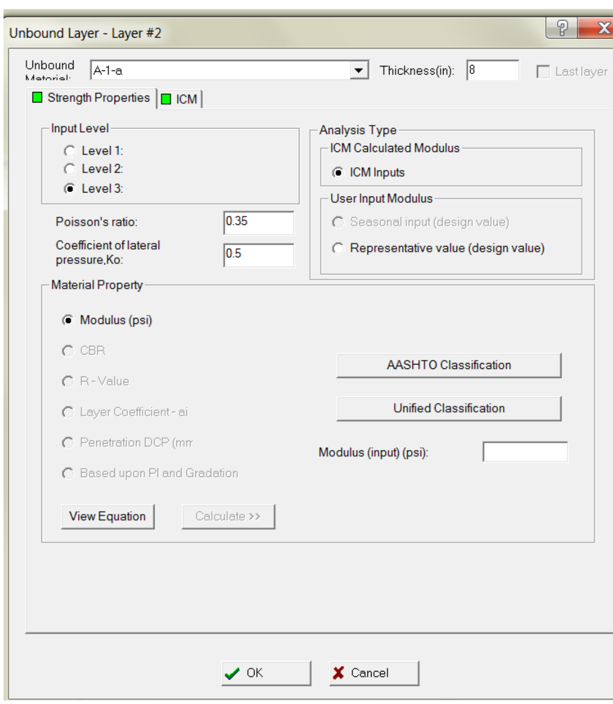
The input modules for base, subbase, and subgrade layers are similar with one exception of subgrade layer thickness which is assumed to be semi-infinite. The input data required for the analysis are the resilient modulus (M_r), Poisson's ratio, coefficient of lateral earth pressure, thickness and Integrated Climatic Model (ICM) inputs, as shown in Figure 3.20. The layer thickness, Poisson's ratio, coefficient of lateral earth pressure, and ICM inputs remain same for three levels of input. The main difference between three levels of input is the estimation or M_r. The Level 1 input module estimates M_r using k_1, k_2, and k_3 constants which are typically obtained from M_r. testing in the laboratory. In addition, these constants can be entered for seasonal variation if the data is available. The Level 2 input module is used for estimating M_r



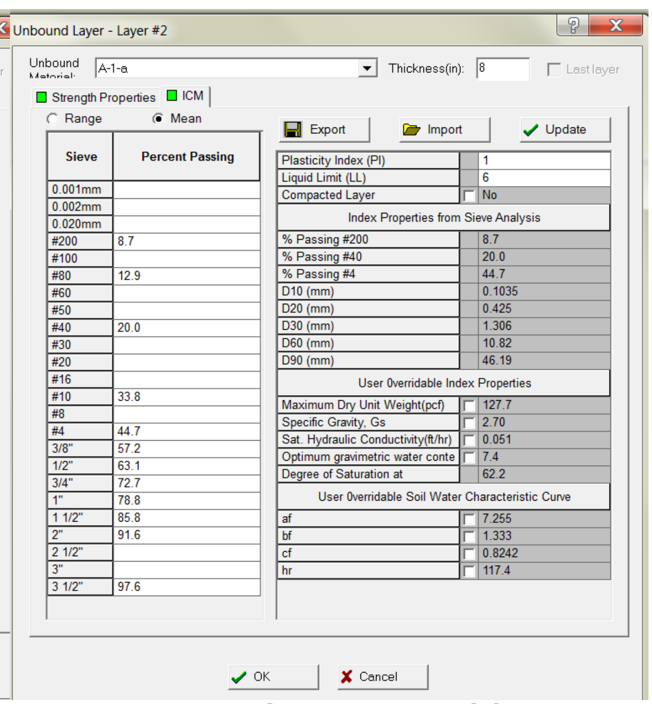
a) Level 1 Input Module



b) Level 2 Input Module



c) Level 3 Input Module



d) ICM Input Module

Figure 3.20 Layer 2 Input Data of Different Levels

values from commonly available soil parameters like California Bearing Ratio (CBR), R-value, etc. The Level 3 input module allows user to directly input M_r values. The ICM input module (Figure 3.20d) requires soil properties like Atterberg Limits, sieve analysis, etc.

3.4.3 CREEP COMPLIANCE FOR THERMAL CRACKING

To predict thermal cracking of asphalt concrete layers, the software uses creep compliance and mixture coefficient of thermal expansion information which is obtained from laboratory testing or can be estimated from the models embedded in the software, as shown in Figure 3.21. There is no major difference between three levels of inputs because most of the information provided for asphalt concrete layer can be used for estimating inputs for this module.

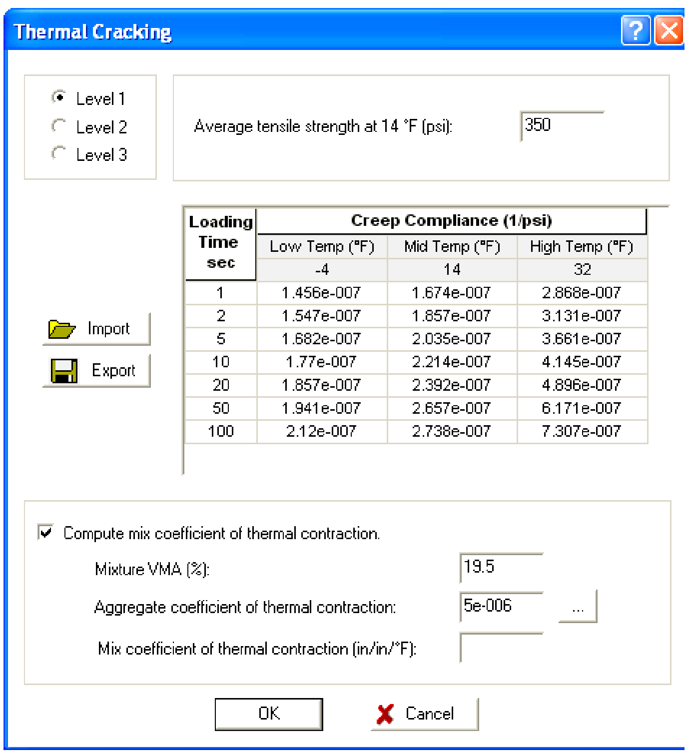


Figure 3.21 Creep Compliance and Thermal Coefficient of Expansion Module

3.5 OUTPUT OF MEPDG SOFTWARE

The output of MEPDG is given in the form of HTML language or in the form of Microsoft Excel sheets. The output consists of multi tabs giving the information on the inputs provided by the users including default values, analysis information like climatic data, distress summary, different graphs showing the different failure criteria's versus time, layer modulus, AC modulus and reliability summary. Only two outputs are discussed here for the sake of brevity.

3.5.1 CLIMATE

The climate sheet (Table 3.1) gives the pavement temperature information estimated from the weather station data available through the software. The information it provides includes the average monthly quintile temperatures of each sub layer of the pavement, hourly air temperatures by month, annual climatic statistics which includes rainfall, wind, sun, frost etc.

3.5.2 DISTRESS TARGET

The program output provides accumulation of different distresses over time and a typical output is summarized in Table 3.2. The first column shows the pavement age which is given in the form of month or the year. The second column shows the starting month. The columns three through eight show the accumulation of distress over time. The predicted stresses are longitudinal cracking, alligator cracking, transverse cracking, and rutting in asphalt concrete layer as well as total pavement. All of the distresses are accumulated in the form of IRI which is shown in Table 3.2 is as column eight. The IRI is based on initial value of IRI provided by the user which represents roughness measured at the end of construction. The program has a default value of 63 but it can be higher. A value of 111.6 shown in Table 3.2 is increase from 63 (default value). In other words, the IRI value will be higher if initial IRI value of more than 63 is selected. The column nine shows accumulation of heavy trucks over time which is based on the input values provided in the traffic module.

Table 3. 1 Average Monthly Quintile Temperatures of Surface Layer

Month	1st Quintile (°F)	2nd Quintile (°F)	3rd Quintile (°F)	4th Quintile (°F)	5th Quintile (°F)	Mean Temp. (°F)	Std. Dev. (°F)
January	56.2	66.0	70.7	77.0	88.2	71.6	11.3
February	62.0	69.7	73.5	81.5	92.4	75.8	10.9
March	65.6	72.0	76.4	86.1	96.9	79.4	11.5
April	68.4	74.1	79.0	89.4	100.8	82.3	12.0
May	74.1	78.0	83.7	94.3	105.1	87.1	11.7
June	77.9	81.3	87.1	96.4	105.6	89.7	10.5
July	79.3	82.7	88.6	98.8	108.0	91.5	10.9
August	79.2	82.4	88.0	98.5	108.4	91.3	11.1
September	78.0	81.0	85.0	94.5	104.8	88.6	10.1
October	73.4	77.8	81.0	89.4	99.4	84.2	9.6
November	66.8	72.8	76.3	83.0	92.9	78.4	9.4
December	59.6	68.7	73.1	78.5	88.5	73.7	10.2

Table 3. 2 Predicted Distresses

Pavement Age		Month	Long. Crack. (ft/mi)	Alligator Cracking (%)	Transverse Cracking (ft/mi)	Subtotal AC Rutting (in)	Total Rutting (in)	IRI (in/mi)	Heavy Trucks (cumulative)
mo	yr								
1	0.08	Oct.	0	0.24	0	0.017	1.212	111.6	144,578
2	0.17	Nov.	0	0.41	0	0.02	1.307	115.5	289,156
3	0.25	Dec.	0	0.52	0	0.021	1.351	117.4	433,734
4	0.33	Jan.	0	0.61	0	0.022	1.379	118.6	578,313
5	0.42	Feb.	0	0.70	0	0.023	1.406	119.7	722,891
6	0.5	Mar.	0	0.81	0	0.024	1.433	120.9	867,469
7	0.58	Apr.	0	0.91	0	0.025	1.454	121.8	1,012,050
8	0.67	May	0	1.04	0	0.026	1.479	122.9	1,156,630
9	0.75	Jun.	0	1.17	0	0.028	1.504	124.0	1,301,200
10	0.83	Jul.	0	1.31	0	0.031	1.528	125.0	1,445,780
11	0.92	Aug.	0	1.45	0	0.032	1.548	125.9	1,590,360
12	1	Sep.	0	1.54	0	0.033	1.559	126.4	1,734,940
13	1.08	Oct.	0	1.61	0	0.033	1.568	126.9	1,885,300
14	1.17	Nov.	0	1.65	0	0.033	1.572	127.1	2,035,660
15	1.25	Dec.	0	1.69	0	0.033	1.576	127.3	2,186,020
16	1.33	Jan.	0	1.72	0	0.033	1.58	127.5	2,336,380
17	1.42	Feb.	0	1.76	0	0.033	1.584	127.7	2,486,740
18	1.5	Mar.	0	1.80	0	0.033	1.589	128.0	2,637,110
19	1.58	Apr.	0	1.83	0	0.034	1.593	128.2	2,787,470

3.6 INPUTS USED FOR SENSITIVITY EVALUATION OF MEPDG

The inputs required for the software are not always available; therefore, it is necessary to use default values for performing the sensitivity analysis. The sensitivity analysis was performed using New_HMA.dgp file which is provided with the software. In case data was not available, the default values included in the file were used. The available data used in the sensitivity analysis is discussed in the following sections.

3.6.1 PROJECT INFORMATION

This module requires information about the target failure criterion, the general project information like starting of the construction, type of construction, etc. In this study, the sensitivity runs were performed for new construction of flexible pavements with the design life of 10 years. The remaining information was similar to the one provided by the New_HMA.dgp file.

3.6.2 TRAFFIC

MEPDG requires very detailed truck traffic information, including the hourly and monthly variation of traffic volume, axle load spectra of different axle types for different truck classes, tire pressure, axle spacing, traffic wander and other general traffic data. From the traffic data collected by Caltrans (Zhang and Harvey, 2009 and Tran and Kevin, 2007), it is found that the AADTT at some of interstate junctions is more than 10,000. In addition, the Indian roads also experience heavy traffic loads; therefore, the selection of higher AADTT is more appropriate for the analysis. Therefore, the pavement structure was analyzed for AADTT value of 10,000. The remaining information was similar to the one provided by the New_HMA.dgp file.

3.6.3 CLIMATE

A detailed climatic data which includes hourly temperature, precipitation and wind speed is needed for predicting pavement distress with the MEPDG. The impacts of different climatic regions were analyzed by using El Paso, Houston, and Miami weather station data provided with the software.

3.6.4 PAVEMENT LAYER THICKNESS AND MATERIAL PROPERTIES

The main focus of this study was to identify appropriate pavement structures and asphalt concrete properties that influence design life of pavements. Therefore, pavement structure data for Texas conditions was obtained and used for the analysis.

The pavement layer thicknesses and modulus values encountered within the state of Texas has been collected by Murphy (1998). The collected data consists of different pavements types. Since the analysis focused on flexible pavements, the selected pavement layer thicknesses are shown in Table 3.3. Murphy had identified subgrade layer thickness as well as asphalt concrete layer modulus values. Since program assumes subgrade to be semi-infinite and requires dynamic modulus rather than elastic modulus, these inputs were either not used or modified in this study. In addition, the base and subgrade resilient modulus values are required while Murphy's survey provided only elastic modulus values. To provide resilient modulus values, a relationship developed by Ping, et al. (2002) between resilient modulus and elastic modulus was used to convert elastic modules from Murphy's survey to resilient modus values for analysis. Another important issue to keep in mind is that the minimum elastic modulus of 1 ksi for Subgrade and 5 ksi for base may not be commonly used but it is quite possible for Indian road conditions.

Table 3. 3 Flexible Pavement Structure Used in Texas (Murphy,1998)

Flexible Pavements	
Asphalt Concrete Surface Thickness (in)	6, 8, 10, 12
Base Thickness (in)	6, 10, 14, 18
Base Modulus (ksi)	5 and 171
Subgrade Modulus (ksi)	1 and 40

The commonly used asphalt binders and asphalt concrete mix types have been identified by Sugandh et al. (2007). Sugandh et al. and Hrdlicka et al. (2007) also performed dynamic modulus tests on binder as well asphalt concrete mixes. From their study, two mix types (Type D and CMHB-C), and three binder types (PG 67-22, PG 70-22, and PG76-22) were selected for analysis in this study. The binder rheological properties, mix design information, and dynamic modulus values of the selected mix types are reproduced in Tables 3.4 through 3.7.

The asphalt binders used in MEPDG for the evaluations were obtained from Ultrapave and Wright and their Superpave properties are included in Table 3.4. The PG 67-22 was an unmodified binder while PG 76-22 was modified with 3% SBR. The PG 70-22 was modified binder with 3% SBS and was from different source than PG67-22 or PG76-22 binder. Hrdlicka et al. also performed binder tests at different temperatures (Table 3.5) which can be used for Level 1 design (Figure 3.13). The mix types used are CMHB-C and Type D and mix design information is shown in Table 3.6. CMHB C is a coarse mix has an maximum aggregate size of ¾ inch and while Type D is a finer mix having an maximum aggregate size of ½ inch. The dynamic modulus values for five temperatures and four frequencies are shown in Table 3.7.

