AUBURN UNIVERSITY

Final Report

INVESTIGATION OF THE 2002 DESIGN GUIDE FOR PAVEMENT STRUCTURES (FLEXIBLE PAVEMENTS)

Prepared by

M. Stroup-Gardiner
Casey W. Robinson

April 2006

AUBURN
GINN COLLEGE OF ENGINEERING

Highway Research Center
Harbert Engineering Center
Auburn University, Alabama 36849-5337

www.eng.auburn.edu/research/centers/hrc.html

INVESTIGATION OF THE 2002 DESIGN GUIDE FOR PAVEMENT STRUCTURES (FLEXIBLE PAVEMENTS)

Final Report 930-554

Mary Stroup-Gardiner
Casey W. Robinson

Civil Engineering Department
238 Harbert
Auburn, Alabama 36849
(334) 844-6280

Prepared for
Alabama Department of Transportation

April 10, 2006

DISCLAIMER

THE CONTENTS OF THIS REPORT REFLECT THE VIEWS OF THE AUTHORS WHO ARE RESPONSIBLE FOR THE FACTS AND ACCURACY OF THE DATA PRESENTED HERIN. THE CONTENTS DO NOT NECESSARILY REFLECT THE OFFICILA VIEWS OR POLICIES OF THE ALABAMA DEPARTMTNE OF TRANSPORTATION OR AUBURN UNIVERSITY. THIS REPORT DOES NOT CONSTITUTE A STANDARD, SPECIFICATION, OR REGULATION

EXECUTIVE SUMMARY

This report details the evaluation of the 2002 Design Guide for flexible pavement design. The objectives of the research were to determine: 1) user-friendliness, 2) information needed to be collected by ALDOT to run the program, and the 3) reasonableness of the output for flexible pavement design.

The 2002 beta version of the software was used to complete pavement designs that considered varying HMA layer thickness (3 levels), thickness (2 levels) of a single crushed aggregate base, subgrade type (2 levels), and traffic (3 levels). Prior to conducting the pavement designs, each of the software input screens were evaluated for information needed for the 2002 Design Guide as compared to the 1993 Design Guide. Once a listing of the information needed to run the program was developed, variables such as traffic growth, percent of traffic in design lane, and material layer moduli were standardized for all runs. The output, in the form of predicted pavement distresses, was evaluated for reasonableness. The ALDOT Traffic Statistics website was used to define a number of the traffic input parameters. Material properties, such as resilient modulus, were selected based on typical values correlated with traditional properties (e.g. California Bearing Ratio). The beta version of the program was consistently unreliable with numerous programming problems. A thorough literature review of the equations selected for use in the program was conducted to establish whether the problem with the output could be the model(s) used or simply "bugs" in the programming code. A second evaluation of the flexible pavement design software was conducted once it was released to the public to determine if the program, as released, produced.

The ***Project Information*** screens require the same information as is currently logged for all Alabama Department of Transportation projects, so no change in ALDOT practices are needed for this section.

The selection of the ***Analysis Parameters*** requires the user define the threshold values for individual pavement distresses. It is recommended that ALDOT review their pavement management data base in order to develop rational levels of distresses for specific time periods (e.g, 5, 10, 15, and 20 years).

The Alabama ***Traffic*** Statistic website is a very good, very easy to use source for most of the traffic parameters needed for the analysis. Information that is available and needed include:

- AADT
- TADT
- Percent heavy trucks
- Functional classification

Load distribution (spectra) can be accounted for in the analysis by selecting the appropriate roadway functional class. Care needs to be taken to make sure that this is done as the screen for this selection is one of the very few that are not easily found. Adjustment of the default percent trucks should be considered critical when the AADTT is less than 20%. Using the default values without adjustment for actual traffic conditions could result in pavements that are substantially over- or under-designed. The directional distribution in the design hourly volume ranges from 55 to 70%, with the average being 60% for sites studies in Alabama. An increase in the default directional distribution from 50 to 60% should be considered to avoid possibly designing inadequate pavement structures.

A linear relationship for traffic growth in Alabama was seen for all of the data evaluated. This type of relationship is recommended as the default for Alabama. The linear change in traffic with time, over a 10-year period, was well correlated with the AADT. The default Level 3 percent growth of 4% appears to be high for facilities with less than 20,000 ADT. If at all possible, historical traffic data for local facilities should be used for estimating the percent growth to avoid the possibility of over-design.

The inputs for ***Climate*** can be obtained from a number of files included in the 2002 Design Guide software. While the weather data may not be specifically developed for the project site, there seems to be a substantial amount of information available for cities throughout the state.

The ***Material Characterization*** screens require the most information that, at this time, is the least well-defined for Alabama materials. Development of a catalogue or GIS presentation of various material properties for Alabama will require the most effort on the part of the DOT. It is strongly suggested that ALDOT focus on developing guidelines for material property inputs for pavement design.

The ***Output*** for the 2002 Design Guide (publicly released version) indicate that the results consider the percent of heavy vehicles and the thickness of the HMA layer in the prediction of pavement distresses. However, the current models are insensitive to the base thickness and the type of subgrade. These limitations need to be explored very thoroughly before the 2002 Design Guide is used by ALDOT.

TABLE OF CONTENTS

TABLE OF TABLES

TABLE OF FIGURES

CHAPTER I. INTRODUCTION

The American Association of State and Highway Officials (AASHO) Committee on Design published the first Design Guide in 1961. The procedure was based on the results of the AASHO Road Test that was conducted in Ottawa, Illinois in the late 1950s to early 1960s. This design method introduced a road user definition of pavement failure rather that one based upon strict structural failure concepts (i.e. cracking and deformation). The idea was that roads are to function safely and smoothly. New concepts introduced by AASHO included serviceability and performance. The design equations developed for flexible pavements at the AASHO Road test were based upon an analysis of the effect of structural design and loading upon the performance of the flexible-pavement test sections (Yoder & Witczak, 1975). The developments from the AASHO Road Test led to the original American Association of State Highway and Transportation Officials (AASHTO) Design Guide.

The first AASHTO Design Guide was published in 1972 and was purely empirical. The empirical approach is based on the results of experiments or experience; it is the observed. The 1986 AASHTO Guide for Design of Pavement Structures, revision of the original design guide, was published in two volumes. Volume 1 of the Guide included design concept changes as to: pavement design and management, reliability, load equivalency values, resilient modulus, and layer coefficients. This Guide included an overview of the mechanistic-empirical design procedure, but did not incorporate the procedure. The term mechanistic refers to the mechanical properties of materials and involves the calculation of responses such as stress, stains, and deflections.

Volume 2 of the 1986 Guide was documentation of the research and development for Volume 1 (AASHTO, 1986).

The 1993 Design Guide was a second revision, which included an overview of the mechanistic-empirical design procedure. Further, it listed numerous benefits with developing and implementing the procedure. The many potential advantages of a mechanistic-empirical procedure led the development of the forthcoming Guide for Design of New and Rehabilitated Pavement Structures. The 2002 Design Guide is based on procedures that use existing mechanistic-empirical concepts, including a methodology for calibration, validation, and adaptation to local conditions; and, it is accompanied by computational software. For flexible pavements, the 2002 Design Guide incorporates mechanistic-empirical relationships to limit distresses (i.e. fatigue cracking, rutting, and thermal cracking) and ride quality (IRI) through adequate structural design.

Research Program

Objectives

The preliminary objectives of this research were to:

- Characterize the fundamental changes between the 1993 and the 2002 Design Guides.
- Catalogue and assess the inputs required to utilize the 2002 Design Guide.
- Evaluate the beta version (2002 DG) and the publicly released (post 2002) for flexible pavement design.

Scope

The objectives of the project were to evaluate the 2002 Design Guide user-friendliness, information needed to be collected by ALDOT to run the program, and the reasonableness of the output for flexible pavement design. The 2002 beta version of the software was used to complete pavement designs that considered varying HMA layer thickness (3 levels), thickness (2 levels) of a single crushed aggregate base, subgrade type (2 levels), and traffic (3 levels). Prior to conducting the pavement designs, each of the software input screens were evaluated for information needed for the 2002 Design Guide as compared to the 1993 Design Guide. Once a listing of the information needed to run the program was developed, variables such as traffic growth, percent of traffic in design lane, and material layer moduli were standardized for all runs. The output, in the form of predicted pavement distresses, was evaluated for reasonableness. The ALDOT Traffic Statistics website was used to define a number of the traffic input parameters. Material properties, such as resilient modulus, were selected based on typical values correlated with traditional properties (e.g. California Bearing Ratio). The beta version of the program was consistently unreliable with numerous programming problems. A thorough literature review of the equations selected for use in the program was conducted to establish whether the problem with the output could be the model(s) used or simply "bugs" in the programming code. A second evaluation of the flexible pavement design software was conducted once it was released to the public to determine if the program, as released, produced

CHAPTER II. LITERATURE REVIEW OF MODELS BACKGROUND

The major change between the 1993 and 2002 Design Guide is the new Design Guide's emphasis on estimating pavement distress(es) using mechanistic equations. Mechanistic design procedures provide a scientific basis of relating the mechanics of structural behavior to loading (National Highway Institute [NHI], 1999). In other words, pavements can be modeled as multi-layered elastic or visco-elastic structures on elastic or visco-elastic foundations. Using this assumption, it is possible to determine the critical stress and strain from deflection at any point within or below the pavement structure.

Empirical design is based on experiments or experience. It generally requires observations to be made in order to establish the relationships between the variables and the outcomes of the trials. An advantage of empirical procedures is that it is based on some actual data. Numerous complex factors may influence pavement performance, and mechanistic methods will not accurately model these factors. Thus, empirically based procedures are used to calibrate the models with observations of performance. The disadvantage is that empirical procedures are limited to original test conditions (NHI, 1999). Due to the limitations of both procedures, a mechanistic-empirical (M-E) design procedure evolved (AASHTO, 1993). M-E design procedures translate the analytical calculations of pavement response [mechanistic] to performance [empirical], which are the physical distresses (i.e. fatigue cracking, thermal cracking, and rutting) (AASHTO, 1993).