Table 3. 4 Rheological Properties of Asphalt Binders (Sugandh et.al.,2007)

Asphalt Producer	Ultrapave	Wright	Ultrapave
PG grade	67-22	70-22	76-22
Modifier	0%	3.0% SBS	3.5% SBR
Rotational Viscosity, @ 135°C	0.587	1.4	1.367
Softening Point, F	N/T	137	N/T*
Penetration @25 °C	N/T	56	N/T
G*/sinδ @ 10rad/sec, kPa	2.5	1.517	3.02
Phase Angle @ 10rad/sec	81.1	74.2	70.7
Specific Gravity @ 60°F	N/T	1.038	N/T
Elastic Recovery @ 10°C	N/T	52.5	N/T
RTFO Aging	Not Tested (N/T)		
G*/sinδ @ 10rad/sec, kPa	6.35	3.388	10.6
Phase Angle @ 10rad/sec	85.2	69.8	85.4
Change in mass	N/T	0.019	N/T
PAV Aging	Not Tested (N/T)		
G*/sinδ @ 10rad/sec, kPa	3086	2184.8	2585
S, -12 °C @ 60sec	122	1.335	114
m, -12 °C @ 60sec	0.325	0.3283	0.317

Table 3. 5 G* and Sin d Properties of Asphalt Binders (Sugnadhet.al., 2007)

Binder Type	Temperature °F	Angular frequency = 10 rad/sec	
		G*, psi	δ, °
U 67	169	1,830	73.9
	147	6,370	71.2
	126	29,400	68.6
W 70	169	1,830	73.9
	147	6,370	71.2
	126	29,400	68.6
U 76	169	5,020	73.9
	147	16,000	71.2
	126	58,900	68.6

Table 3. 6 Job Mix Formula for Type D and CMHB-C Mix Designs (Sugandhet.al., 2007)

Binder Grade	PG67-22, PG70-22, PG76-22	
Mix Type	Type D	CMHB-C
Binder Content,%	4.5	4.9
Sieve Size	Percent Passing	
3/4	100.0	100.0
1/2	100.0	100.0
3/8	97.0	61.0
No. 4	65.0	34.8
No. 10	35.5	20.4
No. 40	17.1	11.6
No. 80	6.6	8.8
No. 200	2.6	7.0
Maximum Specific Gravity	2.550	2.423
Aggregate Bulk Specific Gravity	2.655	2.591
Air Voids	4.0	4.0
VMA	14.3	17.6
VFA	71.9	77.3
V_{eff} %	3.6	3.7

Table 3. 7 Dynamic Modulus Values for Type D and CMHB-C Mixes (Sugandh et.al., 2007)

Mix Type	Temperature °F	Dynamic Modulus | E*|, (psi) Test Frequency, Hz 1	2	5	10
CMHB-C for PG Grade 70 -22	10	2,218,300	2,477,000	2,795,000	3,097,000
	40	1,413,400	1,628,367	1,932,333	2,154,133
	70	522,000	639,000	838,000	994,000
	100	221,000	265,000	324,000	396,000
	130	136,000	145,000	174,000	197,000
Type D for PG Grade 70 -22	10	2,690,000	3,018,000	3,394,000	3,746,000
	40	1,723,000	1,975,000	2,298,000	2,581,000
	70	815,000	947,000	1,173,000	1,333,000
	100	318,000	363,000	465,000	549,000
	130	192,000	208,000	250,000	276,000

3.6.5 REMAINING INPUTS

The other inputs required for the analysis were not changed from default values. To perform analysis, the New_HMA.dgp file was renamed for each of the sensitivity analysis and only values mentioned in the previous sections were changed. The results of the sensitivity analysis are presented in the next chapter.

Chapter 4

RESULTS

4.1 INTRODUCTION

In this chapter, the results of the sensitivity analysis are summarized and discussed. The main focus of the analysis was to identify influence of binder type on performance, influence of environmental condition on performance and influence of mix type on performance. The pavement structure of Table3.3 was used for the analysis with AADTT of 10,000. The results of the sensitivity analysis are discussed in the following sections.

4.2 INFLUENCE OF BINDR TYPE ON PERFORMANCE

To identify influence of binder type on performance, three binder types U67, W70, and U 76 (Table 3.4) were selected. The U67 and U76 asphalt binder was from different source than W70 asphalt. The mix type selected for analysis was Type D and dynamic modulus tests performed by Sugandh et al. (2007) were used as an input in the program. The El Paso weather station data was used for the analysis. The AADTT of 10,000, design life of 10 years, and initial IRI of 63 in./mile were selected for the analysis.

The predicted distresses, for the analyzed conditions, are summarized in Table 4.1. Three predicted distresses namely: IRI, permanent deformation in AC layer, and the total permanent deformation predicted are summarized in the table because change in binder type influences rutting in the pavement. Overall, the predicted IRI varied from 81 to 244 in./mile. The change in IRI was most significant in the presence of poor quality base and subgrade material.

Table 4. 1 Influence of Binder Type on Pavement Performance

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172			AC Perm. Def., in Distress Target 0.25			Total Perm. Def., in Distress Target 0.75		
				U 67	W 70	U 76	U 67	W 70	U 76	U 67	W 70	U 76
6	6	5	1	204	209	224	0.26	0.3	0.37	*2.73*	*2.37*	*2.95*
6	6	5	40	95	98	101	0.30	0.31	0.40	0.55	0.54	0.66
6	6	171	1	152	159	157	0.21	0.20	0.32	*2.03*	*2.19*	*2.18*
6	6	171	40	89	88	93	0.37	0.31	0.48	0.47	0.43	0.59
6	10	5	1	195	244	212	0.25	0.27	0.37	*2.63*	*3.00*	*2.84*
6	10	5	40	98	102	104	0.29	0.26	0.39	0.59	0.57	0.71
6	10	171	1	138	141	182	0.26	0.24	0.36	*1.7*	*1.77*	*2.54*
6	10	171	40	89	88	97	0.38	0.32	0.45	0.48	0.43	0.62
6	14	5	1	195	230	202	0.25	0.27	0.37	*2.63*	*2.87*	*2.68*
6	14	5	40	100	104	107	0.28	0.25	0.38	0.63	0.61	0.75
6	14	171	1	127	131	133	0.31	0.27	0.45	*1.42*	*1.52*	*1.59*
6	14	171	40	89	88	94	0.39	0.32	0.51	0.48	0.43	0.6
6	18	5	1	181	218	194	0.25	0.27	0.37	*2.39*	*2.74*	*2.58*
6	18	5	40	102	107	109	0.28	0.25	0.37	0.66	0.64	*0.78*
6	18	171	1	121	123	128	0.34	0.30	0.50	*1.26*	*1.32*	*1.44*
6	18	171	40	89	88	94	0.40	0.33	0.52	0.48	0.43	0.61
8	6	5	1	165	156	175	0.16	0.19	0.25	2.23	1.90	2.42
8	6	5	40	89	89	93	0.26	0.27	0.35	0.46	0.45	0.56
8	6	171	1	142	147	147	0.21	0.19	0.32	1.79	1.89	1.93

Table 4.1 Influence of Binder Type on Pavement Performance (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172			AC Perm. Def., in Distress Target 0.25			Total Perm. Def., in Distress Target 0.75		
				U 67	W 70	U 76	U 67	W 70	U 76	U 67	W 70	U 76
8	6	171	40	87	88	91	0.33	0.31	0.44	0.43	0.43	0.53
8	10	5	40	89	90	94	0.25	0.21	0.33	0.48	0.46	0.58
8	10	171	1	128	132	157	0.26	0.23	0.28	*1.47*	*1.54*	*2.1*
8	10	171	40	87	86	91	0.34	0.29	0.38	0.43	0.39	0.53
8	14	5	1	159	176	166	0.17	0.16	0.26	*2.10*	*2.34*	*2.24*
8	14	5	40	91	92	95	0.24	0.21	0.32	0.50	0.48	0.61
8	14	171	1	121	125	128	0.30	0.26	0.42	*1.30*	*1.37*	*1.45*
8	14	171	40	87	88	91	0.35	0.32	0.40	0.43	0.43	0.54
8	18	5	1	154	173	163	0.18	0.17	0.27	*2.00*	*2.28*	*2.19*
8	18	5	40	92	93	96	0.24	0.2	0.32	0.53	0.51	0.63
8	18	171	1	116	119	122	0.33	0.28	0.46	*1.16*	*1.21*	*1.32*
8	18	171	40	88	86	92	0.36	0.30	0.46	0.44	0.39	0.55
10	6	5	1	171	152	153	0.19	0.17	0.20	*2.50*	*1.96*	*1.96*
10	6	5	40	86	85	88	0.24	0.24	0.30	0.45	0.39	0.46
10	6	171	1	132	134	135	0.20	0.23	0.26	*1.50*	*1.60*	*1.64*
10	6	171	40	85	84	87	0.29	0.24	0.34	0.38	0.34	0.43
10	10	5	1	138	147	148	0.21	0.18	0.39	*1.04*	*1.85*	*3.00*
10	10	5	40	87	87	89	0.23	0.19	0.28	0.42	0.40	0.47
10	10	171	1	122	125	125	0.24	0.21	0.30	*1.31*	*1.37*	*1.38*

Table 4.1 Influence of Binder Type on Pavement Performance (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172			AC Perm. Def., in Distress Target 0.25			Total Perm. Def., in Distress Target 0.75		
				U 67	W 70	U 76	U 67	W 70	U 76	U 67	W 70	U 76
10	10	171	40	85	84	87	0.30	0.25	0.35	0.38	0.34	0.43
10	14	5	1	130	146	147	0.21	0.18	0.21	*1.92*	*1.83*	*1.88*
10	14	5	40	87	88	90	0.23	0.24	0.27	0.44	0.46	0.50
10	14	171	1	116	120	135	0.27	0.23	0.26	*1.17*	*1.23*	*1.64*
10	14	171	40	85	84	87	0.30	0.25	0.36	0.38	0.34	0.43
10	18	5	1	134	139	145	0.22	0.16	0.22	1.55	1.75	1.85
10	18	5	40	88	91	90	0.22	0.23	0.26	0.46	0.52	0.52
10	18	171	1	111	113	114	0.29	0.32	0.36	1.05	1.08	1.12
10	18	171	40	85	85	87	0.31	0.32	0.36	0.38	0.39	0.43
12	6	5	1	134	125	138	0.13	0.15	0.17	1.59	1.37	1.69
12	6	5	40	83	82	85	0.20	0.20	0.23	0.33	0.32	0.37
12	6	171	1	123	124	125	0.17	0.4	0.21	1.33	1.35	1.38
12	6	171	40	82	82	83	0.23	0.19	0.26	0.30	0.28	0.34
12	10	5	1	139	135	139	0.18	0.15	0.18	1.69	1.61	1.69
12	10	5	40	84	84	85	0.19	0.16	0.23	0.35	0.36	0.39
12	10	171	1	116	119	118	0.19	0.17	0.24	1.16	1.20	1.21
12	10	171	40	82	82	83	0.23	0.19	0.27	0.30	0.28	0.34
12	14	5	1	126	135	136	0.18	0.16	0.18	1.39	1.76	1.65
12	14	5	40	84	85	86	0.19	0.16	0.22	0.37	0.35	0.41

Table 4.1 Influence of Binder Type on Pavement Performance (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172			AC Perm. Def., in Distress Target 0.25			Total Perm. Def., in Distress Target 0.75		
				U 67	**W 70**	**U 76**	**U 67**	**W 70**	**U 76**	**U 67**	**W 70**	**U 76**
12	14	171	1	111	112	125	0.21	0.24	0.21	*1.03*	*1.05*	*1.38*
12	14	171	40	82	84	83	0.23	0.25	0.29	0.30	0.34	0.36
12	18	5	1	125	133	137	0.19	0.16	0.19	*1.36*	*1.56*	*1.64*
12	18	5	40	86	86	87	0.18	0.19	0.22	0.39	0.34	0.43
12	18	171	1	106	108	108	0.23	0.20	0.28	*0.91*	*0.85*	*0.97*
12	18	171	40	82	81	83	0.24	0.20	0.27	0.3	0.28	0.34

For instance, the predicted IRI varied between 204 and 224 for all three binder types and for AC layer thickness of 6 in., base layer thickness of 6 in. (modulus of 5 ksi), and subgrade modulus of 1 ksi. On the other hand, increase in just subgrade modulus to 40 ksi resulted in predicted IRI between 95 and 101. The predicted IRI suggests that increase in subgrade modulus increases life of pavement in comparison to binder type. The predicted data also suggests that modulus of base and subgrade significantly influences IRI in comparison to layer thickness. For instance, the predicted IRI varied between 195 and 244 for all binder type and for AC layer thickness of 6 in., base layer thickness of 10 in.(modulus o 5 ksi), and subgrade modulus of 1 ksi. The increase in AC layer thickness to 12 in. and base layer to 18 in. only resulted in decreasing IRI values to 125 which is higher than 101 predicted previously indicating quality of subgrade is more important for IRI rather than the thickness or binder type.

The influence of binder type on permanent deformation of AC layer was mostly insignificant. The permanent deformation for three binder types and all of the pavement structure varied between 0.12 and 0.52. Overall, the maximum AC layer permanent deformation for three binder types and similar structure varied between 0.3 and 0.5 inches. The higher difference between binder types was also prominent in lower strength subgrade materials. Also, the data suggested that the AC layer permanent deformation is more for higher PG grade than lower PG grade indicating that the dynamic modulus values may not be very well correlated to distress models within the program. Another aspect observed from the data is that W70 binder exhibited lower AC deformation under higher strength subgrade material in comparison to U 67 or U76 binder type indicating that W70 binder type may be of better quality than U67 or U76. Also data suggests that influence of binder type can be better identified in the presence of strong

pavement support structure rather than poor pavement support structure. Similar trends were observed for total permanent deformation predicted using similar pavement structures.

4.2 INFLUENCE OF MIX TYPES AND ENVIRONMANTEL CONDITIONS

To identify influence of mix types on performance, two mix types Type D and CMHB-C commonly used in the state of Texas were selected. The volumetric mix design information is included in Table 3.6 and laboratory measured dynamic modulus is included in Table 3.7. The binder used for preparing mixes was W70 consisting of 3% SBS. Two weather stations Miami and Houston were selected for analysis to simulate weather conditions experienced in India. The AADTT of 10,000, design life of 10 years, and initial IRI of 63 in./mile were selected for the analysis. The pavement structure of Table 3.3 was used for the evaluation. The remaining inputs were not altered from New_HMA.dgp file.

The results of the analysis are summarized in Tables 4.2 and 4.3. Table 4.2 summarizes predicted IRI and Longitudinal cracking while Table 4.3 summarizes predicted permanent deformation in the AC as well as total pavement. For all mix types, pavement structure, and weather stations, the IRI value varied between 80 and 335. Again, higher IRI values obtained for lower strength base and subgrade materials. In terms of mixes, the predicted IRI of Type D mix was lower in comparison to CMHB-C mixes. In addition, the predicted IRI was higher for lower strength subgrade and higher strength base than for higher strength subgrade and lower strength base suggesting that the subgrade strength significantly influences IRI. In general, IRI values were within acceptable limit of 172 in./mile for base modulus higher than 5 ksi and subgrade modulus higher than 25 ksi (based on interpolation). Although not very significant, it was observed that higher IRI values were predicted for Miami in comparison to Houston for

Table 4. 2 Terminal IRI and Longitudinal Cracking for Different Mix Types and Weather Conditions

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172				Longitudinal Cracking, ft/mi Distress Target: 2000			
				HC*	HD+	MC#	MD&	HC	HD	MC	MD
6	6	5	1	*307*	*259*	*335*	*276*	138	259	335	276
6	6	5	40	108	99	110	100	*10300*	*9980*	*10300*	*10100*
6	6	171	1	168	168	169	163	0	0	0	0
6	6	171	40	93	88	93	88	214	185	214	201
6	10	5	1	*284*	*245*	*306*	*259*	45	7	44	6
6	10	5	40	113	102	115	103	*10400*	*10200*	*10400*	*10200*
6	10	171	1	150	144	150	144	0	0	0	0
6	10	171	40	93	88	93	88	15	15	13	14
6	14	5	1	*264*	*231*	*283*	*243*	27	3	18	3
6	14	5	40	116	105	119	106	*10400*	*10200*	*10400*	*10300*
6	14	171	1	138	132	138	132	0	0	0	0
6	14	171	40	93	88	93	88	5	2	3	2
6	18	5	1	*248*	*219*	*264*	*229*	79	3	31	3
6	18	5	40	119	107	122	89	*10400*	*10200*	*10400*	*3660*
6	18	171	1	130	123	130	124	0	0	0	0
6	18	171	40	93	88	93	88	8	1	4	1
8	6	5	1	208	*189*	*215*	193	0	0	0	0
8	6	5	40	95	90	95	89	*8130*	*5550*	*8540*	6170
8	6	171	1	155	149	155	149	0	0	0	0

HC* Houston CMHB-C; HD+ Houston Type D; MC# Miami CMHB-C; MD& Miami Type D

Table 4.2 Terminal IRI and Longitudinal Cracking for Different Mix Types and Weather Conditions (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172				Longitudinal Cracking, ft/mi Distress Target: 2000			
				HC	HD	MC	MD	HC	HD	MC	MD
8	6	171	40	91	87	90	86	40	27	48	35
8	10	5	1	198	*182*	*205*	*185*	0	0	0	0
8	10	5	40	96	91	96	91	*8500*	*5740*	*8810*	*6270*
8	10	171	1	136	135	140	135	0	0	0	0
8	10	171	40	91	87	90	86	7	3	5	3
8	14	5	1	193	*177*	*198*	*181*	0	0	0	0
8	14	5	40	98	92	98	92	*8400*	*5320*	*8690*	*5810*
8	14	171	1	132	126	132	126	0	0	0	0
8	14	171	40	91	87	91	86	10	1	6	1
8	18	5	1	*187*	*173*	*192*	*176*	0	0	0	0
8	18	5	40	99	93	99,2	93	*8090*	*4650*	*8380*	*5100*
8	18	171	1	125	119	125	119	0	0	0	0
8	18	171	40	99	87	91	86	19	2	12	1
10	6	5	1	172	160	175	162	0	0	0	0
10	6	5	40	90	86	90	86	1590	363	1900	400
10	6	171	1	144	139	144	139	0	0	0	0
10	6	171	40	88	85	88	84	7	2	6	1
10	10	5	1	167	156	170	152	0	0	0	0
10	10	5	40	91	87	91	87	1490	285	1710	347