M-E design is robust because it combines the factors of mechanical modeling and performance observations in determining the required pavement thickness for a set of design conditions. M-E methods rely on mechanistic pavement modeling to adapt to new design conditions (i.e. heavier loads, new pavement materials, and environmental conditions) (Timm, Newcomb, & Birgisson, 1999). It is hypothesized that M-E design procedures will model pavements more precisely than that of traditional empirical equations that are used for flexible pavements. Thus, yielding several benefits, which include:

- Improved reliability for design, which decreases the conservatism built into design,
- Ability to predict specific distress types,
- Ability to extrapolate data from limited field and laboratory results,
- Improved pavement performance prediction,
- Consideration of new loading conditions,
- Better utilization of available materials,
- Improved procedures to define material properties,
- Better defined properties of existing pavement layers,
- Accommodation of environmental effects, and
- Consideration of aging effects.

Developing an M-E design procedure requires a pavement response model, material characterization, load characterization, pavement distress and transfer functions, and failure criteria. These basic components of the M-E design process are shown in Figure 2-1. Material properties, initial layer thickness, and load configurations are input into a pavement model, which calculates stresses and strains at critical locations. The stresses and strains are utilized to forecast the number of loads until failure for each condition. The number of expected loads must also be determined. The process iterates for each seasonal condition and load condition, and then damage is computed through a ratio of incremental damage to terminal damage. Damage exceeding unity indicates failure. If failure occurs, the layer properties and thickness are adjusted so that unity is not exceeded. When damage is much less than unity, the pavement has been over designed. If the damage is near but not exceeding unity the design thickness has been achieved (Timm et al., 1999).

Some agencies realized the advantages of an M-E design process and made moves to adopt this form of design procedure. Some of the notable agencies include the Washington State Department of Transportation (DOT), Illinois DOT, Minnesota DOT, and South Africa (Timm et al., 1999). Other equations evaluated are from Monismith, Asphalt Institute, and Shell (Timm et al., 1999; Ali & Tayabji, 1998). Similar steps were used in the development of each of the agencies' M-E design processes. The differences in the basic components will be shown in the following sections.

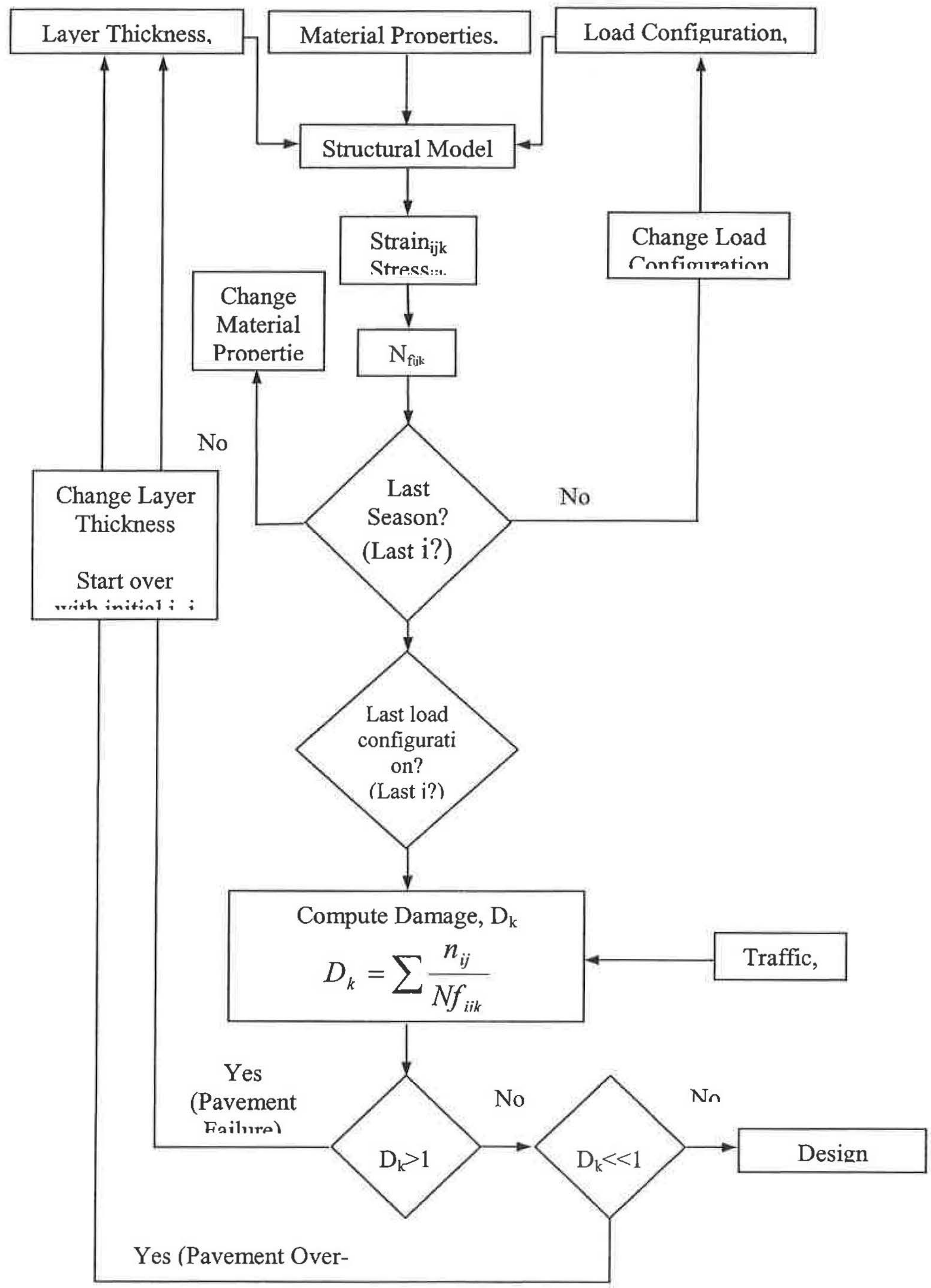


Figure 2-1. Mechanistic-Empirical Design Flowchart (after Timm et al., 1999)

Pavement Response Model

A pavement response model is a critical part of the M-E design process. This component is used for calculating the engineering responses (i.e. stress, strain, deflection) of a pavement when loads are applied (Timm et al., 1999). The first pavement model was a one-layer system developed by Boussinesq. These equations were originally developed for a homogeneous, isotropic, linearly elastic material. Burmister extended Boussinesq's one layer theory into a multiple layer system. Certain essential considerations were made regarding the conditions of the individual layers and the conditions at the layer boundaries. The top layers were assumed to be homogeneous, isotropic, elastic, and to extend infinitely in the horizontal direction, but finitely in vertical direction. The bottom layer was considered infinite in the horizontal and vertical direction. Burmister also assumed that the layers were in continuous contact and that the surface layer was free of shear and normal stresses outside the loading area (Yoder, & Witczak, 1975). The work of Boussinesq and Burmister later gave way for the development of layered elastic analysis models.

Layered elastic analysis programs have been widely utilized in flexible pavement analysis. Many computer models currently exist for use in pavement analysis. Washington State DOT used CHEVNL, which was originally developed by Chevron Research Company (Pierce, Jackson, & Mahoney, 1993). The program allows for only one uniform circular load applied vertically to the pavement. CHEVNL can analyze up to 15 layers of which all have finite thickness, except for the semi-infinite bottom layer. Each layer extends infinitely in the lateral direction, and the surface of the top layer sustains zero shear force. Vertical load and contact pressure are input, which allows the program to calculate the load radius. The material properties needed for each layer are: modulus, Poisson's ratio, and layer thickness. Cylindrical coordinates (r, z) are used to express distances, where r equals zero and extends through the center of the load. 30 radial and 30 vertical locations may be selected for analysis. Some of the operating characteristics are as follows (Timm et al., 1999):

- Up to 15 layers may be incorporated into the program.
- Any modulus value may be assumed for materials.
- Poisson's ratio may be any value other than one.
- The mathematics is simple and self-contained.
- The effects of multiple wheel configurations must be computed outside the program by utilizing superposition principles.

The Illinois DOT utilized a finite element (FE) based analysis program, ILLI-PAVE. Unlike layered elastic analysis programs, FE programs are able to model non-linear pavement systems more precisely by accounting for the change in modulus within each pavement layer. FE based analysis is very complex; thus, there is a significant increase in computation time (Timm et al., 1999). In ILLI-PAVE, principal stresses in the unbound layers (i.e. base, subbase, and subgrade) are modified at the end of each iteration so that the Mohr-Coulomb failure criteria are not exceeded (Thompson, 1987). Also, the program represents wheel loads as a single interior circular area where edge loads of dual wheels may not be considered directly (Timm et al., 1999).

The Minnesota DOT used WESLEA (Waterways Experiment Station Layered Elastic Analysis) to calculate pavement response. WESLEA has the ability to analyze up to five pavement layers, which all may extend infinitely in the horizontal direction. The last layer is semi-infinite in the vertical direction. The user has to indicate Poisson's ratio, modulus, thickness, and interface conditions for each layer. The loading conditions must also be specified. Up to 20 uniform, circular loads may be entered. Each of the loads must be specified in terms of load magnitude, radius, and x, y coordinates. Lastly, the user enters evaluation points. Up to 50 evaluation locations may be specified in terms of layer number and x, y, z coordinates (Timm et al., 1999).

South Africa uses a static, linear elastic multilayer analysis program for its structural analysis (Theyse, De Beer, & Rust, 1996).

Material Characterization

Material characterization is extremely important to the M-E design process. The values of Poisson's ratio and the layer modulus must be determined from laboratory testing, field measurements (nondestructive testing), or assumed. Environmental conditions affect the layer modulus (Timm et al., 1999). The modulus of asphalt concrete depends on material characteristics, loading conditions, and temperature. Unbound materials' modulus values depend on stress states, dry density, and moisture content, degree of saturation, gradation, load duration, and frequency (Pierce et al., 1993).