Table 4.2 Terminal IRI and Longitudinal Cracking for Different Mix Types and Weather Conditions (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172				Longitudinal Cracking, ft/mi Distress Target: 2000			
				HC	HD	MC	MD	HC	HD	MC	MD
10	10	171	1	133	127	132	127	0	0	0	0
10	10	171	40	88	85	88	84	12	1	7	0
10	14	5	1	164	154	167	153	0	0	0	0
10	14	5	40	92	88	92	88	1140	184	1280	184
10	14	171	1	125	120	125	119	0	0	0	0
10	14	171	40	88	85	88	84	26	2	17	0
10	18	5	1	162	152	164	154	0	0	0	0
10	18	5	40	93	89	93	88	813	113	893	131
10	18	171	1	120	114	119	114	0	0	0	0
10	18	171	40	88	85	84	84	46	4	54	1
12	6	5	40	86	84	86	83	60	7	70	3
12	6	171	1	133	128	114	128	0	0	0	0
12	6	171	40	85	82	84	81	8	1	5	0
12	10	5	1	151	142	153	143	0	0	0	0
12	10	5	40	87	85	87	84	41	4	45	2
12	10	171	1	125	121	125	120	0	0	0	0
12	10	171	40	85	82	84	81	19	2	14	0
12	14	5	1	149	141	122	142	0	0	0	0
12	14	5	40	88	85	80	85	24	2	0	1

Table 4.2 Terminal IRI and Longitudinal Cracking for Different Mix Types and Weather Conditions (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	Terminal IRI, in/mi, Distress Target 172				Longitudinal Cracking, ft/mi Distress Target: 1710			
				HC	HD	MC	MD	HC	HD	MC	MD
12	14	171	1	119	114	118	114	0	0	0	0
12	14	171	40	85	82	84	81	39	3	30	1
12	18	5	1	147	135	149	140	0	0	0	0
12	18	5	40	89	86	89	85	14	1	15	1
12	18	171	1	113	109	113	109	1	0	1	0
12	18	171	40	85	82	84	82	63	7	54	6

Table 4. 3 AC Layer and Total Pavement Deformation for Different Mix Types and Weather Conditions

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	AC Perm. Def., in Distress Target 0.25				Total Perm. Def., in Distress Target 0.75			
				HC	HD	MC	MD	HC	HD	MC	MD
6	6	5	1	0.39	0.28	0.43	0.30	3.36	3.09	3.45	3.17
6	6	5	40	0.36	0.26	0.38	0.28	0.65	0.53	0.66	0.54
6	6	171	1	0.29	0.20	0.30	0.21	2.40	2.25	2.44	2.29
6	6	171	40	0.41	0.30	0.40	0.32	0.54	0.43	0.54	0.44
6	10	5	1	0.38	0.27	0.42	0.30	3.26	3.01	3.33	3.07
6	10	5	40	0.35	0.26	0.37	0.27	0.70	0.57	0.71	0.59
6	10	171	1	0.34	0.23	0.35	0.24	1.98	1.82	2.00	1.84
6	10	171	40	0.43	0.32	0.44	0.33	0.55	0.43	0.56	0.44
6	14	5	1	0.38	0.27	0.41	0.29	3.10	2.87	3.17	2.94
6	14	5	40	0.34	0.25	0.36	0.27	0.73	0.61	0.75	0.63
6	14	171	1	0.39	0.27	0.40	0.28	1.68	1.52	1.70	1.54
6	14	171	40	0.44	0.32	0.45	0.34	0.55	0.43	0.57	0.45
6	18	5	1	0.38	0.27	0.41	0.29	2.95	2.74	3.02	2.80
6	18	5	40	0.34	0.25	0.36	0.25	0.77	0.65	0.79	0.63
6	18	171	1	0.43	0.30	0.44	0.31	1.49	1.32	1.51	1.34
6	18	171	40	0.45	0.33	0.46	0.34	0.55	0.43	0.57	0.45
8	6	5	1	0.24	0.16	0.25	0.17	2.75	2.51	2.80	2.55
8	6	5	40	0.31	0.22	0.32	0.23	0.53	0.43	0.55	0.44
8	6	171	1	0.27	0.18	0.28	0.19	2.09	1.94	2.12	1.96

Table 4.3 AC Layer and Total Pavement Deformation for Different Mix Types and Weather Conditions (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	AC Perm. Def., in Distress Target 0.25				Total Perm. Def., in Distress Target 0.75			
				HC	HD	MC	MD	HC	HD	MC	MD
8	6	171	40	*0.38*	*0.28*	*0.39*	*0.29*	0.49	0.39	0.50	0.40
8	10	5	1	0.24	0.16	*0.26*	0.17	*2.65*	*2.40*	*2.67*	*2.44*
8	10	5	40	*0.29*	0.21	*0.31*	0.22	0.56	0.46	0.57	0.47
8	10	171	1	*0.29*	0.22	*0.33*	0.23	*1.67*	*1.59*	*1.75*	*1.61*
8	10	171	40	*0.39*	*0.29*	*0.40*	0.30	0.49	0.39	0.51	0.40
8	14	5	1	0.25	0.16	*0.26*	0.17	*2.56*	*2.35*	*2.61*	*2.39*
8	14	5	40	*0.29*	0.21	*0.30*	0.22	0.59	0.48	0.60	0.50
8	14	171	1	*0.37*	*0.26*	*0.37*	*0.26*	*1.52*	*1.37*	*1.54*	*1.39*
8	14	171	40	*0.40*	*0.29*	*0.41*	*0.30*	0.50	0.39	0.51	0.40
8	18	5	1	0.25	0.17	0.27	0.18	*2.48*	*2.28*	*2.54*	*2.33*
8	18	5	40	*0.29*	0.20	*0.29*	0.21	0.61	0.51	0.63	0.52
8	18	171	1	*0.37*	*0.28*	*0.41*	*0.29*	*1.36*	*1.21*	*1.38*	*1.22*
8	18	171	40	*0.40*	*0.30*	*0.41*	0.31	0.50	0.39	0.51	0.40
10	6	5	1	0.19	0.13	0.20	0.13	*2.31*	*2.09*	*2.36*	*2.13*
10	6	5	40	0.27	0.20	*0.28*	0.21	0.46	0.38	0.47	0.39
10	6	171	1	0.25	0.17	0.25	0.12	*1.83*	*1.68*	*1.85*	*1.70*
10	6	171	40	*0.33*	0.24	*0.33*	0.25	0.50	0.34	0.44	0.35
10	10	5	1	0.20	0.13	0.21	0.14	*2.31*	*2.01*	*2.27*	*2.06*
10	10	5	40	*0.27*	0.19	*0.27*	0.20	0.46	0.40	0.49	0.41

Table 4.3 AC Layer and Total Pavement Deformation for Different Mix Types and Weather Conditions (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	AC Perm. Def., in Distress Target 0.25				Total Perm. Def., in Distress Target 0.75			
				HC	HD	MC	MD	HC	HD	MC	MD
10	10	171	1	*0.29*	0.20	*0.29*	0.21	*1.83*	*1.41*	*1.56*	*1.43*
10	10	171	40	*0.33*	0.25	*0.34*	*0.26*	0.43	0.34	0.44	0.35
10	14	5	1	0.21	0.14	0.21	0.14	*2.18*	*1.98*	*2.23*	*2.02*
10	14	5	40	*0.26*	0.19	*0.27*	0.19	0.48	0.42	0.52	0.42
10	14	171	1	*0.33*	0.23	0.33	0.23	*1.55*	*1.23*	*1.38*	*1.25*
10	14	171	40	*0.34*	0.25	*0.35*	*0.26*	0.43	0.34	0.44	0.35
10	18	5	1	0.21	0.14	0.22	0.15	*2.18*	*1.95*	*2.19*	*1.49*
10	18	5	40	0.25	0.18	*0.26*	0.19	0.53	0.44	0.54	0.45
10	18	171	1	*0.35*	0.25	*0.36*	*0.26*	*1.22*	*1.08*	*1.24*	*1.10*
10	18	171	40	*0.34*	0.26	*0.27*	*0.27*	0.43	0.34	0.35	0.35
12	6	5	40	0.22	0.16	0.23	0.17	0.38	0.31	0.39	0.32
12	6	171	1	0.21	0.14	0.21	0.15	*1.55*	*1.43*	*1.11*	*1.45*
12	6	171	40	0.25	0.19	*0.26*	0.19	0.34	0.28	0.35	0.28
12	10	5	1	0.17	0.11	0.17	0.12	*1.91*	*1.73*	*1.96*	*1.76*
12	10	5	40	0.21	0.16	0.22	0.16	0.40	0.34	0.41	0.34
12	10	171	1	0.24	0.17	0.24	0.17	*1.35*	*1.24*	*1.37*	*1.26*
12	10	171	40	*0.26*	0.19	0.26	0.20	0.34	0.28	0.35	0.28
12	14	5	1	0.17	0.12	0.17	0.12	*1.28*	*1.71*	*1.28*	*1.75*
12	14	5	40	0.21	0.16	0.21	0.16	0.23	0.36	0.24	0.36

Table 4.3 AC Layer and Total Pavement Deformation for Different Mix Types and Weather Conditions (Continued)

Asphalt Thickness (in)	Base Thickness (in)	Base Modulus (ksi)	Subgrade Modulus (ksi)	AC Perm. Def., in Distress Target 0.25				Total Perm. Def., in Distress Target 0.75			
				HC	HD	MC	MD	HC	HD	MC	MD
12	14	171	1	*0.26*	0.18	0.26	0.19	*1.20*	*1.08*	*1.21*	*1.10*
12	14	171	40	*0.26*	0.19	*0.27*	0.20	0.34	0.28	0.35	0.28
12	18	5	1	0.18	0.12	0.18	0.11	*1.84*	*1.62*	*1.89*	*1.71*
12	18	5	40	0.21	0.15	0.21	0.16	0.44	0.37	0.45	0.38
12	18	171	1	*0.28*	0.20	*0.28*	0.20	*1.06*	*0.95*	*1.08*	*0.96*
12	18	171	40	*0.26*	0.20	*0.27*	*0.20*	0.34	0.28	0.35	0.28

CMHB-C mixes indicating that harsher weather conditions observed at Miami will reduce life of pavements consisting of CMHB-C mixes. While variation of IRI values for pavements consisting of Type D mixes varied slightly indicating that CMHB-C mixes are more susceptible to changes in weather conditions in comparison to Type D mixes.

For all mix types, pavement structure, and weather stations, the predicted longitudinal cracking varied from 10,400 to 0 ft/mile. Overall, predicted longitudinal cracking was higher for lower AC and base thicknesses and higher base and subgrade modulus values. For instance, longitudinal cracking of more than 10,000 ft/mile was predicted for AC and base thickness of 6 in. and base modulus of 5 ksi and subgrade modulus of 40 ksi. The predicted longitudinal cracking reduced to less than 400 ft/mile for the subgrade modulus of 1 ksi while other properties remained unchanged. As thicknesses of AC and base layers increased this phenomenon of very high longitudinal cracking dissipated. The longitudinal cracking of less than 2,000 ft/mile was observed for AC thickness of 10 in. or greater and base thickness of 6 in. or greater indicating that longitudinal cracking can be reduced by increasing AC layer thickness.

In terms of mix types, Type D mix resisted longitudinal cracking better than CMHB-C mix type especially for pavements with 8 in. or more AC layer thickness. For AC layer thicknesses of 6 in., occasionally CMHB-C mix better resisted longitudinal cracking than Type D mix. Overall, predicted longitudinal cracking was significantly higher for base modulus of 5 ksi and subgrade modulus of 40 ksi and in those instances Type D mix resisted longitudinal cracking better than CMHB-C mixes.

In terms of weather conditions, the predicted longitudinal cracking was no significantly different between Miami and Houston. There were few instances where pavements located in Miami exhibited higher longitudinal cracking but the differences were minimal.

The AC layer and total pavement permanent deformation data is summarized in Table 4.3 and results indicate that stronger subgrade is important for longer performing pavements. The AC layer permanent deformation varied between 0.11 and 0.45 for all pavement structures and weather conditions. The AC permanent deformation was lower for higher subgrade modulus for other conditions remaining same. For AC and base layer thickness of 10 in. and base of modulus of 171 ksi, AC permanent deformation of 0.29 in. was predicted for subgrade modulus of 1 ksi while 0.33 in. was predicted for subgrade modulus of 40 ksi (for CMHB-C mix and Houston weather station). Although less prominent, the AC layer permanent deformation was lower for lower subgrade modulus for higher AC and base layer thicknesses.

In terms of mix type, Type D mix resisted rutting more in comparison to CMHB-C mix type. For above example, the predicted AC permanent deformation was 0.2 and 0.25 in. for Type D mix which was lower than 0.29 and 0.33 in. observed with CMHB-C mix type. The influence of weather condition was insignificant.

In terms of total permanent deformation, the trend was reversed as observed for AC layer permanent deformation. In other words, an increase in subgrade modulus reduced total permanent deformation. For above example, total permanent deformation of 1.83 in. was predicted for subgrade modulus of 1 ksi while only 0.43 in. was predicted for subgrade modulus of 40 ksi. An increase in AC layer of up to 12 in. and base layer of up to 18 in., the total permanent deformation was above target limit of 0.75 in. (for subgrade modulus of 1 ksi)

indicating that the subgrade modulus significantly influences permanent deformation. Since total permanent deformation includes AC layer deformation, the trends similar to the AC layer permanent deformation were observed both in terms of mix types as well as weather conditions. A comparison between weak and strong pavement structure is summarized in the following paragraphs.

Figures 4.1 through 4.6 show predicted Terminal IRI, permanent deformation, and longitudinal cracking for weak and strong pavement structure. The data in Figures 4.1, 4.3, and 4.5 is for the following weak pavement structure: a) AC layer thickness of 6 in., b) base layer thickness of 6 in., c) base modulus of 5 ksi, and subgrade modulus of 1 ksi. While the data in Figures 4.2, 4.4, and 4.6 is for the following strong pavement structure: a) AC layer thickness of 12 in., b) base layer thickness of 18 in., c) base modulus of 171 ksi, and d) subgrade modulus of 171 ksi. The results are shown for mix types Type D and CMHB-C mixed with W70 binder and for Houston and Miami weather conditions. Figure 4.1 data suggests that significantly higher Terminal IRI were predicted with weak pavement structure (250 in./mile or higher) in comparison to strong pavement structure (less than 90). In terms of influence of mix types, the predicted Terminal IRI was lower for Type D mix for weak pavement structure while higher for the strong pavement structure in comparison to CMHB-C mix type. However, the influence of mix type dissipates for stronger pavement structure (overall Terminal IRI varied from 77 to 85 in./mile).

The permanent deformation data is summarized in Figure 4.3 and 4.4. The total permanent deformation of weaker pavement structure was significantly higher (between 3 and 3.5 in.) than for strong pavement structure (between 0.35 and 0.15 in.).

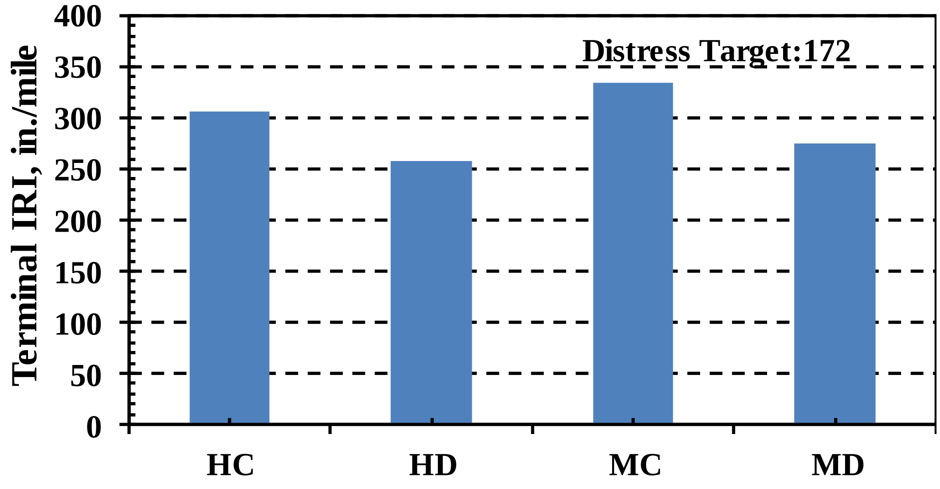


Figure 4.1 Predicted Terminal IRI of Weak Pavement Structure

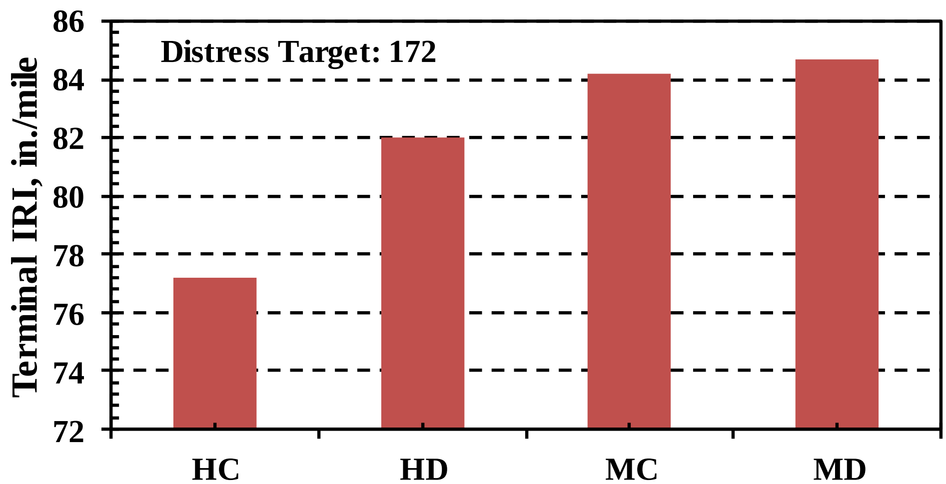


Figure 4.2 Predicted Terminal IRI of Strong Pavement Structure

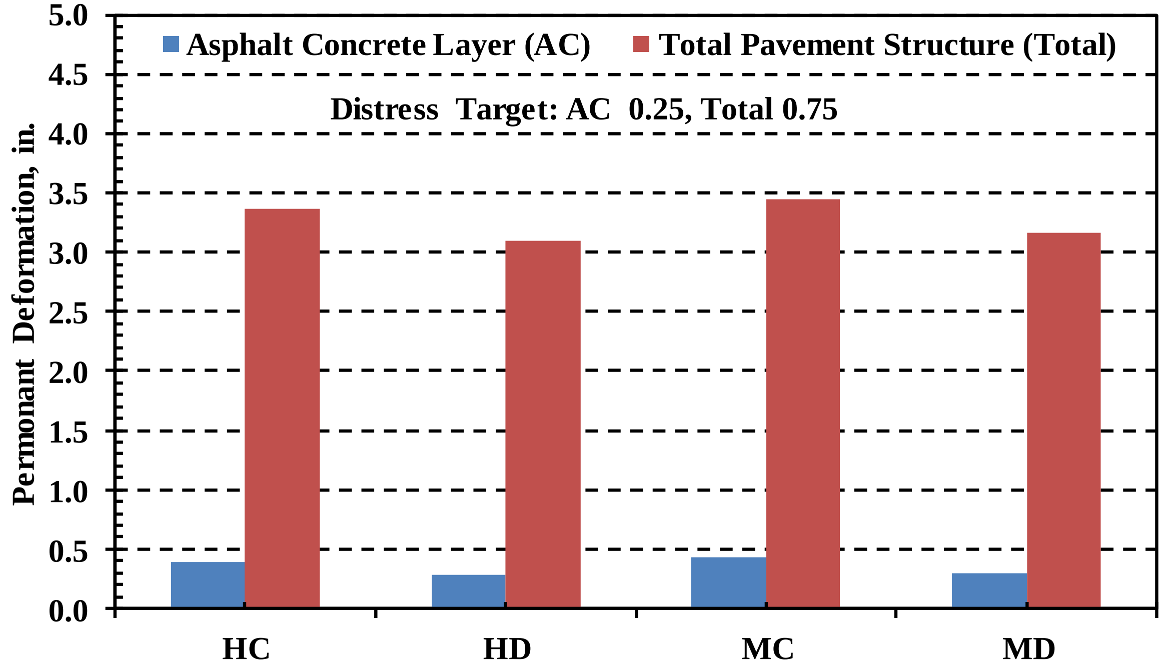


Figure 4.3 Predicted Permanent Deformation of Weak Pavement Structure

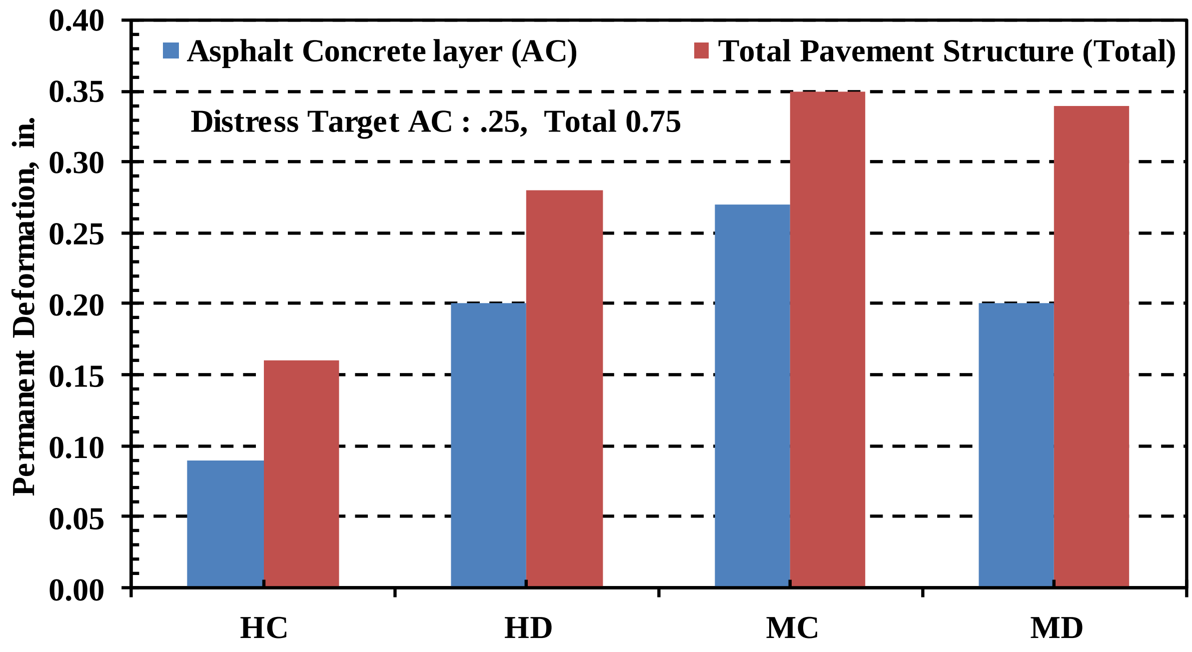


Figure 4.4 Predicted Permanent Deformation of Strong Pavement Structure

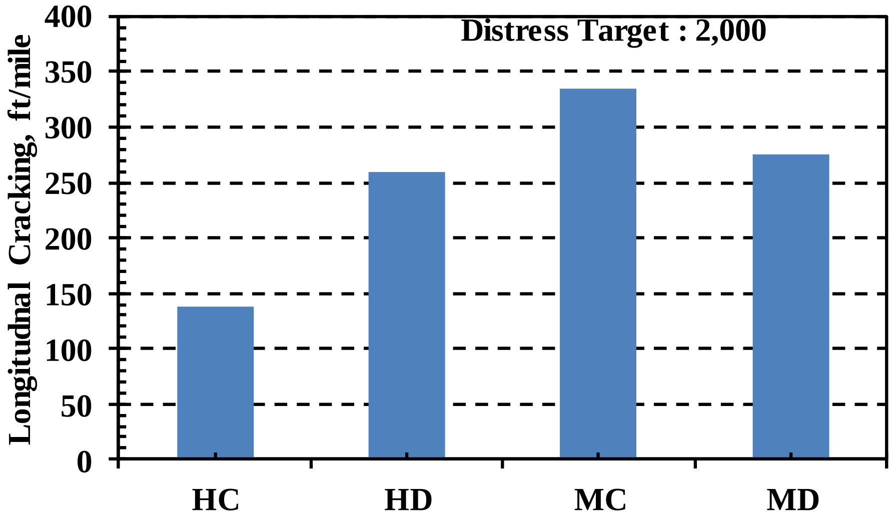


Figure 4.5 Predicted Longitudinal Cracking of Weak Pavement Structure

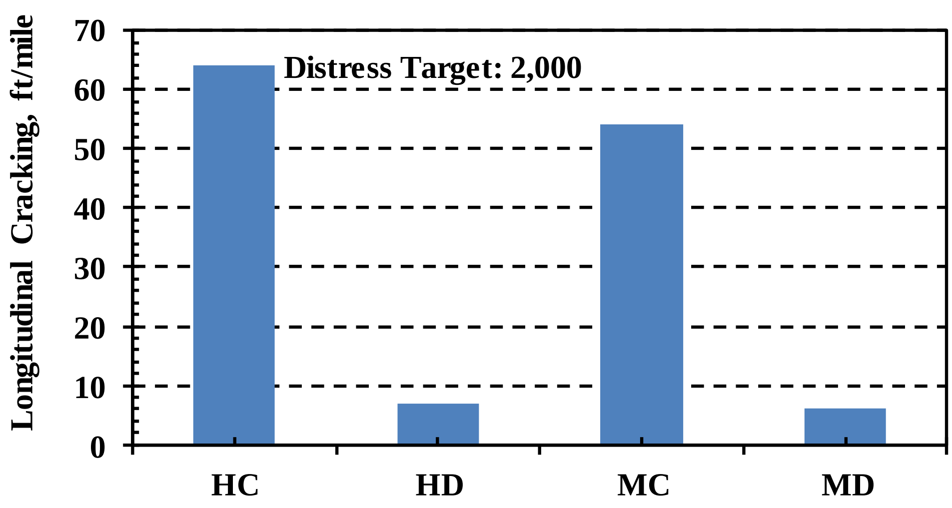


Figure 4.6 Predicted Longitudinal Cracking of Strong Pavement Structure

Similar to Terminal IRI, Type D mix exhibited more resistance to permanent deformation for weak pavement structure and vice versa. In addition, the influence of weather station was less for weaker pavement structure in comparison to strong pavement structure in terms of total as well AC layer permanent deformation.

The longitudinal cracking data is summarized in Figures 4.5 and 4.6. The predicted longitudinal cracking was significantly higher for weaker pavement structure (138 ft/mile or more) than strong pavement structure (66 ft/mile or less). The estimated longitudinal cracking was significantly higher for CMHB-C mix type in comparison to Type D mix for strong pavement structure (Figure 4.6). However, similar trend was not identified for weak pavement structure (Figure 4.5). For weaker pavement structure, higher longitudinal cracking was estimated for Miami in comparison to Houston. On the other hand, lower longitudinal cracking was estimated for Miami in comparison to Houston for stronger pavement structure.

Overall, the analysis suggests that stronger pavement especially subgrade modulus is essential for achieving design life. In addition, Type D (finer mix) is better in terms of resisting cracking or permanent deformation in comparison to CMHB-C mix (coarser mix) type. The influence of weather (for conditions evaluated) was minimal. The influence of binder type was opposite to expected performance. In other words, the predicted permanent deformation in AC layer was higher for higher PG grade while higher PG grade binder is expected to reduce permanent deformation.

Chapter 5

CASE STUDY

In the end, a case study was performed to identify adequacy of existing IRC 37-2001 design guide by predicting performance of pavement using MEPDG. To achieve this objective, traffic and material information from one of the Indian highways was obtained to design pavement structure as per IRC37-2001. The identified pavement structure together with traffic and material information was entered in MEPDG software to predict performance of the highway. The results of the evaluation are presented in this chapter.

5.1 IRC INPUT

For this case study, the traffic data (Appendix A) was provided by Indian Institute of Technology-Madras (India) for a highway near Chitradurga and Hospet Tamilnadu (India). The three traffic levels were selected: 10, 104, and 150 msa. The 10 and 150 msa's were selected to identify adequacy of upper limits of the IRC design guide as shown in Figure 2.2. The 104 msa was selected based on the data obtained from the above mentioned site. Since MEPDG requires AADTT, the selected msa was converted to AADTT using equation 2.3. The parameters assumed or used for this conversion are included in Table 5.1 and AADTT is assumed to be equal to A. For the purpose of this study, design life of the pavement was assumed to be 10 years with linear traffic growth rate of 7.5%. The VDF was estimated to be 4.69 from the site data (Appendix B). The back calculation estimated A or AADTT to be 550, 5731, and 8259 for 10, 104, and 150 msa, respectively.

The IRC design guide uses traffic as well as CBR value of the base, subbase and subgrade layer material to identify adequate pavement structure (including individual layer thickness) as shown in Figure 2.2 and Table 2.2. Although design guide allows for CBR of 2 to 10%, the results presented in previous chapter identified that soft subgrade and base layers lead to pavement failures. Therefore, CBR value of 10% was selected for this study. Since design guide provides pavement structure for 100 or 150 msa, the pavement structure was interpolated for 104 msa. Since design guide suggests changing asphalt layer thickness for higher traffic loads (from 100 to 150 msa), only asphalt layer thickness was interpolated. The asphalt layer thickness was changed from 130 mm to 133 mm while base and subgrade layer thickness remained same. The identified pavement structure is summarized in Table 5.2.

Table 5. 1 Estimation of Initial Daily Traffic

$$\boldsymbol{N} = \frac{\boldsymbol{365} * [(\boldsymbol{1}+\boldsymbol{r})^{\boldsymbol{n}} - \boldsymbol{1}] * \boldsymbol{A} * \boldsymbol{D} * \boldsymbol{F}}{\boldsymbol{r}}$$	
N is the cumulative number of ESALs to be catered for in the design in terms of million ESALs.	10, 100 or 150 msa
A is initial traffic in the year of completion of construction in terms of the number of CVPD	Estimated from the equation
D is lane distribution factor,	4.5
F is vehicle damage factor	4.69
n is design life in years	10
r is annual linear growth rate of commercial	7.5%

Table 5. 2 IRC Predicted Pavement Structure for Different Traffic Conditions

Cumulative. Traffic, msa	California Bearing Ratio, %	Total Pavement Thickness, mm (in.)	Bituminous Surfacing		Granular Base, mm (in.)	Granular Subbase, mm (in.)
			Wearing Course, mm (in.)	Binder Course, mm (in.)		
10	10	540 (21.26)	40 (1.57)	50 (1.97)	250 (9.84)	200 (7.87)
100		630 (24.80)	50 (1.97)	133 (5.24)	250 (9.84)	200 (7.87)
150		650 (25.59)	50 (1.97)	150 (5.91)	250 (9.84)	200 (7.87)

5.2 MEPDG INPUT

The pavement material properties, traffic information, and environmental condition needed to perform MEPDG runs are discussed as follows:

- Since weather information is not available for the site, it was decided to use El Paso weather station data.
- Only, asphalt concrete mix gradation was available for this analysis; therefore, Level 3 input module was selected. The mix gradation used for the analyses is shown in Table 5.3. Since El Paso weather conditions were selected for the analyses, four PG grades were selected PG 82-22, PG 76-22, PG 70-22, and PG 64-22. The remaining asphalt concrete properties were kept similar to that of New_HMA.dgp file
- In addition to CBR value of 10, CBR values of 15, 20 and 25% were selected to identify benefits of strengthening foundation layers (base, subbase, and subgrade layers). Since IRC does not have a provision for more than 10% CBR, the pavement structure identified for 10% CBR value was used for remaining CBR values as shown in Table 5.4. The CBR value was converted to modulus as discussed in Chapter 4.

- For this study, three daily traffic levels (AADTT values of 550, 5731, and 8259) were selected. In addition, the vehicle class distribution, hourly truck traffic distribution, number of axles per truck and traffic growth factor were changed from the New_HMA.dgp file as shown in Figure 5.1. The changes were made based on the data obtained from Indian Institute of Technology-Madras (IIT-M)and relevant information is included in Appendix A and B. The raw data obtained from IIT-M is included in Appendix A while the data analysis to identify factors like VDF is included in Appendix B.
- The initial IRI value of 63 in./mi. was replaced with 100 in./mi. because it is difficult to obtain an IRI value of 63 at the end of the construction in India.