Poisson's Ratio

Poisson's ratio is a material response parameter that represents the ratio of transverse strain to axial strain when the material is loaded. Since Poisson's ratio has been difficult to measure, its values are usually assumed. Poisson's ratio is represented by the following equation:

$$\nu = \frac{\varepsilon_t}{\varepsilon_a} \tag{2.1}$$

Where: ϵ_t = transverse strain

ϵ_a = axial strain.

Layer Modulus

Layer modulus refers to the stiffness that a material exhibits when exposed to loading or displacement. Elastic or Young's modulus and resilient modulus are important for M-E design (Timm et al., 1999). Elastic modulus is the slope of the stress-strain curve that results when either load or displacement is applied to the material. The elastic modulus, E, is defined by the following equation:

$$E = \frac{\sigma}{\varepsilon} \tag{2.2}$$

Where: σ = stress due to applied load within proportional limit

ϵ = corresponding strain.

Resilient modulus is commonly referred to as the measure of resilient behavior of a material when subjected to load or displacement and can serve as a proxy for elastic modulus in linear elastic models. The resilient modulus, M_R, is defined by:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2.3}$$

Where: σ_d = applied deviator stress

ϵ_r = recoverable strain.

Load Characterization

Load characterization affects pavement distresses; thus, it is important to pavement design. Some agencies use weigh in motion (WIM) stations to determine traffic loading; however, most agencies adopted the equivalent single axle load (ESAL) approach to characterize traffic loads. ESAL is a term derived from the AASHO road test and is defined as a 80 kN single axle with dual tires inflated to 485 kPa. Using the ESAL approach, other types of axles and axle weights are empirically converted to an ESAL so that they cause equivalent damage. The equivalency concept essentially converts traffic streams into a single number for use in design (Timm et al., 1999). South Africa, on the other hand, uses a standard design load of 40 kN dual wheel load at 350 mm spacing between centers and a uniform contact pressure of 520 kPa for load characterization (Theyse et al., 1996).

Although the ESAL approach is typically used, there are numerous disadvantages. One disadvantage is that the ESAL is empirically based. Further, the ESAL converts traffic to an axle type that might not accurately represent pavement damage when the opportunity to precisely model individual axle types exists. M-E design, however, relies on mechanical modeling. Furthermore, M-E design has the ability to adapt to new traffic load conditions by dissection the load into its components: weight per tire, tire pressure, and wheel spacing (Timm et al., 1999).

Pavement Distress and Transfer Functions

The three major modes of distress for HMA pavement are fatigue cracking, permanent deformation (rutting), and thermal cracking. These distresses lead to a reduction in the serviceability of asphalt concrete pavements (Monismith, 1992). The distresses are discussed below.

Fatigue Cracking

Fatigue cracks are a series of interconnected cracks that usually occur when the pavement has been trafficked to the limit of its fatigue life by repetitive axle load applications and results in the fatigue failure of the HMA surface/base mixtures. Figure 2-2 illustrates fatigue cracking. "Bottom-up" fatigue cracking, also called alligator cracking, occurs when cracks initiate at the bottom of the asphalt surface or stabilized base where the tensile stress and strain are high.

Fatigue cracks may also start at the surface and migrate downward in a mode known as "Top-Down" fatigue cracking. It has been suggested that this surface cracking is due to the tensile strains in the surface of the asphalt cement (Brown, Kandhal, and Zhang, 2001). Once the damage initiates at the critical location and as time goes by, traffic repetition causes the cracks to propagate through the entire bound layer (NCHRP, 2002).

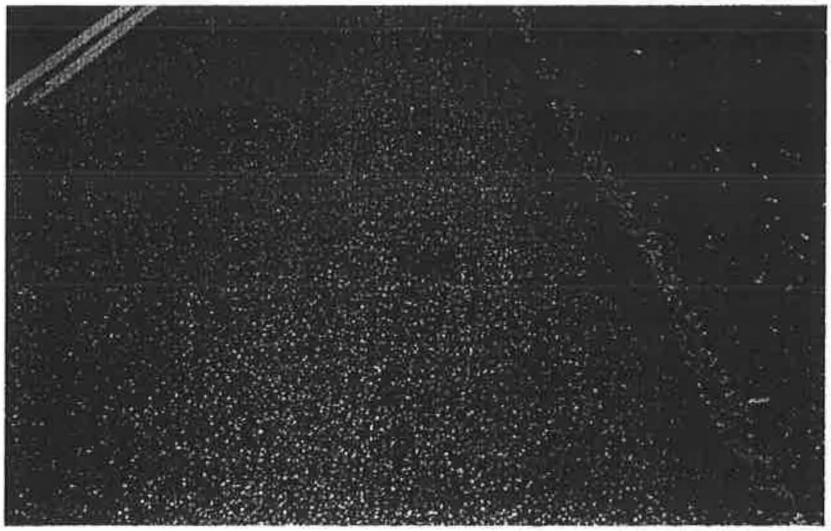


Figure 2-2. Fatigue Cracking

Transfer Functions: Fatigue cracking transfer functions are important to M-E design. Many agencies have evaluated the functions and some examples are as follows.

Monismith: According to Monismith, tensile strains at the bottom of the surface asphalt layer must be determined in order to predict fatigue. The following laboratory model related the number of loadings until failure to the tensile strain and modulus of the asphalt bound material (Monismith & Epps, 1969):

$$\log N_f = 14.82 - 3.291 * \log\left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 * \log\left(\frac{E_{ac}}{1000}\right) \tag{2.4}$$

Where: N_f = number of loads until failure

ϵ_t = tensile strain at the bottom of the surface asphalt layer

E_{ac} = modulus of the asphalt bound material (psi)

Washington State DOT: Washington State's model was based on Monismith's laboratory model, but with the laboratory relationship shifted to represent a field prediction. The equation is shown below (Mahoney & Pierce, 1996):

$$N_{field} = (N_{lab})(SF) \tag{2.5}$$

where: N_{lab} = relationship from laboratory data
$= 10^{(14.82-3.291\log\varepsilon_t-0.854\log E_{ac})}$

SF = shift factor; which can range from approximately 3 to 10, depending on asphalt cement (AC) thickness, ESAL level, climate, and construction quality.

Illinois DOT: Illinois DOT's model used a strain based fatigue algorithm in their design procedure. The relationship is based upon the consideration of mixture composition factors, field calibration studies, and split strength characteristics. The Illinois DOT fatigue transfer function is of the form (Thompson, 1987):

$$N_f = 5*10^{-6}\left(\frac{1}{\varepsilon_t}\right)^{3.0} \tag{2.6}$$

Minnesota DOT: Minnesota DOT utilized the Illinois DOT's equation because it was the most accurate in predicting the fatigue cracking observed at the Minnesota Road Research Project (Mn/ROAD). However, refinements of the coefficients were needed to produce a damage value of one. The adjusted equation is (Timm et al., 1999):

$$N_f = 2.83*10^{-6}\left(\frac{10^6}{\varepsilon_t}\right)^{3.206} \tag{2.7}$$

where: N_f = number of loads until failure (Typical definition of failure, 10% cracking)

South Africa: According to Theyse et al. (1996), South Africa used a strain-based algorithm to define fatigue cracking. The South African transfer function considers reliability and fatigue crack initiation. A shift factor is used to convert the crack initiation to the total fatigue life after surface cracks appear. Asphalt depth should be considered in the determination of the shift factor. The South African fatigue algorithm is as follows (Theyse et al., 1996):

$$N_f = 10^{A\left(1-\frac{\log \varepsilon_t}{B}\right)} \tag{2.8}$$

where: N_f = number of repeated loads until crack initiation (A shift factor converts the crack initiation life to the total fatigue life after surface cracks appear on the road (Theyse et al., 1996).

ϵ_t = tensile microstrain at the bottom or in the asphalt layer

A, B = regression constants

Asphalt Institute: The Asphalt Institute established the following equation (Ali & Tayabji, 1998):

$$N_f = 0.0796 * \varepsilon_t^{-3.291} * E_{ac}^{-0.854} \tag{2.9}$$

where: N_f = number of loads until failure (Definition of failure, 50% cracking (NHI, 1999)

E_{ac} = dynamic modulus of elasticity of asphalt concrete (psi)

Shell: The Shell equation is similar to that of Asphalt Institute transfer function. The equation is (Ali & Tayabji, 1998):

$$N_f = 0.0685 * \varepsilon_t^{-5.671} * E_{ac}^{-2.363} \quad (2.10)$$

where: N_f = number of loads until failure (Definition of failure, 20% cracking (NHI, 1999))

Rutting

A rut, shown in Figures 2-3 and 2-4, is a surface depression that occurs in the wheel paths and is usually accompanied by pavement uplift along the sides and is due to permanent deformation in any of the pavement layers or subgrade. Significant rutting can lead to structural failure of the pavement (Shahin, 2002). Rutting is caused by the lateral movement of material due to traffic load, and it develops gradually as the number of load repetitions accumulates.

- Rutting in the HMA only is a function of the shear stresses caused by the traffic loads exceeding the shear strength of the mix in the upper approximately 100 mm (4 in).
  - Some HMA properties that increase the rutting potential in the upper layers include excessive asphalt binder content, insufficient aggregate structure, or too low a viscosity asphalt binder.
- Rutting that extends into deeper layers is due to an inadequate design (the goal of the original AASHTO design is to protect the subgrade).
  - Could be an inadequate initial design or an unanticipated increase in traffic loads

The use of excessive asphalt is the most common cause for this phenomenon. If too much asphalt cement is contained in the mix, there is a loss of internal friction between aggregate particles and results in the loads being carried by the asphalt cement rather than the aggregate structure (Roberts et al., 1991).