Table 5. 3 Asphalt Concrete Mix Gradation

Sieve Size	Percent Passing or Retained
Cumulative % Retained 3/4 inch sieve	18
Cumulative % Retained 3/8 inch sieve	35
Cumulative % Retained #4 sieve	45
% Passing #200 sieve	8

Table 5. 4 Pavement Structure and Material Properties Used for MEPDG Runs

California Bearing Ratio, %	Estimated Elastic Modulus from IRC-37, ksi	Binder Grade, PG	AADTT, number	Total Pavement Thickness, in.	Bituminous Layer, in.	Granular Base, in.	Granular Subbase, in.
10, 15, 20, and 25	15.6, 20.2, 24.3, and 28.0	64-22, 70-22, 76-22, and 82-22	550	21.26	3.54	9.84	7.87
			5,731	24.82	7.20	9.84	7.87
			8,259	25.59	7.87	9.84	7.87

Traffic

Initial two-way AADTT: 550
Number of lanes in design direction: 2
Percent of trucks in design direction (%): 50
Percent of trucks in design lane (%): 75
Operational speed (mph): 60

Traffic -- Volume Adjustment Factors

Monthly Adjustment Factors (Default value of 1.00)

Vehicle Class Distribution
(Level 3, Default Distribution)

AADTT distribution by vehicle class

Class 4	5.4%
Class 5	69.8%
Class 6	24.1%
Class 7	0.0%
Class 8	0.7%
Class 9	0.0%
Class 10	0.0%
Class 11	0.0%
Class 12	0.0%
Class 13	0.0%

Hourly truck traffic distribution
by period beginning:

Midnight	3.2%	Noon	3.6%
1:00 am	3.7%	1:00 pm	3.2%
2:00 am	3.7%	2:00 pm	4.1%
3:00 am	5.5%	3:00 pm	3.8%
4:00 am	4.8%	4:00 pm	3.9%
5:00 am	4.7%	5:00 pm	4.7%
6:00 am	5.0%	6:00 pm	4.7%
7:00 am	4.5%	7:00 pm	4.6%
8:00 am	5.5%	8:00 pm	3.2%
9:00 am	4.2%	9:00 pm	4.4%
10:00 am	3.8%	10:00 pm	3.7%
11:00 am	3.7%	11:00 pm	3.6%

Traffic Growth Factor (7.5% Linear Growth for all Class)

Traffic -- Axle Load Distribution Factors

Level 3: Default

Traffic -- General Traffic Inputs

Mean wheel location (inches from the lane marking): 18
Traffic wander standard deviation (in): 10
Design lane width (ft): 12

Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Figure 5. 1 Traffic Input for MEPDG Runs

5.3 MEPDG RUN RESULTS

The results of the runs are summarized in Tables 5.5 through 5.7 and Figures 5.2 through 5.10. The results presented in tables and figures suggest that designed pavement structure is adequate to transport daily truck traffic for 10 years. For instance, the traffic level of 10 msa for weakest structure only creates a total permanent deformation of 0.32 in. which is less than 0.75 in. as required by IRC code or MEPDG failure criterion. Similarly, the Terminal IRI value only increased by 20in./mile (initial IRI was 100 in./mile) indicating no premature failure of pavement. This trend was observed for higher msa's as well. In fact, the level of distress decreased with increase in msa (Figures 5.2 through 5.4). The reason for decrease can be attributed to increase in asphalt concrete layer thickness from 3.54 to 7.20 or higher because of IRC design code.

The influence of CBR value is shown in Figures 5.5 and 5.6 and the results suggest that increase in CBR value decreased the distress levels predicted. The total permanent deformation decreased from 0.24 to 0.16 (Figure 5.5) with increase in CBR from 10% to 25% when 150 msa of truck traffic passed through the pavement. The change in Terminal IRI was not as significant as the value only decreased from 118 to 114 (Figure 5.6) for the similar conditions.

The influence of PG grade is shown in Figures 5.7 through 5.10. The increase in higher temperature PG grade from 64-22 to 82-22 decreased terminal IRI (Figure 5.7) and longitudinal cracking (Figure 5.8) but there was no change in alligator cracking (Figure 5.9) and permanent deformation (Figure 5.10) for 10 msa and CBR value of 10%. Similar trends were observed for other CBR and traffic levels.

The runs indicate that IRC code is adequate for 10 year design life of pavements for traffic levels of 150 msa with adequate foundation (CBR value of 10% or more).

Table 5. 5 Predicted Pavement Performance for Traffic Loading of 10 msa

Asphalt Thickness, in	Base, Subbase and Subgrade Modulus, psi	AADTT	Distress Target :		172	2000	25	0.25	0.75
			PG Grade	CBR, %	Terminal IRI, in/mi	AC Surface Down Cracking (Long. Cracking), ft/mile	AC Bottom Up Cracking (Alligator Cracking), %	AC Permanent Deformation, in	Total Permanent Deformation, in
3.54	15596	550	64-22	10	122	15.90	0.10	0.01	0.32
3.54	15596	550	70-22	10	121	15.30	0.10	0.01	0.32
3.54	15596	550	76-22	10	121	14.70	0.10	0.01	0.32
3.54	15596	550	82-22	10	121	14.30	0.10	0.01	0.32
3.54	20217	550	64-22	15	119	10.00	0.10	0.01	0.27
3.54	20217	550	70-22	15	119	9.70	0.10	0.01	0.27
3.54	20217	550	76-22	15	119	9.50	0.10	0.01	0.26
3.54	20217	550	82-22	15	119	9.30	0.10	0.01	0.26
3.54	24303	550	64-22	20	118	6.60	0.10	0.01	0.23
3.54	24303	550	70-22	20	118	6.50	0.10	0.01	0.23
3.54	24303	550	76-22	20	118	6.40	0.10	0.01	0.23
3.54	24303	550	82-22	20	118	6.30	0.10	0.01	0.23
3.54	28034	550	64-22	25	117	4.60	0.10	0.01	0.21
3.54	28034	550	70-22	25	117	4.50	0.10	0.01	0.21
3.54	28034	550	76-22	25	117	4.50	0.10	0.01	0.21
3.54	28034	550	82-22	25	117	4.40	0.10	0.01	0.21

Table 5. 6 Predicted Pavement Performance for Traffic Loading of 104 msa

Asphalt Thickness, in	Base, Subbase and Subgrade Modulus, psi	AADTT	Distress Target :		172	2000	25	0.25	0.75
			PG Grade	CBR, %	Terminal IRI, in/mi	AC Surface Down Cracking (Long. Cracking), ft/mile	AC Bottom Up Cracking (Alligator Cracking), %	AC Permanent Deformation, in	Total Permanent Deformation, in
7.2	15596	5731	64-22	10	119	0.10	0.10	0.01	0.25
7.2	15596	5731	70-22	10	119	0.10	0.10	0.01	0.26
7.2	15596	5731	76-22	10	119	0.10	0.10	0.01	0.25
7.2	15596	5731	82-22	10	119	0.10	0.10	0.01	0.25
7.2	20217	5731	64-22	15	117	0.10	0.10	0.01	0.21
7.2	20217	5731	70-22	15	117	0.10	0.10	0.01	0.21
7.2	20217	5731	76-22	15	117	0.10	0.10	0.01	0.20
7.2	20217	5731	82-22	15	117	0.10	0.10	0.01	0.20
7.2	24303	5731	64-22	20	116	0.20	0.10	0.01	0.18
7.2	24303	5731	70-22	20	116	0.10	0.10	0.01	0.18
7.2	24303	5731	76-22	20	116	0.10	0.10	0.01	0.18
7.2	24303	5731	82-22	20	116	0.10	0.10	0.01	0.18
7.2	28034	5731	64-22	25	116	0.40	0.10	0.01	0.18
7.2	28034	5731	70-22	25	116	0.30	0.10	0.01	0.17
7.2	28034	5731	76-22	25	115	0.30	0.10	0.01	0.17
7.2	28034	5731	82-22	25	115	0.10	0.00	0.01	0.16

Table 5. 7 Predicted Pavement Performance for Traffic Loading of 150 msa

Asphalt Thickness, in	Base, Subbase and Subgrade Modulus, psi	AADTT	Distress Target :		172	2000	25	0.25	0.75
			PG Grade	CBR, %	Terminal IRI, in/mi	AC Surface Down Cracking (Long. Cracking), ft/mile	AC Bottom Up Cracking (Alligator Cracking), %	AC Permanent Deformation, in	Total Permanent Deformation, in
7.87	15596	8300	64-22	10	118	0.00	0.10	0.01	0.24
7.87	15596	8300	70-22	10	118	0.00	0.10	0.01	0.24
7.87	15596	8300	76-22	10	118	0.00	0.10	0.01	0.24
7.87	15596	8300	82-22	10	118	0.00	0.10	0.01	0.24
7.87	20217	8300	64-22	15	117	0.00	0.10	0.01	0.20
7.87	20217	8300	70-22	15	117	0.00	0.10	0.01	0.20
7.87	20217	8300	76-22	15	116	0.00	0.10	0.01	0.20
7.87	20217	8300	82-22	15	116	0.00	0.10	0.01	0.20
7.87	24303	8300	64-22	20	116	0.00	0.10	0.01	0.18
7.87	24303	8300	70-22	20	116	0.00	0.00	0.01	0.18
7.87	24303	8300	76-22	20	116	0.00	0.00	0.01	0.18
7.87	24303	8300	82-22	20	116	0.00	0.00	0.01	0.18
7.87	28034	8300	64-22	25	115	0.10	0.00	0.01	0.17
7.87	28034	8300	70-22	25	115	0.00	0.00	0.01	0.16
7.87	28034	8300	76-22	25	115	0.00	0.00	0.01	0.16
7.87	28034	8300	82-22	25	115	0.00	0.00	0.01	0.16