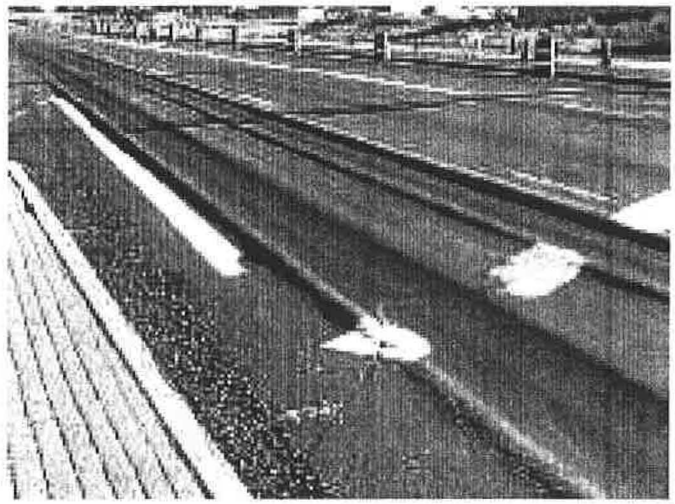


Figure 2-3. Example of Severe Rutting

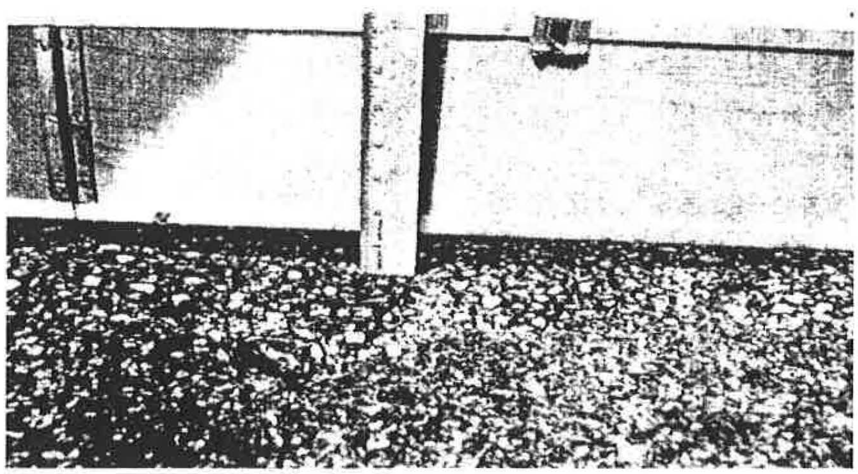


Figure 2-4. Close-up View of Rutting

Pavement materials sustain permanent deformation under a given set of material, load and environmental conditions, in three stages: primary, secondary, and tertiary (NCHRP, 2002). The three stages are described below and illustrated in Figure 2-5.

- ***Primary Stage***: high initial level of rutting, with a decreasing rate of plastic deformations, primarily associated with volumetric change.

- ***Secondary Stage***: small rate of rutting with a constant rate of change that is also associated to volumetric changes; however, shear deformations increase at an increasing rate.

- ***Tertiary Stage***: high level of rutting mainly associated with plastic (shear) deformations under no volume change conditions (i.e. the mixture densifies to a point of low air voids during secondary compaction, becomes unstable and deforms plastically) (Federal Highway Administration, 1997). In this stage, the rate of deformation seems to accelerate until complete failure occurs.

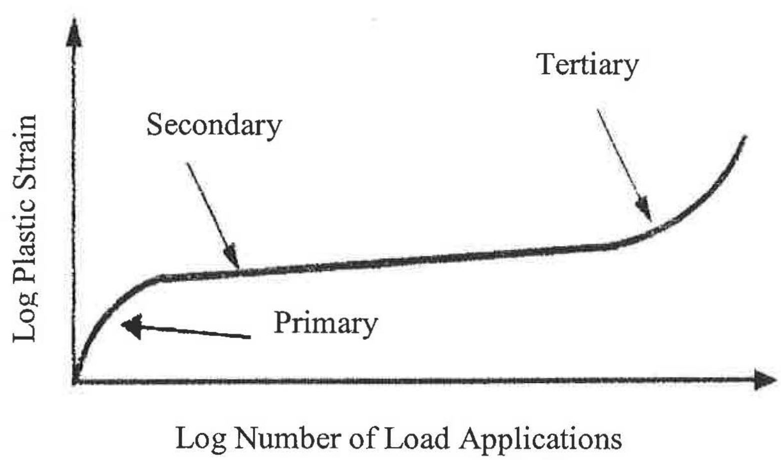


Figure 2-5. Three Stages of Rutting (National Highway Institute [NHI], 1997)

Rutting may occur in any part of the pavement structure; however, it is generally characterized empirically by the vertical compressive strain (ϵ_v) at the top of the subgrade. Thus, most transfer functions that characterize rutting involve ϵ_v.

Washington State DOT: The Washington State equation is (Mahoney & Pierce, 1996):

$$N_f = \left[\frac{1.05*10^{-2}}{\varepsilon_v}\right]^{4.4843} \tag{2.11}$$

where: N_f = loads to failure (Allowable rut depth, 12.7mm)

Minnesota DOT: The Minnesota rutting transfer function is (Timm et al., 1999):

$$N_R = 5.5*10^{15}\left(\frac{1}{\varepsilon_v}\right)^{3.949} \tag{2.12}$$

where: N_R = number of loads to cause a 20 mm depth rut.

South Africa: The South African rutting equation accounted for the vertical strain at the top of the subgrade, road category (reliability), and terminal rut condition (Theyse et al., 1996):

$$N_r = 10^{(A-10\log\varepsilon_v)} \tag{2.13}$$

where: N_r = number of loads until failure of the subgrade

A = regression constant that depends on road category and terminal rut condition.

Mainly rut depths define the coefficient "A". Terminal rut depths of 10 mm and 20 mm correspond to an "A" approximately equal to 33.5 and 36.5, respectively (Theyse et al., 1996).

Asphalt Institute: The Asphalt Institute has the following rutting prediction model (NHI, 1999):

$$N_f = 1.365x10^{-9}(\varepsilon_v)^{-4.477} \tag{2.14}$$

where: N_f = Number of allowable load applications (Allowable rut depth, 13 mm (0.5 in))

ε_v = Vertical compressive strain on top of the subgrade

Shell: Shell developed the following rutting prediction equation (NHI, 1999):

$$N_f = 6.15x10^{-7}(\varepsilon_v)^{-4}(50\%\,\mathrm{Re}\,liablity) \tag{2.15}$$

where: N_f = Number of allowable load applications (Allowable rut depth, 13 mm (0.5 in))

ε_v = Vertical compressive strain on top of the subgrade

Thermal Cracking

Thermal cracks are non-load associated transverse cracks that can be caused by shrinkage of the HMA surface due to low temperatures, hardening of the asphalt, and/or temperature cycling (NCHRP, 2002, February). Thermal cracks usually initiate at the top of the HMA and propagate downward through the layer. When the temperature decreases and the pavement cools and tries to contract, the contraction is restrained.

Internal tensile stresses accumulated in this process result in a series of cracks when the tensile stress exceeds the tensile strength of the AC. An example of thermal cracking is shown in Figure 2-6. This distress is critical to pavement performance because cracking of the pavement surface could quickly lead to a dysfunctional pavement, and deterioration of the pavement structure may be accelerated by the introduction of unwanted moisture into the base courses and subgrade (Shen & Kirkner, 2001).

According to Roberts et al (1991), binder properties such as stiffness, cement consistency (penetration at 77°F), and temperature susceptibility play a significant role in thermal cracking. HMA mixes that have high stiffness modulus at low temperatures have a tendency to crack. If the asphalt cement stiffness is decreased by selecting a better binder, cracking may be minimized.

Figure 2-6. Thermal Cracking

The prediction of thermal cracking is a complicated process. Models for predicting thermal cracking evolved from research conducted by Strategic Highway Research Program (SHRP) (Hilton & Roque, 1994). This model is utilized in the 2002 Design Guide. The pavement distress models are as follows:

- Stress Intensity factor model

$$K = \sigma(0.45 + 1.99 C_o^{0.56}) \tag{2.16}$$

where: K = stress intensity factor

σ = far-field stress from pavement response model at the depth of crack tip

C_o = current crack length

- Crack depth (fracture) model

$$\Delta C = A(\Delta K)^n \tag{2.17}$$

where: ΔC = change in crack depth due to a cooling cycle

ΔK = change in the stress intensity factor due to a cooling cycle

A, n = fracture parameters determined from testing

- Crack amount model

$$AC = \frac{\beta_1 N\left(Log \frac{C}{D}\right)}{\sigma} \tag{2.18}$$

where: AC = observed amount of thermal cracking

β_1 = regression coefficient determined through field calibration (found to be 381.4)

N() = standard normal deviation evaluated at ()

σ = standard normal deviation of the log of the depth of the cracks in the pavements (found to be 0.654)

C = crack depth

D = thickness of surface layer

Failure Criteria

Miner's Hypothesis is a simple damage model frequently used in M-E pavement analysis. It determines the portion of pavement life consumption and then calculates the sum of incremental damage until the limit for failure is reached (Timm et al., 1999). The equation for Miner's hypothesis is:

$$D = \sum_{i=1}^{k} \sum_{j=1}^{m} \frac{n_{ij}}{N_{ij}} \tag{2.19}$$

where: D = cumulative damage

i = seasonal condition

j = load configuration

n_{ij} = number of applied loads, j, in condition i

N_{ij} = number of allowable repetitions of load, j, in condition i

The life of the pavement has been consumed when damage value, D, is exceeds 1.

2002 Design Guide Equations

The discussion thus far has been a generic discussion of models. The Design Guide team used these previously developed models to select and refine equations for use in the 2002 Guide. The 2002 Design Guide utilizes the following distress equations:

$$\textit{AC Fatigue:}\quad N_f = \beta_{f1} k_1 \left(\frac{1}{\varepsilon_t}\right)^{k_2 \beta_{f2}} \left(\frac{1}{E}\right)^{k_3 \beta_{f3}} \tag{2.20}$$

where: N_f = Number of repetitions to fatigue cracking

ϵ_t = tensile strain at the critical location (in/in)

E = stiffness of the material (psi)

k_1, k_2, k_3 = equation coefficients

β_{f1}, β_{f2}, β_{f3} = calibration factors

$$\textit{AC Rutting:}\quad \log\left(\frac{\varepsilon_p}{\varepsilon_r}\right) = k_1\beta_{r1} + k_2\beta_{r2} \log T + k_3\beta_{r3} \log N \tag{2.21}$$

where: ϵ_p = plastic strain (in/in)

ϵ_r = resilient strain (in/in)

T = Layer Temperature (^{o}F)

N = Number of load repetitions

k_1, k_2, k_3 = equation coefficients

β_{r1}, β_{r2}, β_{r3} = calibration factors

Equation 2.20 illustrates that the Design Guide team came up with their own coefficients for fatigue cracking. However, it is of the common form for fatigue equations. The rutting equation, equation 2.21, is not of the common format of the other rutting equations presented previously in this chapter.

For performance prediction, the 2002 Design Guide considers permanent deformation in the asphalt concrete layer as well as in the entire pavement. The Design Guide uses an approach that models both the primary and secondary stages incorporating two major simplifications. First, the primary stage is modeled using an extrapolation of the trend of the secondary stage. Second, the tertiary portion is not taken into account. Permanent deformation tests leading up to this stage are very time consuming and difficult because of the many types of behavior and material properties that are involved; therefore, little research has been devoted to this type of analysis. True plastic shear deformations are not modeled within the system, actually, few, if any rutting prediction models incorporate this stage (NCHRP, 2002).

$$\textit{Thermal Fracture: } C_f = \beta_{t1} k_1 * N\left(\frac{\log C / h_{ac}}{\sigma}\right) \tag{2.22}$$

where: C_f = observed amount of thermal cracking (ft/500ft)

k_1 = regression coefficient determined through field calibration

N () = standard normal distribution evaluated at ()

σ = standard deviation of the log of the depth of cracks in the pavement

C = crack depth (in)

h_{ac} = thickness of asphalt layer

The 2002 Guide also utilizes the following equations:

Cement Treated Base (CTB) Fatigue: $$N_f = 10^{\left(\frac{k_1\beta_{c1} - \left(\frac{\sigma_s}{M_r}\right)}{k\beta_{c2}}\right)} \qquad (2.23)$$

where: N_f = Number of repetitions to fatigue cracking

k_1, k_2 = equation coefficients

β_{c1}, β_{c2} = calibration factors

σ_s = tensile stress (psi)

M_r = modulus of rupture (psi)

Subgrade Rutting: $$\delta_a(N) = \beta_{s1} k_1 \varepsilon_v h \left(\frac{\varepsilon_o}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^{\beta}} \right|$$

where: δ_a = permanent deformation for the layer

N = number of repetitions

ϵ_v = average vertical strain (in/in)

h = thickness of the layer (in)

k_1 = equation coefficient

β_{s1} = calibration factor

ϵ_o, β, ρ = material properties

ϵ_r = resilient strain (in/in)

AC Cracking: $$FC = C_3 + \left(\frac{C_4}{1 + e^{C_1 - C_2(Damage)}}\right) \qquad (2.25)$$

Table 2-1. Constants for Top Down and Bottom Up AC Cracking

	Top Down Cracking	Bottom Up Cracking
C1	6	6
C2	6	6
C3	50	15
C4	950	85

*No units were defined in the guide.
Currently, the Design Guide team did not define Damage.

Cement Treated Base (CTB) Cracking: $FC_{CTB} = C_1 + \left(\frac{C_2}{1+e^{C_3 - C_4(Damage)}}\right)$ (2.26)

where: C1 (CTB) = 6

C2 (CTB) = 6

C3 (CTB) = 15

C4 (CTB) = 85

International Roughness Index

In the , International Roughness Index (IRI) is used as a measurement of ride quality. IRI is defined as a standard roughness measurement related to measurements obtained by road meters installed on vehicles or trailers (Shahin, 2002). IRI is further defined as a property of the true profile (a two dimensional slice of the road surface, obtained along an imaginary line), and therefore it can be measured with any valid profiler. IRI summarizes the roughness qualities that impact vehicle response, and is applicable when a roughness measure is needed that relates to: overall vehicle operating cost, overall ride quality, dynamic wheel loads (essentially, damage to the road from heavy trucks and braking and cornering safety limits applicable to passenger cars), and overall surface condition. An IRI of 0.0 equals a profile that is perfectly flat (smooth). The common units for IRI are meters per kilometer (m/km) or millimeters per meter (mm/m). IRI is based on the average rectified slope (ARS), which is a filtered ratio of the accumulated suspension movement of a vehicle (in., mm., etc.) divided by the distance traveled by the vehicle during the measurement (mi, km, etc.). IRI is equivalent to average rectified slope (ARS) multiplied by 1,000 (National Asphalt Pavement Association [NAPA], N. D.). There is no theoretical upper limit to roughness; however, pavements with IRI values above 8 m/km are almost impassable except at reduced speeds (Sayers & Karamihas, 1998). The user has the option of choosing any or all of the distress criteria for a given pavement type; yet, pavement smoothness can only be verified provided all distresses are chosen for any given pavement type (NCHRP, 2002).

In the Guide, there are constants for IRI, but the equations were not included in the software. Further, depending on the type of structure, the constant definition changes.

IRI Flexible Pavements with Granular Base (GB)

where: C1 – Site Factor: C1(GB) = 0.0463

C2 – Transverse: C2(GB) = 0.00119

C3 – Rutting: C3(GB) = 0.1834

C4 – Fatigue: C4(GB) = 0.00384

C5 – Block: C5(GB) = 0.00736

C6 – Longitudinal: C6(GB) = 0.00115

Std. Dev. = 0.0387

IRI Flexible Pavements with ATB

C1 – Age: C1(ATB) = 0.0099947

C2 - Freezing Index: C2(ATB) = 0.0005183

C3 – Fatigue: C3(ATB) = 0.00235

C4 – Transverse: C4(ATB) = 18.36

C5 – Patching: C5(ATB) = 0.9694

Std. Dev. = 0.292

IRI Flexible Pavements with CTB

C1 – Fatigue: C1(CTB) = 0.00732

C2 – Rutting: C2(CTB) = 0.07647

C3 –TransverseC3(CTB) = 0.0001449

C4 – Block: C4(CTB) = 0.00842

C5 – Longitudinal: C5(CTB) = 0.0002115

Std. Dev. = 0.229

CHAPTER III. INITIAL EVALUATION OF THE 2002 DESIGN GUIDE (Beta Version)

The 2002 Design Guide requires users to provide inputs for Project Input, Traffic, Climate, Structure (specifically, Material Characterization), and Distress Potential areas. Each area will be discussed below. Output generated from Beta Version 0.088 of the 2002 Design Guide Version will also be discussed in this chapter. The input screens for the Guide are included in Appendix A.

Project Input Screens

The screens contained within project input are: 1) General Information, 2) Site/Project Identification, and 3) Analysis Parameters. The General Information screen allows the user to enter basic information such as project name, design life, construction months, and pavement type. In the Site/Project Identification screen, the user enters various information that will be helpful in identifying and documenting sites. Within the Analysis Parameters, the user inputs limitations on the distresses that are expected to influence pavement life and enters the analysis type (e.g. probabilistic or deterministic). Each input screen will be discussed in more detail in the following sections.

General Information

The first input screen that the user enters in the 2002 Design Guide Software is general information. This screen allows the user to assign a name to the project and input the design life, base/subbase construction month, pavement construction month, and the month open to traffic. The design life is the expected service life of the pavement without rehabilitation. The program predicts pavement performance over the design life (typically 20 years for new construction projects) starting with the month that the pavement is open to traffic (National Cooperative Highway Research Program [NCHRP], 2002). This prediction does not incorporate maintenance activities. The type of pavement, flexible or rigid, is then selected. Choosing the type of pavement is important because it relates to the distresses that must be considered in addition to the computational and performance models that are employed. All pavements with an asphalt concrete surface (new or rehabilitated) are treated as flexible pavements and those with a concrete surface are treated as rigid pavements (NCHRP, 2002). This document will focus on flexible pavements, and in this screen, the user may classify the design type: new, restoration, or rehabilitation.

Site/Project Identification

The Site Identification screen allows the user to enter the site location, project and section identifications, functional class (i.e. principal and minor arterials, collectors, or local routes and streets), station/mile post information, and traffic direction. The information entered in this screen will not affect the analysis or design processes; however, it may be useful when designing different sections on a large project site. This screen may be used for identification and documentation purposes (NCHRP, 2002); however, its usefulness will be dependent on the agency and its practices.

Analysis Parameters

In this screen basic criteria for performance prediction are determined. First the user must select the analysis type—either deterministic (i.e., 50% reliability) or probabilistic (i.e., choose desired reliability) solutions. If a probabilistic solution is preferred, the mean and statistical variability (variance) values of all input variables will be required. Alternatively, the deterministic solution simply requires the mean values. A significant amount of uncertainty and variability exists in pavement design and construction, in addition to the application of traffic loads and climatic factors of the design life. The design reliability is established through the simulation of many of these uncertainties and variability by establishing the potential error of selected design inputs and various models in the design procedure. Therefore, the designer must choose what limiting amount of distress can occur for a given level of design reliability (NCHRP, 2002). For instance, one criterion might limit the permanent deformation in the asphalt only to 0.25 inches. If the simulation process shows that the trial pavement design produces an excessive amount of rutting, then the trial design must be modified to reduce this deformation.

The 2002 Design Guide verifies the trial design against user input performance criteria. The user must set the limits of AC bottom up alligator cracking (fatigue cracking), chemically stabilized layer (fatigue fracture), AC thermal fracture (thermal cracking), permanent deformation (AC only and total), AC surface down cracking (longitudinal cracking), and terminal IRI (International Roughness Index). The Guide uses the terms AC and HMA interchangeably. Again the design must meet the performance criteria at a given level of reliability or it must be modified and iterated until the criteria are met. Once the designs meet the performance criteria, structurally and functionally, they can be further considered for other evaluations such as life cycle cost analysis (NCHRP, 2002). The previously mentioned distresses were described in Chapter II.

Traffic

Traffic data are extremely important in the design and analysis of pavement structures and are usually expressed in terms of equivalent single axle loads (ESALs). The 2002 Design Guide is based on mechanistic-empirical design procedures; thus, it uses axle load spectra as opposed to the ESAL approach, where load spectra are the percent of axles per axle weight category. This requires that wheel loads be characterized in order to perform mechanistic simulations; therefore, the traffic stream must be carefully characterized. .