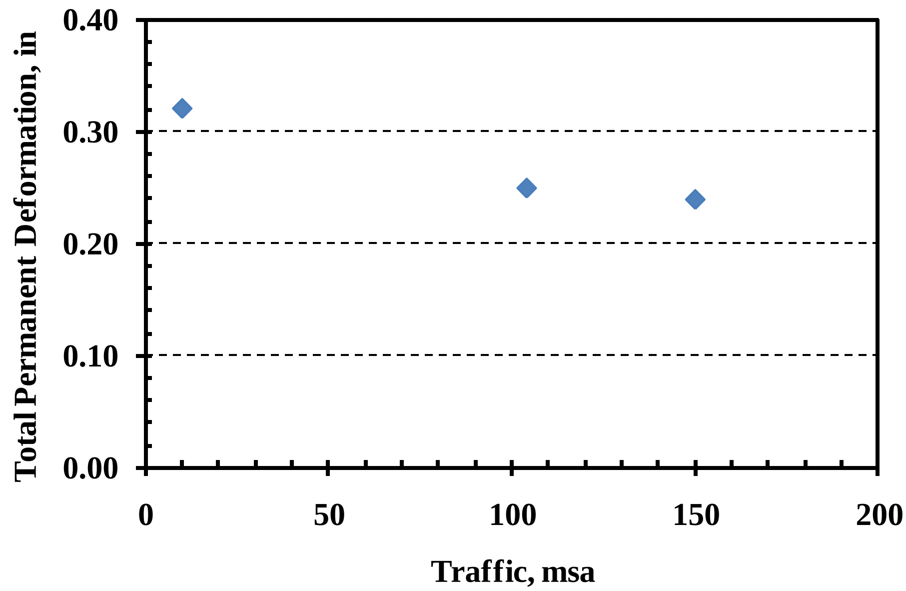


Figure 5. 2 Influence of AADTT on Total Permanent Deformation

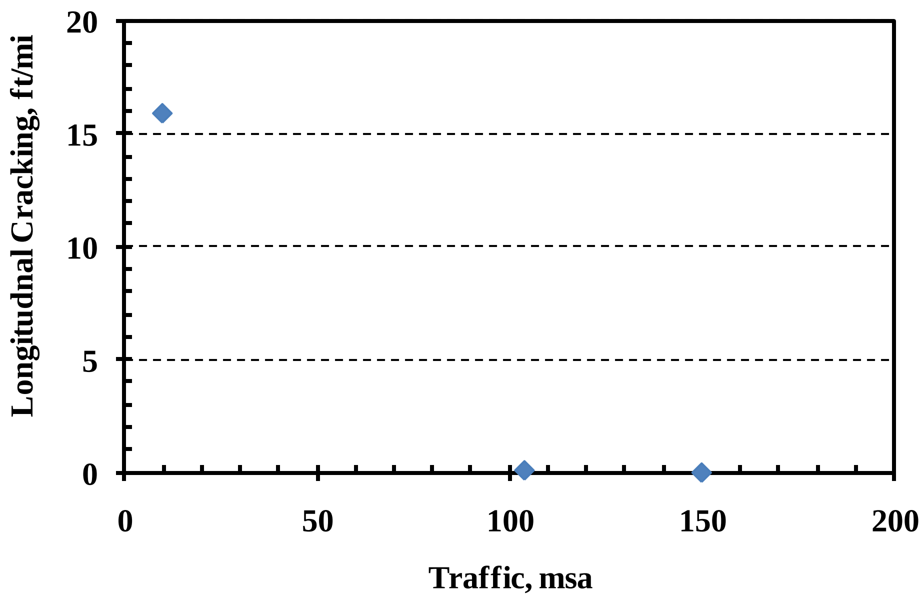


Figure 5. 3 Influence of AADTT on Longitudinal Cracking

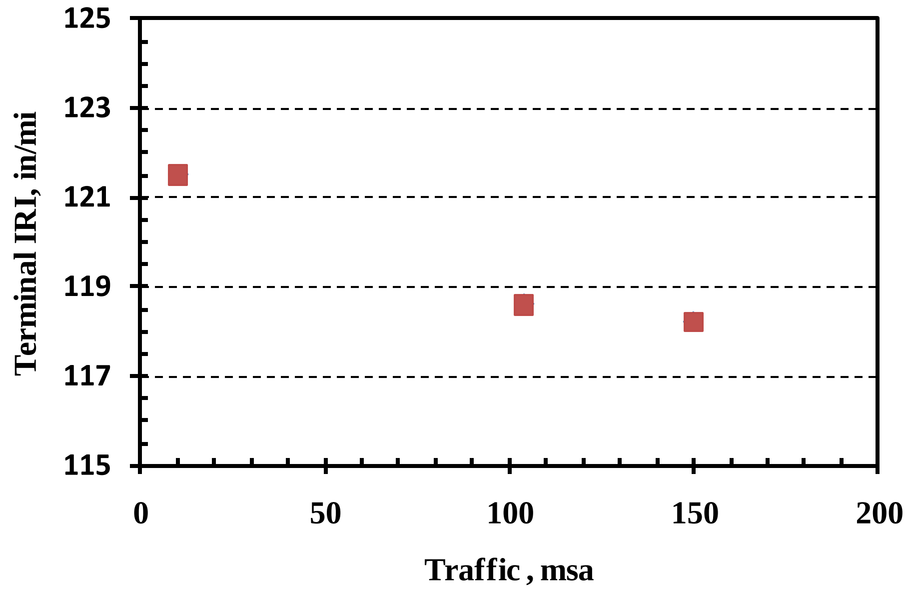


Figure 5. 4 Influence of AADTT on Terminal IRI

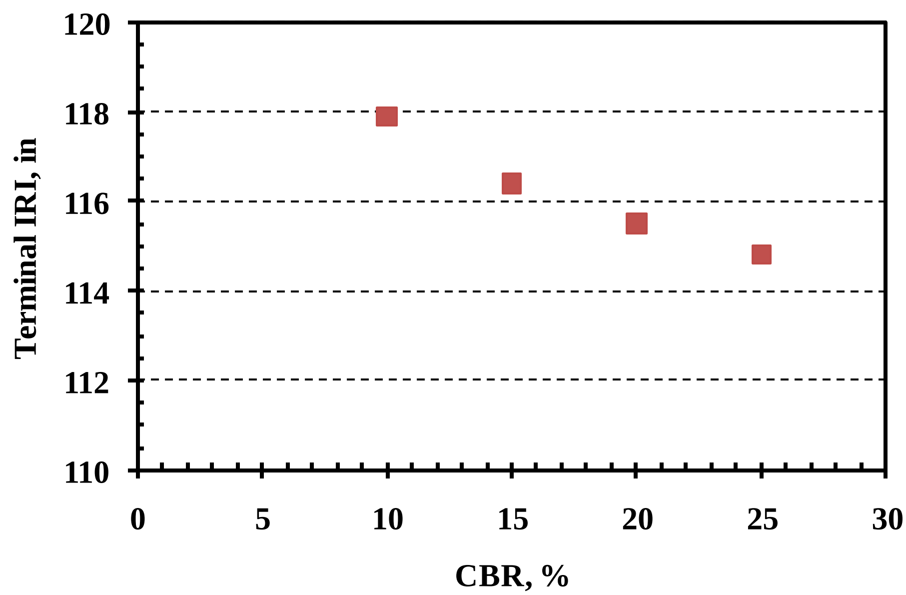


Figure 5. 5 Influence of CBR on Terminal IRI

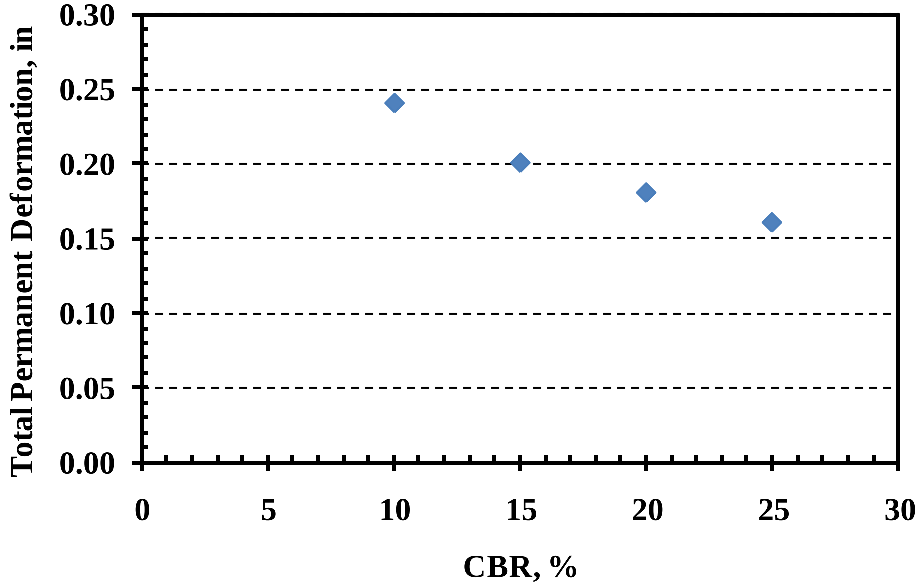


Figure 5. 6 Influence of CBR on Total Permanent Deformation

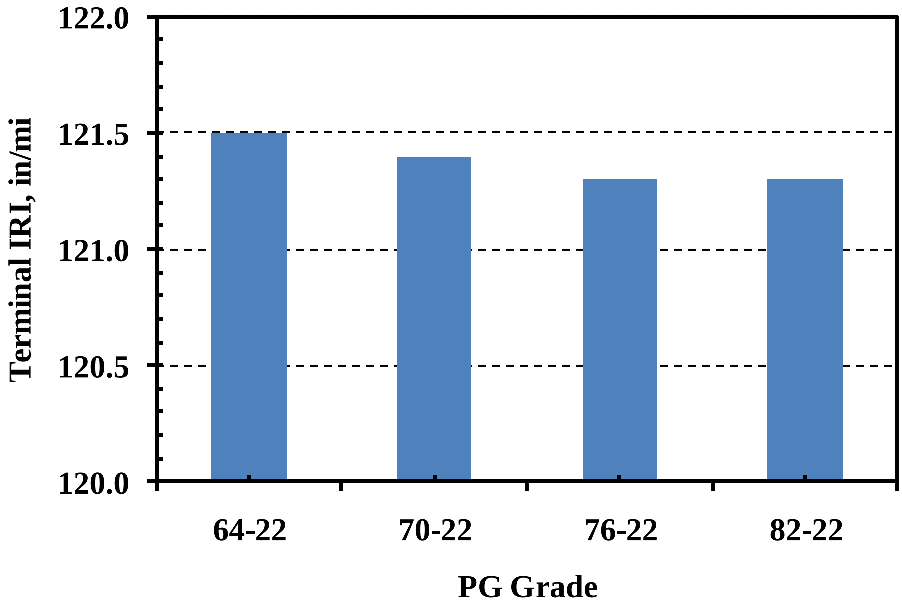


Figure 5. 7 Influence of PG Grade on Terminal IRI

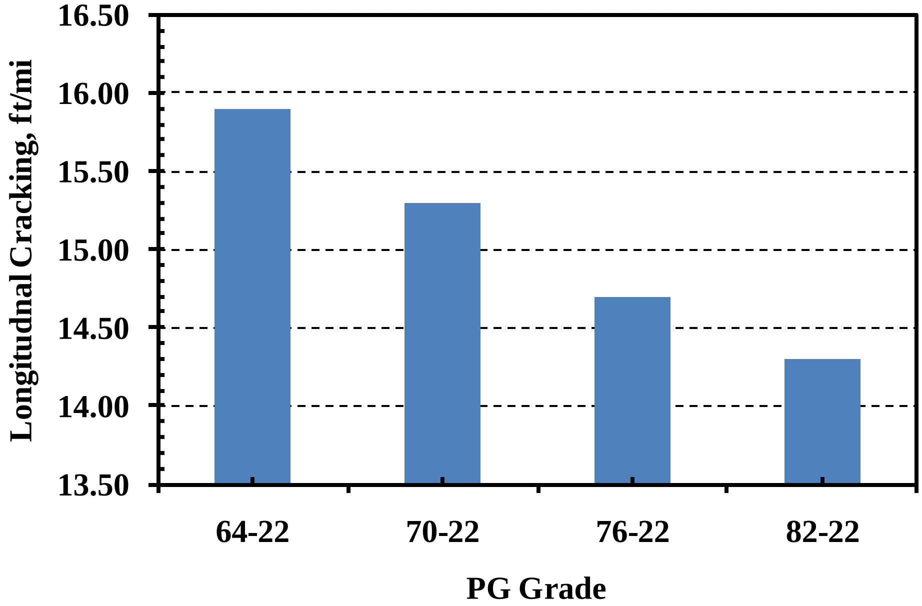


Figure 5. 8 Influence of PG Grade on Longitudinal Cracking

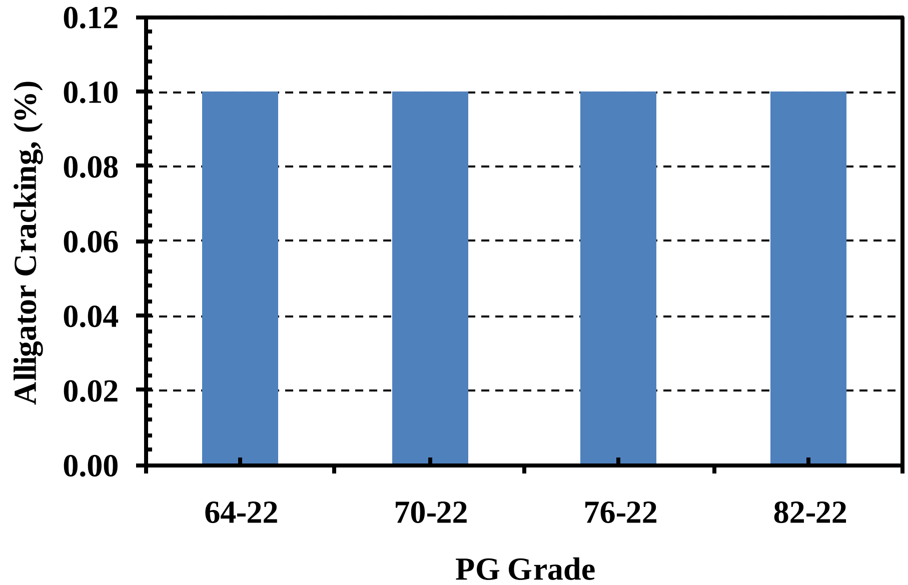


Figure 5. 9 Influence of PG Grade on Alligator Cracking

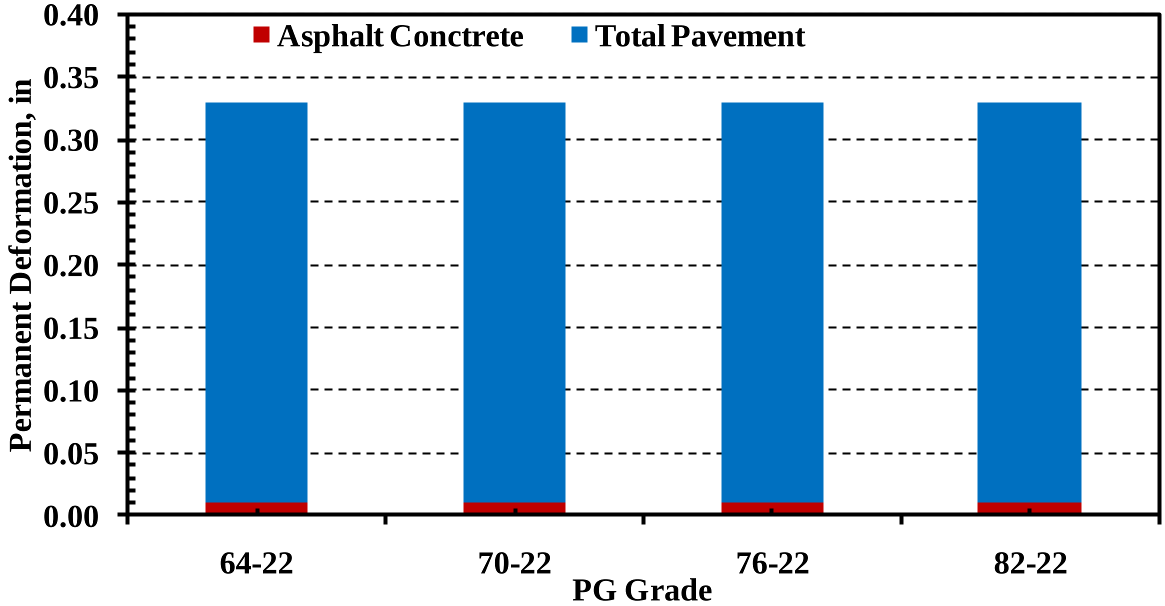


Figure 5. 10 Influence of PG Grade on Permanent Deformation

The results shown in Table 5.5 through 5.7 suggest that all of the pavement structures are adequate; however, the evaluations summarized in Chapter Four suggest that some of the pavement structures are not adequate. To identify reasons for the change in predicted performance, the variables varied from Chapter Four were identified and changed one at a time to identify their influence. The variables changed were: Level 1 versus Level 3 for asphalt concrete layers, traffic inputs like AADTT, hourly traffic distribution, and axle load configuration.

To identify influence of Level 1 versus Level 3, binder and asphalt concrete dynamic modulus test data obtained by Sugandh et al. (2007) was utilized. The dynamic modulus data for binder type PG76-22 and mix Type-D is included in Tables 5.8 and 5.9, respectively. For comparison purposes, the traffic input as shown in Figure 5.1 and Appendix B (axle load configuration as shown in Figure 3.7) was kept the same while asphalt concrete layer dynamic

modulus and binder properties were entered using Level 1. The runs were performed for 10 and 150 msa for all four CBR values. The results are summarized in Table 5.10 and suggest that there is negligible influence of using Level 1 versus Level 3 inputs. The asphalt concrete layer permanent deformations obtained with Level 1 are similar to the ones observed with Level 3 inputs. The other predicted stresses were similar as shown in Table 5.10 indicating that models incorporated in MEPDG are adequate in terms of predicting dynamic modulus values of asphalt concrete. Therefore, changes in predicted performance cannot be attributed to dynamic modulus prediction models.

Table 5. 8 Dynamic Modulus Data of the Type D Valero PG 76-22

Temperature °F	Mixture E* (psi)			
	1	**2**	**5**	**10**
10	2196000	2488000	2867000	3174000
40	1385000	1568000	1849000	2076000
70	659000	753000	942000	1087000
100	312000	351000	441000	505000
130	183000	196000	227000	252000

Table 5. 9 Superpave Binder Test Data

Temperature °F	Angular frequency = 10 rad/sec	
	G*, psi	**Delta (°)**
168.8	3340	67.8
147.2	10000	65.9
125.6	41100	60.8

Table 5. 10 Pavement Performance For Indian Traffic Conditions and Level 1 Data for Asphalt Concrete Layer

Asphalt Thickness, in	Base, Subbase and Subgrade Modulus, psi	AADTT	Distress Target :		172	2000	25	0.25	0.75
			PG Grade	CBR, %	Terminal IRI, in/mi	AC Surface Down Cracking (Long. Cracking), ft/mile	AC Bottom Up Cracking (Alligator Cracking), %	AC Permanent Deformation, in	Total Permanent Deformation, in
3.54	15596	550	76-22	10	123	30	0.20	0.01	0.34
3.54	20217	550	76-22	15	120	16	0.20	0.01	0.28
3.54	24303	550	76-22	20	119	10	0.10	0.01	0.25
3.54	28034	550	76-22	25	118	6	0.10	0.01	0.22
7.87	15596	8300	76-22	10	119	0	0.10	0.01	0.26
7.87	20217	8300	76-22	15	117	0	0.10	0.01	0.22
7.87	24303	8300	76-22	20	116	0	0.10	0.01	0.19
7.87	28034	8300	76-22	25	116	0	0.10	0.02	0.18

The second change was the axle load configuration. The default axle load configuration is shown in Figure 3.7 which was changed with the data obtained from IIT-M (Appendix B). The second set of runs were performed by using default axle load configuration values shown in Figure 3.7 and using the inputs discussed in previous section with one PG grade of 76-22 and two traffic loads (10 and 150 msa). The results are summarized in Table 5.11. The results suggest that the permanent deformation increased with use of default axle load configuration indicating that Indian axle loads configuration imparts lower traffic loads in comparison to USA. The total permanent deformation increased to 0.42 from 0.32 (Table 5.5) for 10 msa of traffic and CBR value of 10%. The increase was similar for other evaluated conditions indicating that the axle load configuration influences predicted performance.

In the end, the MEPDG runs were performed using default traffic values of New_HMA.dgp file with few exceptions (the changed values were: AADTT of 550, linear traffic growth of 7.5%, and percentage of trucks in design lane of 75%) The runs were performed for Level 1 as well as Level 3 inputs for asphalt concrete. The Level 1 inputs were as per Tables 5.8 and 5.9. The results of runs are summarized in Table 5.12. The results suggest that the traffic level is significantly different in India as compared to USA. The permanent deformation increased for evaluated traffic conditions from 0.42 to 0.60 for 10 msa and CBR value of 10%. The biggest increase was observed with longitudinal cracking where it increased to 1120 ft/mile from 14 ft/mile. The further evaluation indicated that only Class 4, 5 and 6 trucks were travelling in the evaluated site in India which imparts lower traffic loads in comparison to Class 7 or higher Class vehicles.

Overall, the IRC design guide is adequate for traffic levels evaluated if foundation layers are constructed with CBR of 10 or higher.

Table 5. 11 Outputs of Pavement Structures with Default Axle Load Data

Ashphalt Thickness (in)	Base, Subbase & Subgrade Modulus (psi)	AADTT	Distress Target :		172	2000	25	0.25	0.75
			PG Grade	CBR	Terminal IRI, in/mi	AC Surface Down Cracking (Long. Cracking) ft/mile	AC Bottom Up Cracking (Alligator Cracking) %	AC Permanent Deformation, in	Total Permanent Deformation, in
3.54	15596	550	82-22	10	126	81	0.70	0.08	0.42
3.54	20217	550	82-22	15	123	36	0.40	0.08	0.35
3.54	24303	550	82-22	20	121	19	0.30	0.07	0.31
3.54	28034	550	82-22	25	120	12	0.20	0.07	0.28
7.87	15596	8300	82-22	10	126	32	0.60	0.15	0.43
7.87	20217	8300	82-22	15	124	36	0.40	0.15	0.39
7.87	24303	8300	82-22	20	123	35	0.30	0.16	0.36
7.87	28034	8300	82-22	25	122	33	0.30	0.16	0.34

Table 5. 12 Outputs of Pavement Structures with Default Traffic Inputs at Different Levels of Inputs for Asphalt Layer

Ashphalt Thickness (in)	Base, Subbase And Subgrade Modulus (psi)	Level	Distress Target		172	2000	25	0.25	0.75
			PG Grade	CBR, %	Terminal IRI, in/mi	AC Surface Down Cracking (Long. Cracking), ft/mile	AC Bottom Up Cracking (Alligator Cracking), %	AC Permanent Deformation, in	Total Permanent Deformation, in
3.54	15596	3	64-22	10	136	1120	4.10	0.19	0.60
3.54	15596	3	76-22	10	133	1140	3.10	0.15	0.54
3.54	15596	3	82-22	10	132	1150	2.70	0.14	0.52
3.54	15596	1	76-22	10	133	1410	3.40	0.15	0.54
3.54	28034	1	76-22	15	133	1410	3.40	0.15	0.54

Chapter 6

CLOSURE

6.1 SUMMARRY

The main objective of the study was the evaluation of the pavements using MEPDG for India using Texas environmental conditions. The objective has been achieved by understanding the MEPDG and using the different pavement structure being used in Texas according to survey conducted by Murphy in 1997 for the sensitivity analysis of pavement structure are performed and evaluated at El Paso, Houston in Texas and Miami in Florida. The environmental conditions of El Paso were used as climatic model in MEPDG to evaluate the pavement structures according to the IRC 37: 2001 code, which is being followed in India.