Traffic Input Screens

The traffic input screen allows the user to enter two-way average annual daily truck traffic (AADTT), number of lanes in the design direction, percent of trucks in the design direction, percent of trucks in the design lane, and operational speed. Other inputs, which will be discussed later in this section, needed for these main categories are:

- Traffic volume adjustment factors
    - Monthly Adjustment
    - Vehicle Class Distribution
    - Hourly Truck Distribution
    - Truck Growth Factor
- Axle load distribution factors
- General traffic inputs
    - Number of axles/truck
    - Axle configuration

Three assumptions are made by the 2002 mechanistic design procedure concerning traffic data. Specifically, the axle load distribution by axle type and vehicle class (www.2002designguide.com):

1. Remains constant from year to year, but the vehicle class distributions can change from year to year.
2. Does not change throughout the day or over the week (weekday versus weekend and night versus day). However, the vehicle class or truck distributions can change over the time of day or day of the week.
3. Does not change from site to site within a specific region.

Within the traffic input screens the designer has the option of using specific or general input data for the design process. Per the 2002 Design Guide, the input levels are:

Level 1 - Provides the highest level of accuracy; thus, these inputs have the lowest level of uncertainty or error. This input level would generally be used for pavements with high traffic volume, or where there is dire safety or economic consequences of early failure. The inputs for this level would include site-specific axle load spectra data.

Level 2 - Inputs are at an intermediate level and would be similar to the procedures used with previous editions of the AASHTO Guide. The inputs for this level would include regional axle load spectra data and project-related volume and classification information.

Level 3 - Provides the lowest level of accuracy. The designer might utilize this level when there are minimal consequences of early failure (low volume roads). This level is usually used when the designer only has a value for AADTT. Traffic inputs would either be user selected default values or typical averages for the region.

Traffic must be distributed by direction and by lanes. The directional distribution factor (default value = 50%) accounts for the percentage of trucks in one direction of the total truck traffic population. The lane distribution factor accounts for the percentage of trucks of the total truck population in one lane and one direction. Table 3-1 shows the recommended default lane distribution factors. These default values are typically within the range recommended in the 1993 Guide (Table 3-2). The number of lanes, percent of trucks in the design direction, and percent of trucks in the design lane are significant because the heaviest loaded lane is critical for design. These factors depend on the area (urban or rural), the highway size (number of lanes), and possibly truck type.

Table 3-1. Recommended Lane Distribution Factors (www.2002designguide.com)

Number of Lanes	Default Values
1 lane	1.0
2 lanes	0.9
3 lanes	0.6
4 lanes	0.4

Table 3-2. 1993 Guide's Lane Distribution Factors (AASHTO, 1993)

Number of Lanes	Default Values
1 lane	1.0
2 lanes	0.8-1.0
3 lanes	0.6-0.8
4 lanes	0.5-0.75

Operational speed is directly correlated to loading frequency, and Figure 3-1 shows that frequency corresponds to the complex modulus of HMA materials, E*. Figure 3-1 can be expressed as the following equation:

$$Y = aX^b \tag{4.1}$$

where: $Y = E^*$

a = material constant (mix-specific)

X = frequency

b = material constant (mix specific)

This equation shows that mix stiffness (i.e., modulus) is dependent upon frequency; therefore, stiffness will be a function traffic speed. Further, axle loads applied at lower speeds can cause higher pavement damage than those at higher speeds (Hajek, 1995). Traffic speed also has an effect on rutting and fatigue cracking, as shown in Figure 3-2.

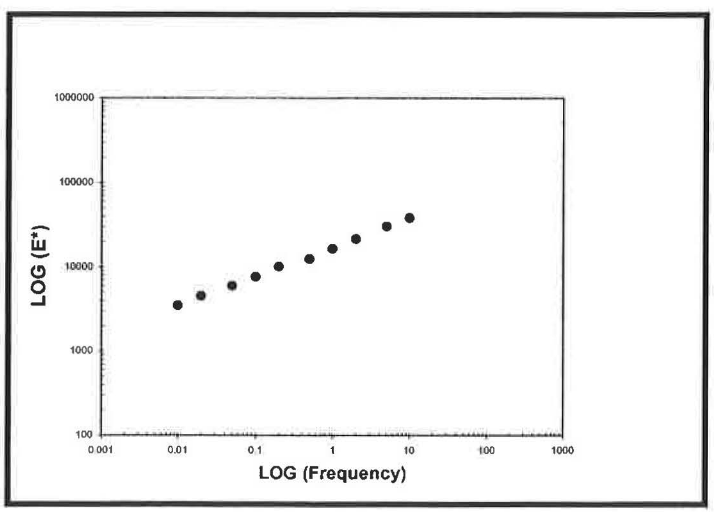


Figure 3-1. Example of Log (Frequency) vs. Log (E*)

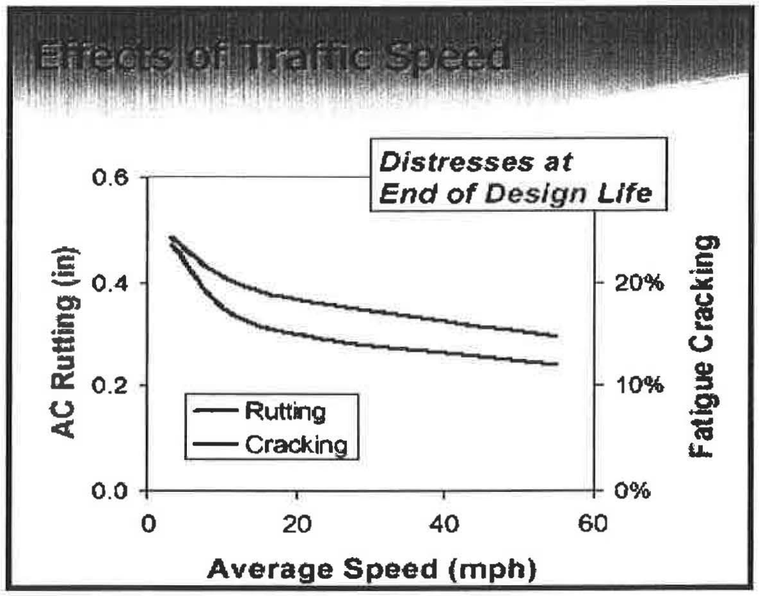
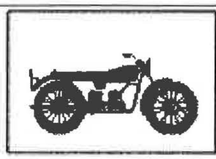
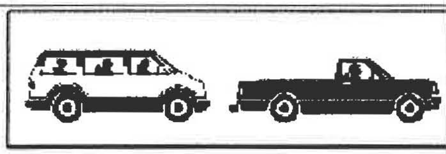
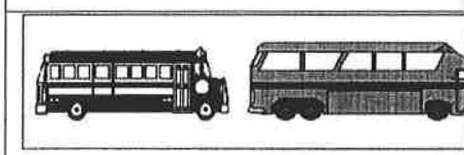
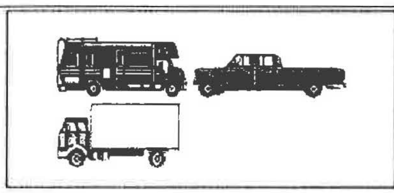
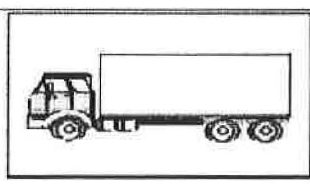
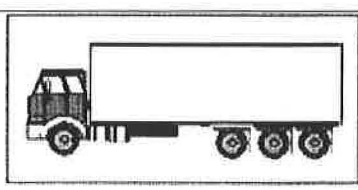


Figure 3-2. Example of Effects of Traffic Speed on AC Rutting and Fatigue Cracking (www.2002designguide.com)

Traffic Volume Adjustment Factors

In the 2002 Design Guide, there are thirteen vehicle classifications, consistent with the FHWA class system (Figure 3-3). The truck traffic distribution factor (TTDF), which is the percentage of the AADTT in classes 4 (buses) through 13 (7 or more axles, multi-trailers) are used in the design software. The monthly distribution factors are used to split the AADTT into monthly average daily truck traffic (MADTT) values/volumes. A safe assumption to use for the monthly adjustment, shown in a screen capture from the 2002 Design Guide in Figure 3-4, is that the same amount of traffic per vehicle class type occurs. However, if the volume of a certain class increases for a season or a single month, then the designer has the option of making the necessary adjustment.

	Vehicle Class	*Description*
	1	Motorcycles
	2	Passenger Cars (With 1- or 2-Axle Trailers)
	3	2 Axles, 4-Tire Single Units, Pickup or Van (With 1- or 2-Axle Trailers)
	4	Buses
	5	Tire Single Unit (Includes Handicapped-Equipped and Mini School Buses)
	6	3 Axles, Single Unit
	7	4 or More Axles, Single Unit

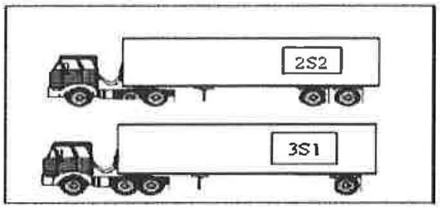
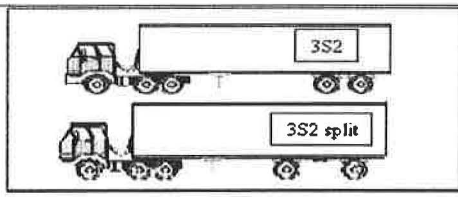
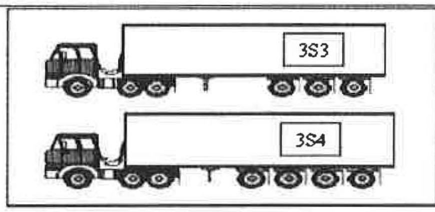
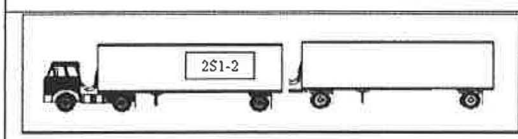
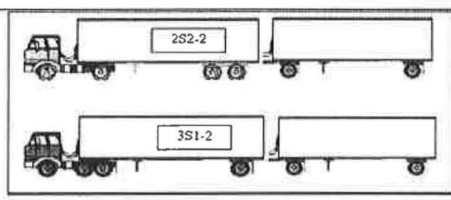
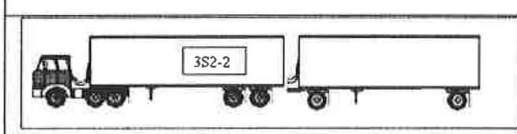
	8	3-4 Axles, Single Trailer
	9	5 Axles, Single Trailer
	10	6 or More Axles, Single Trailer
	11	5 or Less Axles, Multi-Trailers
	12	6 Axles, Multi-Trailers
	13	7 or More Axles, Multi-Trailers

Figure 3-3. FHWA Vehicle Classification (TxDOT, 09/2001)

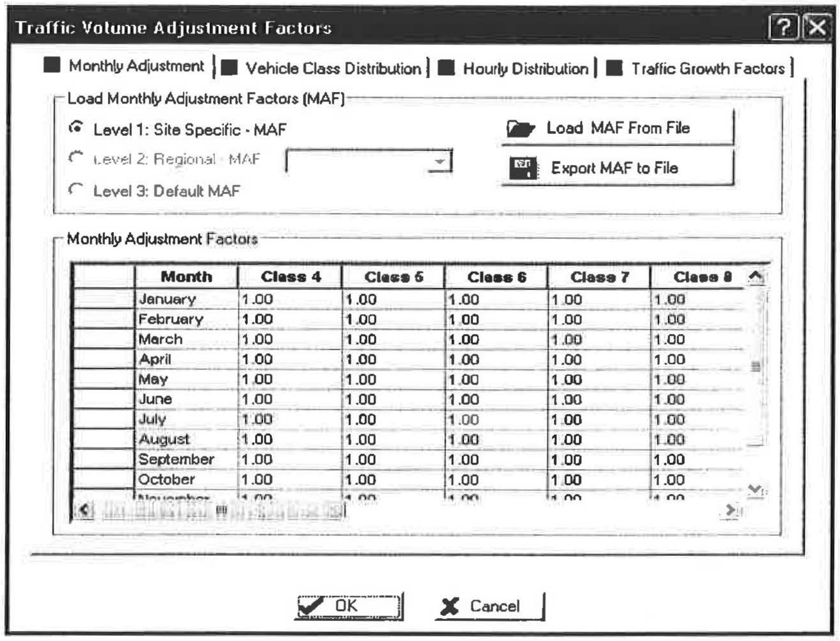
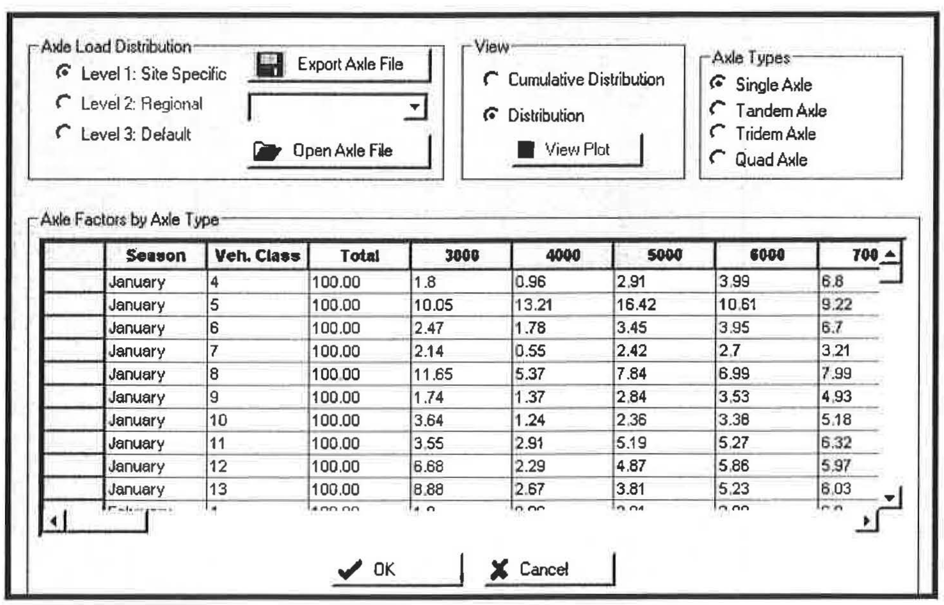


Figure 3-4. Example of Traffic Volume Adjustment Factors for Monthly Adjustment (NCHRP, 2002)

Vehicle class distribution is simply taking the average annual daily truck traffic (AADTT) and determining the percentage of trucks in each vehicle class. Hourly distribution factors distribute the monthly MADTT by the hour of the day, and assigns traffic beginning at midnight. The sum of both the AADTT distribution and the hourly distribution must equal 100%. The traffic growth factor is input for traffic prediction over the design life. The user has the option of entering no growth, linear growth, or compound growth along with a percentage. Growth plots may be obtained from this input in future versions of the software. Vehicle class distribution factors are required to view the effects of traffic growth (NCHRP, 2002).

Axle Load Distribution Factors

In the axle load distribution input screen, shown in Figure 3-5, the axle load distribution factors by axle type (single, tandem, tridem, and quad) are illustrated in a table. The user can view data as either a distribution or a cumulative distribution. A plot is expected to be available in future software versions. A percentage, ranging from 0 to 100, is input per vehicle class corresponding to the appropriate weight. These data are shown by season. An axle load distribution factor versus vehicle class plot is shown in Figure 3-6. The plot is of axle load distribution factors versus vehicle class for a 3000 lb single axle for the month of January. Figure 3-6 shows that the primary vehicle classes are classes five and eight.



Season	Veh. Class	Total	3000	4000	5000	6000	700
January	4	100.00	1.8	0.96	2.91	3.99	6.8
January	5	100.00	10.05	13.21	16.42	10.61	9.22
January	6	100.00	2.47	1.78	3.45	3.95	6.7
January	7	100.00	2.14	0.55	2.42	2.7	3.21
January	8	100.00	11.65	5.37	7.84	6.99	7.99
January	9	100.00	1.74	1.37	2.84	3.53	4.93
January	10	100.00	3.64	1.24	2.36	3.38	5.18
January	11	100.00	3.55	2.91	5.19	5.27	6.32
January	12	100.00	6.68	2.29	4.87	5.86	5.97
January	13	100.00	8.88	2.67	3.81	5.23	6.03

Figure 3-5. Example of Axle Load Distribution Factors for Single Axles (NCHRP, 2002)

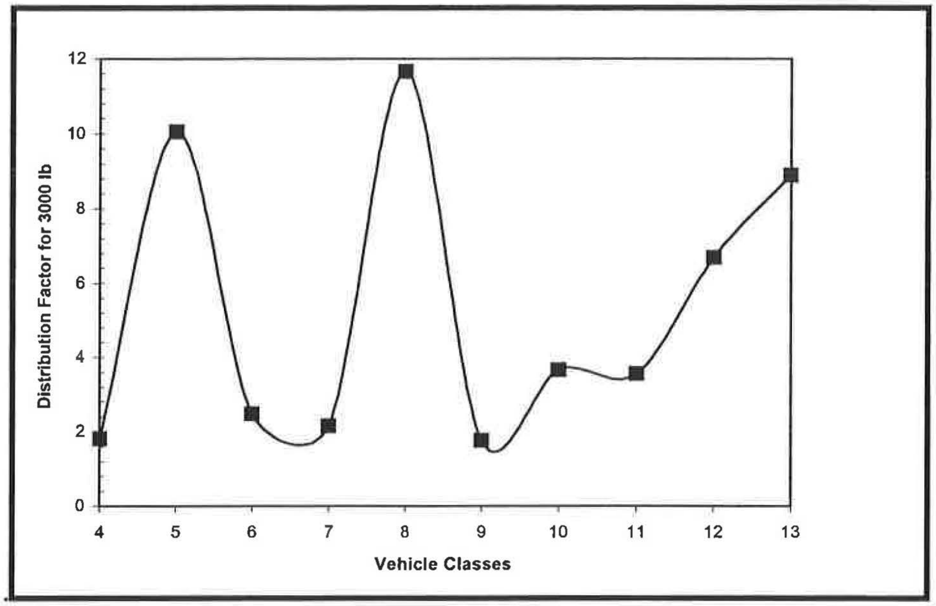


Figure 3-6. Example Plot of Axle Load Distribution Factors vs. Vehicle Class for a 3000 lb Single Axle for January (After 2002 Guide)

General Traffic Inputs

This input screen allows the user to account for lateral traffic wander by input of the mean wheel location, the traffic wander standard deviation, and the design lane width. The user also can input the number of axles per truck and axle configuration information. Default values are provided for tire and axle load data, but agencies will have the option of using site-specific values. The default values were developed from information contained in the LTPP database (www.2002designguide.com).

Traffic wander is the damaging effect of each axle extended over the lane. Traffic wander has a significant effect on rutting and fatigue cracking, as shown in Figure 3-7. Traffic wander standard deviation is a measure of how far from the mean wheel path tires travel in a lateral direction (Timm, 1996). Figure 3-8 is an illustration of wheel wander standard deviation. Data obtained from the Mn/ROAD research facility database for several days in March 1996 indicate a wheel wander standard deviation of 9.4 inches (Timm, 1996). Assuming that the Mn/ROAD data are representative of typical wheel wander, the default value of 9 inches in the 2002 guide for traffic wheel wander standard deviation is sensible.