6.2 CONCLUSIONS

The conclusions made from the study are as follows:

- Pavement structure is very important for the pavement performance.
- Pavement performance is very sensitive to the base thickness.
- Pavement performance is very sensitive to the base modulus.
- Pavement performance is sensitive to the climatic changes.
- The impact of PG grade binder on the pavement performance is moderate.
- The asphalt mix has minimal effect on the pavement performance.

- Pavement performance is highly sensitive to AADTT.
- The Impact of Axle load data on pavement performance is high.
- The vehicle classification has also significant influence on the pavement performance.
- The pavements designed with IRC37:2001 guidelines for CBR values greater than 10% are performing well at the considered traffic levels

6.3 RECOMMENDATIONS FOR FURTHER STUDY

The recommendations for the further study are:

- The sensitivity analysis of the flexible pavements for the asphalt layer inputs like PG grade, asphalt concrete Poisson's ratio, and asphalt concrete volumetric properties.
- The sensitivity analysis of the flexible pavements for the traffic inputs like tire pressure, traffic speed, etc.
- The IRC 37: 2001 code being practiced needs revision to include higher CBR values.

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APPENDIX A

DODDABALLAPURA TO DEVANAHALLI

Table A. 1 Light Commercial Vehicles

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF)

Light Commercial Vehicles

Axle Load Category (Tonnes)	Number Of Axles (n)	% Of Each Category (n/N)* 100	Cumulative Percentage	Equivalency Factors (E)	Equivalent Standard Axles (n * E)	% Of Damaging Effect (E X n / B) *100
0.900 - 1.810	8	30.77	30.77	0.002	0.02	0.18
1.810 - 2.720	6	23.08	53.85	0.009	0.05	0.60
2.720 - 3.630	3	11.54	65.38	0.031	0.09	1.03
3.630 - 4.540	3	11.54	76.92	0.080	0.24	2.66
5.440 - 6.350	3	11.54	88.46	0.350	1.05	11.62
8.160 - 9.070	2	7.69	96.15	1.550	3.10	34.32
10.890 - 11.790	1	3.85	100.00	4.480	4.48	49.60
N =	**26**			**B=**	**9.03**	**100**

No of Axles Weighed, X = N = 26	Total Damaging Effect, Z = B = 9.03
No of Vehicles Weighed, Y = 13	Axle Equivalency, (Z/X) = 0.35

Vehicle damage factor, (Z/Y)= 0.695

Table A. 2 Two Axle Trucks

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Two Axle Trucks						
Axle Load Category (Tonnes)	**Number Of Axles (n)**	**% Of Each Category (n/N)*100**	**Cumulative Percentage**	**Equivalency Factors (E)**	**Equivalent Standard Axles (n * E)**	**% Of Damaging Effect (E * n / B) * 100**
0.000 - 0.900	2	0.45	0.45	0.0002	0.00	0.00
0.900 - 1.810	3	0.67	1.12	0.002	0.01	0.00
1.810 - 2.720	24	5.36	6.47	0.009	0.22	0.02
2.720 - 3.630	96	21.43	27.90	0.031	2.98	0.26
3.630 - 4.540	90	20.09	47.99	0.080	7.20	0.62
4.540 - 5.440	46	10.27	58.26	0.176	8.10	0.70
5.440 - 6.350	38	8.48	66.74	0.350	13.30	1.15
6.350 - 7.260	43	9.60	76.34	0.610	26.23	2.27
7.260 - 8.160	17	3.79	80.13	1.000	17.00	1.47
8.160 - 9.070	16	3.57	83.71	1.550	24.80	2.15
9.070 - 9.980	6	1.34	85.04	2.300	13.80	1.20
9.980 - 10.890	6	1.34	86.38	3.270	19.62	1.70
10.890 - 11.790	14	3.13	89.51	4.480	62.72	5.44
11.790 - 12.700	6	1.34	90.85	5.980	35.88	3.11
13.610 - 14.520	4	0.89	93.30	10.000	40.00	3.47
12.700 - 13.610	7	1.56	92.41	7.800	54.60	4.74

Table A.2 Two Axle Trucks (Continued)

Axle Load Category (Tonnes)	Number Of Axles (n)	% Of Each Category (n/N)*100	Cumulative Percentage	Equivalency Factors (E)	Equivalent Standard Axles (n * E)	% Of Damaging Effect (E * n / B) * 100
14.520 - 15.420	7	1.56	94.87	12.500	87.50	7.59
15.420 - 16.320	3	0.67	95.54	15.500	46.50	4.03
16.320 - 17.230	1	0.22	95.76	19.000	19.00	1.65
17.230 - 18.140	4	0.89	96.65	23.000	92.00	7.98
18.140 - 19.051	3	0.67	97.32	27.700	83.10	7.21
19.051 - 19.958	3	0.67	97.99	33.000	99.00	8.59
19.958 - 20.865	5	1.12	99.11	39.300	196.50	17.04
20.865 - 21.772	2	0.45	99.55	46.500	93.00	8.07
21.772 - 22.680	2	0.45	100.00	55.000	110.00	9.54
N =	448			**B=**	**1153.04**	**100**
No of Axles Weighed, X = N =			448	Total Damaging Effect, Z = B =		1153.04
No of Vehicles Weighed, Y =			224	Axle Equivalency, (Z/X) =		2.57
Vehicle damage factor, (Z/Y)=				**5.148**		

Table A. 3 Buses

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Buses						
Axle Load Category (Tonnes)	**Number Of Axles (n)**	**% Of Each Category (n/N)*100**	**Cumulative Percentage**	**Equivalency Factors (E)**	**Equivalent Standard Axles (n * E)**	**% Of Damaging Effect (E * n / B) * 100**
1.810 - 2.720	1	12.50	12.50	0.009	0.01	0.94
2.720 - 3.630	3	37.50	50.00	0.031	0.09	9.67
3.630 - 4.540	2	25.00	75.00	0.080	0.16	16.63
5.440 - 6.350	2	25.00	100.00	0.350	0.70	72.77
N =	**8**			**B=**	**0.96**	**100**
No of Axles Weighed, X = N = 8				Total Damaging Effect,Z = B = 0.96		
No of Vehicles Weighed, Y = 4				Axle Equivalency, (Z/X) = 0.12		
Vehicle damage factor, (Z/Y)= 0.241						

Table A. 4 Three Axle Truck

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Three Axle Truck Single Front Axle						
Axle Load Category (Tonnes)	**Number Of Axles (n)**	**% Of Each Category (n/N1)* 100**	**Cumulative Percentage**	**Equivalency Factors (E)**	**Equivalent Standard Axles (n * E)**	**% Of Damaging Effect (E *N / B1) * 100**
1.810 - 2.720	4	2.47	2.47	0.009	0.04	0.06
2.720 - 3.630	6	3.70	6.17	0.031	0.19	0.30
3.630 - 4.540	25	15.43	21.60	0.080	2.00	3.26
4.540 - 5.440	32	19.75	41.36	0.176	5.63	9.17
5.440 - 6.350	53	32.72	74.07	0.350	18.55	30.21
6.350 - 7.260	31	19.14	93.21	0.610	18.91	30.80
7.260 - 8.160	8	4.94	98.15	1.000	8.00	13.03
8.160 - 9.070	1	0.62	98.77	1.550	1.55	2.52
9.980 - 10.890	2	1.23	100.00	3.270	6.54	10.65
N1 =	**162**			**B1=**	**61.40**	**100**

Table A. 4 Three Axle Truck (Continued)

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Tandem Rear Axle						
Axle Load Category (Tonnes)	**Number Of Axles (n)**	**% Of Each Category (n/N2)* 100**	**Cumulative Percentage**	**Equivalency Factors (E)**	**Equivalent Standard Axles (n * E)**	**% Of Damaging Effect (E * n / B2) * 100**
3.630 - 4.540	3	1.85	1.85	0.006	0.02	0.00
4.540 - 5.440	10	6.17	8.02	0.013	0.13	0.03
5.440 - 6.350	6	3.70	11.73	0.024	0.14	0.03
6.350 - 7.260	2	1.23	12.96	0.043	0.09	0.02
8.160 - 9.070	1	0.62	13.58	0.110	0.11	0.02
9.070 - 9.980	1	0.62	14.20	0.166	0.17	0.03
9.980 - 10.890	3	1.85	16.05	0.242	0.73	0.14
10.890 - 11.790	1	0.62	16.67	0.342	0.34	0.07
11.790 - 12.700	2	1.23	17.90	0.470	0.94	0.18
12.700 - 13.610	8	4.94	22.84	0.633	5.06	0.99
13.610 - 14.520	4	2.47	25.31	0.834	3.34	0.65
14.520 - 15.420	5	3.09	28.40	1.080	5.40	1.05
15.420 - 16.320	9	5.56	33.95	1.380	12.42	2.42
16.320 - 17.230	10	6.17	40.12	1.730	17.30	3.37
17.230 - 18.140	10	6.17	46.30	2.140	21.40	4.17

Table A.4Three Axle Truck (Continued)

Axle Load Category (Tonnes)	Number Of Axles (n)	% Of Each Category (n/N2)* 100	Cumulative Percentage	Equivalency Factors (E)	Equivalent Standard Axles (n * E)	% Of Damaging Effect (E *n / B2) * 100
18.140 - 19.051	24	14.81	61.11	2.610	62.64	12.20
19.051 - 19.958	13	8.02	69.14	3.160	41.08	8.00
19.958 - 20.865	16	9.88	79.01	3.790	60.64	11.81
20.865 - 21.772	8	4.94	83.95	4.490	35.92	6.99
21.772 - 22.680	4	2.47	86.42	5.280	21.12	4.11
22.680 - 23.587	4	2.47	88.89	6.170	24.68	4.81
23.587 - 24.494	3	1.85	90.74	7.150	21.45	4.18
24.494 - 25.401	5	3.09	93.83	8.200	41.00	7.98
25.401 - 26.308	1	0.62	94.44	9.400	9.40	1.83
26.308 - 27.216	1	0.62	95.06	10.700	10.70	2.08
27.216 - 28.123	4	2.47	97.53	12.100	48.40	9.42

Table A.4 Three Axle Truck (Continued)

Axle Load Category (Tonnes)	Numbe Numbe r Of Axles (n)	% Of Each Categor y (n/N)*100	Cumulativ e Percentage	Equivalency Factors (E)	Equivalent Standard Axles (n X E)	% Of Damaging Effect (E X n / B) * 100
29.030 - 29.937	1	0.62	98.15	15.400	15.40	3.00
29.937 - 30.844	2	1.23	99.38	17.200	34.40	6.70
30.844 - 31.752	1	0.62	100.00	19.200	19.20	3.74
N2 =	**162**			**B2=**	**513.61**	**100**
No of Axles Weighed, X = N1+N2 =			324	Total Damaging Effect, Z = B1+B2 =		575.02
No of Vehicles Weighed, Y =			162	Axle Equivalency, (Z/X) =		1.77
Vehicle damage factor, (Z/Y)=				**3.549**		

Table A. 5 Semi Truck Trailor with Tandem Rear Axle

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Single Front Axle, and Middle Axle						
Axle load category (Tonnes)	**Number of axles (n)**	**% of each category (n/N1)x100**	**Cumulative percentage**	**Equivalency factors (e)**	**Equivalent standard axles (n x e)**	**% of damaging effect (e x n / B1) x 100**
4.540 - 5.440	3	30.00	30.00	0.176	0.53	5.96
5.440 - 6.350	1	10.00	40.00	0.350	0.35	3.95
6.350 - 7.260	3	30.00	70.00	0.610	1.83	20.66
8.160 - 9.070	1	10.00	80.00	1.550	1.55	17.50
9.070 - 9.980	2	20.00	100.00	2.300	4.60	51.93
N1 =	**10**			**B1=**	**8.86**	**100**

Table A.5 Semi Truck Trailor with Tandem Rear Axle (Continued)

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF)						
Tandem Rear Axle						
Axle load category (Tonnes)	**Number of axles (n)**	**% of each category (n/N2)x100**	**Cumulative percentage**	**Equivalency factors (e)**	**Equivalent standard axles (n x e)**	**% of damaging effect (e x n / B2) x 100**
9.070 - 9.980	2	40.00	40.00	0.166	0.33	0.84
9.980 - 10.890	1	20.00	60.00	0.242	0.24	0.61
19.051 - 19.958	1	20.00	80.00	3.160	3.16	7.97
36.288 - 37.000	1	20.00	100.00	35.920	35.92	90.58
N2 =	**5**			**B2=**	**39.65**	**100**
No of Axles Weighed, X = N1+N2 =			15	Total Damaging Effect,Z = B1+B2 =		48.51
No of Vehicles Weighed, Y =			5	Axle Equivalency, (Z/X) =		3.23
Vehicle damage factor, (Z/Y)=				**9.702**		

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Table A. 6 Light Commercial Vehicles-Goods

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) **Light Commercial Vehicles**						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B) x 100**
0.900 - 1.810	19	25.68	25.68	0.002	0.04	0.81
1.810 - 2.720	26	35.14	60.81	0.009	0.23	5.00
2.720 - 3.630	11	14.86	75.68	0.031	0.34	7.29
3.630 - 4.540	11	14.86	90.54	0.080	0.88	18.81
4.540 - 5.440	1	1.35	91.89	0.176	0.18	3.76
5.440 - 6.350	4	5.41	97.30	0.350	1.40	29.92
6.350 - 7.260	1	1.35	98.65	0.610	0.61	13.04
7.260 - 8.160	1	1.35	100.00	1.000	1.00	21.37
N =	**74**			**B=**	**4.68**	**100**
No of Axles Weighed, X = N = 74				Total Damaging Effect,Z = B = 4.68		
No of Vehicles Weighed, Y = 37				Axle Equivalency, (Z/X) = 0.06		
Vehicle damage factor, (Z/Y)= 0.126						

Table A. 7 Light Commercial Vehicles-Passenger

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Light Commercial Vehicles						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B) x 100**
1.810 - 2.720	3	50.00	50.00	0.009	0.03	1.69
5.440 - 6.350	1	16.67	66.67	0.350	0.35	21.92
6.350 - 7.260	2	33.33	100.00	0.610	1.22	76.39
N =	**6**			**B=**	**1.60**	**100**
No of Axles Weighed, X = N = 6				Total Damaging Effect, Z = B = 1.60		
No of Vehicles Weighed, Y = 3				Axle Equivalency, (Z/X) = 0.27		
Vehicle damage factor, (Z/Y)= 0.532						

Table A. 8 Two Axle Trucks

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) **Two Axle Trucks**						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B) x 100**
0.000 - 0.900	2	0.56	0.56	0.0002	0.00	0.00
0.900 - 1.810	3	0.84	1.40	0.002	0.01	0.00
1.810 - 2.720	21	5.90	7.30	0.009	0.19	0.02
2.720 - 3.630	41	11.52	18.82	0.031	1.27	0.11
3.630 - 4.540	56	15.73	34.55	0.080	4.48	0.39
4.540 - 5.440	30	8.43	42.98	0.176	5.28	0.45
5.440 - 6.350	43	12.08	55.06	0.350	15.05	1.29
6.350 - 7.260	32	8.99	64.04	0.610	19.52	1.68
7.260 - 8.160	14	3.93	67.98	1.000	14.00	1.20
8.160 - 9.070	10	2.81	70.79	1.550	15.50	1.33
9.070 - 9.980	17	4.78	75.56	2.300	39.10	3.36
9.980 - 10.890	13	3.65	79.21	3.270	42.51	3.66
10.890 - 11.790	12	3.37	82.58	4.480	53.76	4.62
11.790 - 12.700	8	2.25	84.83	5.980	47.84	4.12
12.700 - 13.610	17	4.78	89.61	7.800	132.60	11.41

Table A. 8 Two Axle Trucks (Continued)

Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B) x 100**
13.610 - 14.520	8	2.25	91.85	10.000	80.00	6.88
14.520 - 15.420	3	0.84	92.70	12.500	37.50	3.23
15.420 - 16.320	5	1.40	94.10	15.500	77.50	6.67
16.320 - 17.230	5	1.40	95.51	19.000	95.00	8.17
17.230 - 18.140	4	1.12	96.63	23.000	92.00	7.91
18.140 - 19.051	5	1.40	98.03	27.700	138.50	11.91
19.051 - 19.958	5	1.40	99.44	33.000	165.00	14.19
19.958 - 20.865	1	0.28	99.72	39.300	39.30	3.38
20.865 - 21.772	1	0.28	100.00	46.500	46.50	4.00
N =	356			**B=**	**1162.41**	**100**
No of Axles Weighed, X = N =			356	Total Damaging Effect, Z =		1162.41
No of Vehicles Weighed, Y =			178	Axle Equivalency, (Z/X) =		3.27
Vehicle damage factor, (Z/Y)=				**6.530**		

Table A. 9 Buses

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF)						
Buses						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B) x 100**
1.810 - 2.720	3	7.14	7.14	0.009	0.03	0.11
2.720 - 3.630	6	14.29	21.43	0.031	0.19	0.77
3.630 - 4.540	10	23.81	45.24	0.080	0.80	3.30
4.540 - 5.440	7	16.67	61.90	0.176	1.23	5.09
5.440 - 6.350	4	9.52	71.43	0.350	1.40	5.78
6.350 - 7.260	6	14.29	85.71	0.610	3.66	15.11
7.260 - 8.160	2	4.76	90.48	1.000	2.00	8.26
8.160 - 9.070	1	2.38	92.86	1.550	1.55	6.40
9.070 - 9.980	1	2.38	95.24	2.300	2.30	9.49
9.980 - 10.890	1	2.38	97.62	3.270	3.27	13.50
12.700 - 13.610	1	2.38	100.00	7.800	7.80	32.20
N =	**42**			**B=**	**24.23**	**100**
No of Axles Weighed, X = N = 42				Total Damaging Effect, Z = B = 24.23		
No of Vehicles Weighed, Y = 21				Axle Equivalency, (Z/X) = 0.58		
Vehicle damage factor, (Z/Y)= 1.154						

Table A. 10 Three Axle Truck

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Single Front Axle						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N1)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B1) x 100**
2.720 - 3.630	5	4.35	4.35	0.031	0.16	0.21
3.630 - 4.540	23	20.00	24.35	0.080	1.84	2.49
4.540 - 5.440	21	18.26	42.61	0.176	3.70	5.01
5.440 - 6.350	10	8.70	51.30	0.350	3.50	4.74
6.350 - 7.260	23	20.00	71.30	0.610	14.03	19.01
7.260 - 8.160	11	9.57	80.87	1.000	11.00	14.91
8.160 - 9.070	16	13.91	94.78	1.550	24.80	33.61
9.070 - 9.980	5	4.35	99.13	2.300	11.50	15.58
9.980 - 10.890	1	0.87	100.00	3.270	3.27	4.43
N1 =	**115**			**B1=**	**73.79**	**100**

Table A.10 Three Axle Truck (Continued)

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Tandem Rear Axle						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N2)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B2) x 100**
4.540 - 5.440	10	8.70	8.70	0.013	0.13	0.02
5.440 - 6.350	12	10.43	19.13	0.024	0.29	0.05
6.350 - 7.260	13	11.30	30.43	0.043	0.56	0.10
7.260 - 8.160	3	2.61	33.04	0.070	0.21	0.04
8.160 - 9.070	1	0.87	33.91	0.110	0.11	0.02
9.070 - 9.980	2	1.74	35.65	0.166	0.33	0.06
9.980 - 10.890	2	1.74	37.39	0.242	0.48	0.08
10.890 - 11.790	1	0.87	38.26	0.342	0.34	0.06
16.320 - 17.230	2	1.74	40.00	1.730	3.46	0.60
17.230 - 18.140	1	0.87	40.87	2.140	2.14	0.37
18.140 - 19.051	4	3.48	44.35	2.610	10.44	1.81
19.051 - 19.958	3	2.61	46.96	3.160	9.48	1.64
19.958 - 20.865	2	1.74	48.70	3.790	7.58	1.31
20.865 - 21.772	4	3.48	52.17	4.490	17.96	3.11
21.772 - 22.680	5	4.35	56.52	5.280	26.40	4.58
22.680 - 23.587	10	8.70	65.22	6.170	61.70	10.70

Table A.10 Three Axle Truck (Continued)

Axle Load Category (Tonnes)	Number of Axles (n)	% of Each Category (n/N2)x100	Cumulative Percentage	Equivalency Factors (e)	Equivalent Standard Axles (n x e)	% of Damaging Effect (e x n / B2) x 100
23.587 - 24.494	9	7.83	73.04	7.150	64.35	11.16
24.494 - 25.401	8	6.96	80.00	8.200	65.60	11.38
25.401 - 26.308	8	6.96	86.96	9.400	75.20	13.04
26.308 - 27.216	6	5.22	92.17	10.700	64.20	11.13
28.123 - 29.030	1	0.87	93.04	13.700	13.70	2.38
29.937 - 30.844	3	2.61	95.65	17.200	51.60	8.95
30.844 - 31.752	4	3.48	99.13	19.200	76.80	13.32
32.660 - 33.566	1	0.87	100.00	23.600	23.60	4.09
N2 =	**115**			**B2=**	**576.67**	**100**
No of Axles Weighed, X = N1+N2 =			230	Total Damaging Effect, Z = B1+B2 =		650.46
No of Vehicles Weighed, Y =			115	Axle Equivalency, (Z/X) =		2.83
Vehicle damage factor, (Z/Y)=				**5.656**		

Table A. 11 Semi Truck Trailor with Tandem Rear Axle

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Single Front Axle and Middle Axle						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N1)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B1) x 100**
2.720 - 3.630	1	16.67	16.67	0.031	0.03	0.57
3.630 - 4.540	1	16.67	33.33	0.080	0.08	1.47
4.540 - 5.440	1	16.67	50.00	0.176	0.18	3.23
5.440 - 6.350	1	16.67	66.67	0.350	0.35	6.41
8.160 - 9.070	1	16.67	83.33	1.550	1.55	28.40
9.980 - 10.890	1	16.67	100.00	3.270	3.27	59.92
N1 =	**6**			**B1=**	**5.46**	**100**

Table A.11 Semi Truck Trailor with Tandem Rear Axle (Continued)

AXLE LOAD DISTRIBUTION AND VEHICLE DAMAGE FACTOR (VDF) Tandem Rear Axle						
Axle Load Category (Tonnes)	**Number of Axles (n)**	**% of Each Category (n/N2)x100**	**Cumulative Percentage**	**Equivalency Factors (e)**	**Equivalent Standard Axles (n x e)**	**% of Damaging Effect (e x n / B2) x 100**
10.890 - 11.790	1	33.33	33.33	0.342	0.34	2.62
20.865 - 21.772	1	33.33	66.67	4.490	4.49	34.45
24.494 - 25.401	1	33.33	100.00	8.200	8.20	62.92
N2 =	**3**			**B2=**	**13.03**	**100**
No of Axles Weighed, X = N1+N2 =			9	Total Damaging Effect, Z = B1+B2 =		18.49
No of Vehicles Weighed, Y =			3	Axle Equivalency, (Z/X) =		2.05
Vehicle damage factor, (Z/Y)=				**6.163**		

APPENDIX B

Table B. 1 Chitradurga to Hospet Day 1

Time	Car/Jeep	Motorcycle	LCV	Bus	Truck			Tractor	
					1-Axle	2-Axle	Multi Axle	with Tailor	without Tailor
9.00am-10.00 am	6	9	2	15	28	50	13	-	-
10.00am-11.00am	14	10	9	5	24	36	9	-	-
11.00am-12.00pm	6	17	3	3	19	52	5	-	-
12.00pm-1.00pm	23	27	2	10	24	44	11	-	-
1.00pm-2.00pm	23	15	2	12	46	57	18	-	1
2.00pm-3.00pm	13	23	4	3	36	65	11	-	-
3.00pm-4.00pm	17	10	4	4	21	71	4	1	1
4.00pm-5.00pm	16	16	3	10	48	70	10	-	1
5.00pm-6.00pm	16	24	2	9	42	81	6	-	2
6.00pm-7.00pm	11	26	3	4	33	67	4	-	1
7.00pm-8.00pm	18	17	1	7	27	37	4	2	-
8.00pm-9.00pm	21	14	6	3	35	72	17	-	-
9.00pm-10.00pm	16	8	6	3	25	48	3	-	-
10.00pm-11.00pm	9	6	3	5	22	46	2	-	-
11.00pm-12.00am	11	1	2	3	32	52	6	-	-
12.00am-1.00am	6	1	2	4	28	43	7	-	1
1.00am-2.00am	2	1	4	8	18	45	4	-	-
2.00am-3.00am	4		2	34	53	59	9	-	-
3.00am-4.00am	2		1	13	35	56	7	-	-

Table B.1 Chitradurga to Hospet Day 1(Continued)

Time	Car/Jeep	Motorcycle	LCV	Bus	Truck			Tractor	
					1-Axle	2-Axle	Multi Axle	with Tailor	without Tailor
5.00am-6.00am	9	3	4	24	52	63	4	-	-
6.00am-7.00am	10	5	6	5	36	55	10	-	-
7.00am-8.00am	21	10	5	9	41	84	11	-	1
8.00am-9.00am	13	6	5	3	32	70	5	-	1
Total - one direction			82	212	803	1387	183	3	9
Grand total - one direction			**2670**						

Table B. 2 Hospet-Chitradurga Day 1

Time	Car/Jeep	Motorcycle	LCV	Bus	Truck			Tractor	
					1-Axle	2-Axle	Multi Axle	with Tailor	without Tailor
9.00am-10.00 am	7	5	2	4	37	50	5	-	-
10.00am-11.00am	6	8	5	8	24	78	7	-	1
11.00am-12.00pm	12	15	4	5	30	66	11	-	-
12.00pm-1.00pm	16	21	7	2	25	43	9	-	-
1.00pm-2.00pm	23	22	7	6	30	41	5	-	-
2.00pm-3.00pm	14	31	2	8	24	50	7	-	-
3.00pm-4.00pm	10	12	6	4	33	60	6	-	4
4.00pm-5.00pm	22	24	5	5	34	65	8	-	-
5.00pm-6.00pm	20	20	5	9	28	70	5	-	3
6.00pm-7.00pm	25	23	8	4	45	78	9	-	1

Table B.2 Hospet-Chitradurga Day 1 (Conntinued)

Time	Car/Jeep	Motorcycle	LCV	Bus	Truck			Tractor	
					1-Axle	2-Axle	Multi Axle	with Tailor	without Tailor
7.00pm-8.00pm	8	14	8	3	26	51	12	-	-
9.00pm-10.00pm	9	5	17	2	35	58	7	-	-
10.00pm-11.00pm	10	5	5	8	31	72	2	-	-
11.00pm-12.00am	8	7	7	8	22	40	5	-	-
12.00am-1.00am	5	2	4	20	31	65	2	-	-
1.00am-2.00am	4	-	3	35	26	55	7	-	-
2.00am-3.00am	12	-	7	32	43	60	3	-	-
3.00am-4.00am	7	-	11	23	46	70	5	-	-
4.00am-5.00am	4	-	8	25	32	55	6	-	-
5.00am-6.00am	5	2	8	3	22	82	11	-	-
6.00am-7.00am	5	3	5	2	36	85	6	-	-
7.00am-8.00am	10	7	10	5	43	75	16	1	3
8.00am-9.00am	12	12	3	2	30	71	7	-	-
Total - one direction			155	225	760	1503	171	1	
Grand Total - one direction			**2815**						

Table B. 3 Chitradurga to Hospet Day 2

Time	Car/Jeep	Motorcycle	LCV	Bus	Truck			Tractor	
					1-Axle	2-Axle	Multi Axle	with Tailor	without Tailor
3.30pm-4.30pm	18	13	1	12	32	67	6	-	-
4.30pm-5.30pm	13	7	2	5	45	72	11	-	-
5.30pm-6.30pm	16	8	2	6	47	81	9	-	-
6.30pm-7.30pm	13	12	3	2	52	77	7	-	-
7.30pm-8.30pm	17	13	5	4	48	78	2	-	-
8.30pm-9.30pm	11	8	2	1	21	58	3	-	-
9.30pm-10.30pm	7	5	1	7	34	67	1	2	1
10.30pm-11.30pm	10	3	8	4	35	63	2	-	-
11.30pm-12.30am	3	4	6	1	21	51	2	-	-
12.30am-1.30am	4	1	4	5	28	60	2	-	-
1.30am-2.30am	6	-	3	13	42	60	1	-	-
2.30am-3.30am	9	-	3	16	33	53	1	-	-
3.30am-4.30am	4	-	8	19	25	54	3	-	-
4.30am-5.30am	6	-	1	22	37	41	8	-	-
5.30am-6.30am	11	2	2	6	40	62	2	-	-
6.30am-7.30am	8	2	2	5	36	84	4	-	1
7.30am-8.30am	8	21	6	9	59	69	9	2	-
8.30am-9.30am	13	21	7	12	41	77	14	3	-
9.30am-10.30 am	11	26	7	10	52	58	5	3	-
10.30am-11.30am	12	46	7	7	45	69	11	-	-
11.30am-12.30pm	22	37	11	4	38	83	5	1	-
12.30pm-1.30pm	15	33	10	10	57	72	4	-	-
1.30pm-2.30pm	12	26	6	4	55	86	6	-	-
Total - one direction			**123**	**189**	**969**	**1648**	**123**	**11**	
Grand Total - one direction			**3063**						

Table B. 4 Hospet-Chitradurga Day 2

Time	Car/Jeep	Motorcycle	LCV	Bus	Truck			Tractor	
					1-Axle	2-Axle	Multi Axle	with Tailor	without Tailor
3.30pm-4.30pm	17	13	2	8	46	62	15	1	-
4.30pm-5.30pm	10	16	1	6	29	71	5	1	-
5.30pm-6.30pm	19	13	-	6	48	105	6	-	-
6.30pm-7.30pm	16	17	-	5	50	108	8	1	-
7.30pm-8.30pm	16	24	2	4	41	89	7	-	-
8.30pm-9.30pm	11	14	1	-	29	70	5	-	-
9.30pm-10.30pm	7	2	1	4	59	79	5	-	-
10.30pm-11.30pm	6	1	6	14	32	115	7	-	-
11.30pm-12.30am	8	2	7	25	37	72	8	-	-
12.30am-1.30am	7	1	4	18	12	76	-	1	-
1.30am-2.30am	6	-	4	20	22	61	1	1	-
2.30am-3.30am	4	-	4	12	31	69	2	-	-
3.30am-4.30am	4	-	4	2	15	82	2	-	-
4.30am-5.30am	2	-	1	2	12	75	4	-	-
5.30am-6.30am	7	-	2	1	7	63	3	-	-
6.30am-7.30am	4	-	1	2	23	101	2	1	-
8.30am-9.30am	11	21	3	6	30	103	4	1	-
9.30am-10.30 am	12	18	3	9	38	59	2		-
10.30am-11.30am	15	21	3	5	19	64	8	1	-
11.30am-12.30pm	18	26	8	5	30	57	5	-	-
12.30pm-1.30pm	6	18	16	7	17	58	3	-	-
1.30pm-2.30pm	7	17	9	7	21	37	3	-	-
2.30pm-3.30pm	13	20	15	4	31	55	11	2	-
Total - one direction			**99**	**174**	**709**	**1799**	**121**	**12**	
Grand Total - one direction			**2914**						

CURRICULUM VITA

Vamsi Krishna Pinnamaneni was born in Vijayawada, Andhra Pradesh, India. The second son of Rama Mohana Rao Pinnamaneni, he has completed schooling in March 2002 from V.P.S Public school, Andhra Pradesh, India and completed his intermediate in April, 2004. He has joined V.R. Siddhartha Engineering College in October 2004 and completed his bachelor's in civil engineering in May 2008 from V.R. Siddhartha Engineering College which is affiliated to Acharya Nagarjuna University. He has done a project on the suitable foundation design for the multi storied building and with an interest in geotechnical engineering he has joined University of Texas at El Paso in the fall of 2008 to pursue master's degree with a specialization in geotechnical engineering and materials. He has got good knowledge and skills in MS Word, MS Excel, MS Project, Primavera and MEPDG. He has worked in the project of Indo-US scientific Technology Forum (IUSSTF) from 2008 to 2010.

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