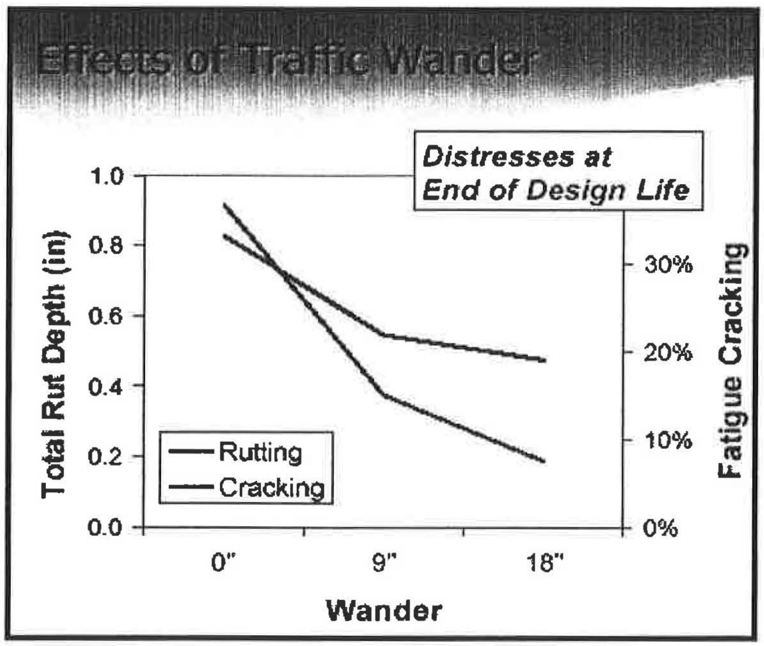


Figure 3-7. Example of Effects of Traffic Wander on Total AC Rutting and Fatigue Cracking (www.2002designguide.com)

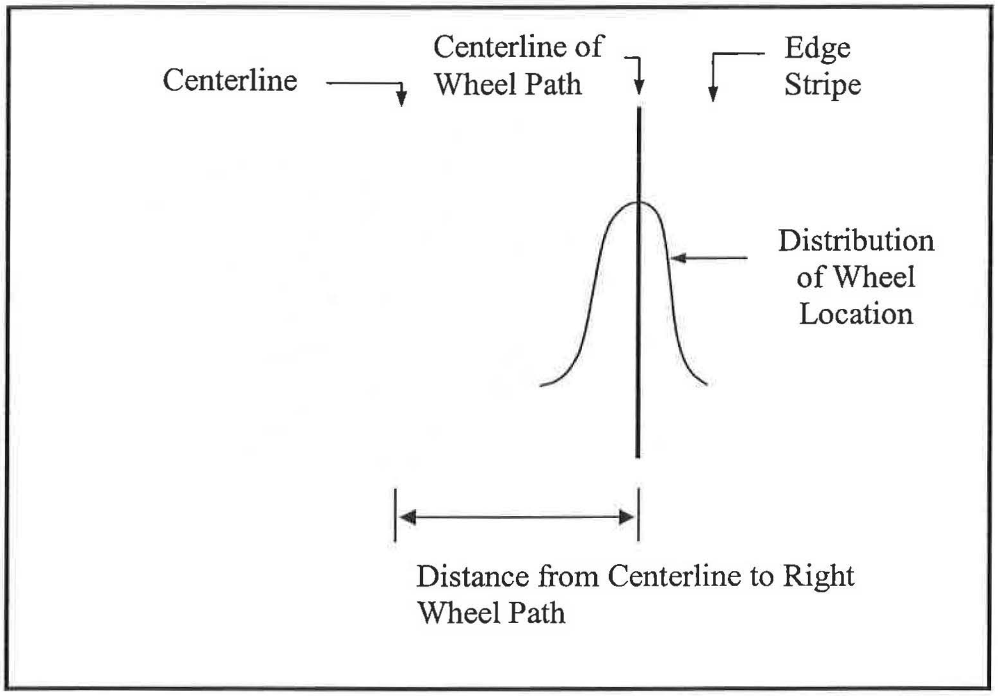


Figure 3-8. Illustration of Wheel Wander Standard Deviation

Number of axles/truck

Truck design has changed significantly through the years, and the number of axles and the type of axle are important factors in load carrying capacity of pavements. Since pavement failure occurs when load repetitions accumulate, traffic should be characterized by the variety of loads and by the number of repetitions of the loads over the pavement design life (Newcomb & Timm, 2001). The input for the number of axles/truck for each vehicle class is typically obtained from weigh-in-motion (WIM) or automatic vehicle classification (AVC) data. The input is determined by dividing the total number of trucks of each type by the total number of trucks weighed. Table 3-3 shows the default values used in the guide.

Table 3-3. Default values for Average Number of Axles/Truck (ERES, 2002)

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

Axle Configuration

In this input screen, average axle width, dual tire spacing, axle spacing, and tire pressure are entered. The default value for average axle width is 8.5 feet, and it is measured from edge to edge of the inside of tires. Dual tire spacing (default value = 12 in) is the center-to-center spacing of dual tires on the end of a single axle. Axle spacing (average default value = 50 in) is the spacing between axles in a tandem, tridem, or quad group. Dual tire spacing and axle spacing are illustrated in Figure 3-9. Tire pressure is a function of whether the tire is in single or dual configuration. Higher tire pressures increase the appearance of rutting and fatigue cracking in asphalt layers (Zhang, Leidy, Kawa & Hudson, 2000).

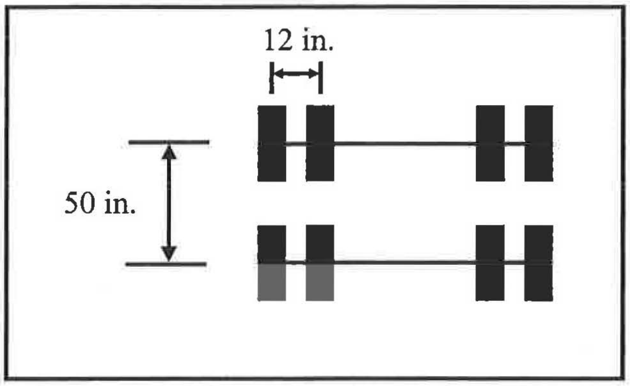


Figure 3-9. Dual Tire and Axle Spacing for a Tandem Axle

2002 Design Guide/DARWin Comparison

DARWin is a computerized version of the pavements design models presented in the 1993 AASHTO Guide for the Design of Pavement Structures (AASHTO, 1997). DARWin utilizes ESALs to characterize the traffic stream for pavement design. Equivalency factors are used to account for the different vehicle types, weights, and axle configurations and are used to determine ESALs. In DARWin, ESALS can be calculated using two methods: a simple procedure (based on ADT and other average inputs) or rigorous procedure (based on inputs for each vehicle classification). The 2002 Design Guide uses load spectra to characterize the traffic stream. Load spectra simply describe the many types of loads that a pavement experiences (Timm & Newcomb, 2002). Although ESALs and load spectra originate from the same information, as illustrated in Figure 3-10, more inputs are required to utilize the 2002 Design Guide than for the 1993 DARWin software. Table 3-4 provides a comparison of the two software programs.

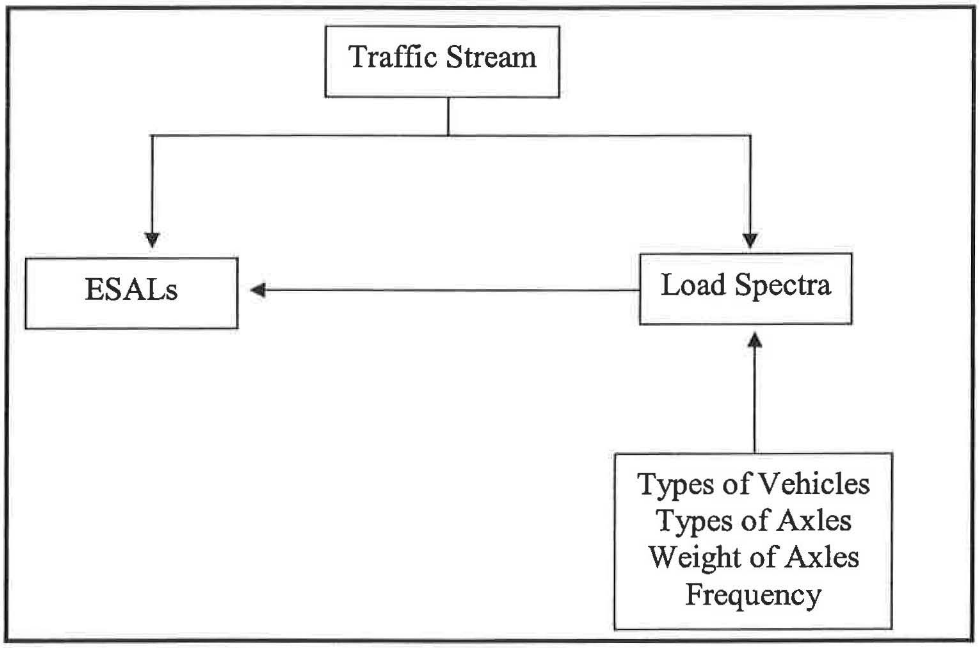


Figure 3-10. Illustration of the Origination of ESALs and Load Spectra

Table 3-4. Design Input Comparison

Inputs	AASHTO 1993	Design Guide 2002
Design Life	X	X
Two-way AADTT		X
Two-way Daily Traffic (ADT)	X	
Number of lanes in Design direction	X	X
Percent of trucks in design direction (%)	X	X
Percent of trucks in design lane (%)	X	X
Operational speed (mph)		X
Monthly Adjustment Factors		X
Vehicle Class Distribution		X
AADTT distribution by vehicle class		X
Growth Rate (%)	X	X
Axle load Distribution Factors		X
Mean Wheel Location		X
Traffic Wander Standard Deviation		X
Design lane width		X
Number of Axles per Truck		X
Axle Configuration		X
Average axle width		X
Dual tire spacing		X
Single Tire (psi)		X
Dual Tire (psi)		X
Tandem Axle		X
Tridem Axle		X
Quad Axle		X
Average axle spacing		X
Percent of Trucks		X
* *Simple Calculation*		
%Heavy Trucks (of ADT) FHWA Class 5 or Greater	X	
Average Initial Truck Factor (ESALs/Truck)	X	
Annual Truck Factor Growth Rate (%)	X	
Annual Truck Volume Growth Rate (%)	X	
* *Rigorous Calculation*		
Vehicle Class 1-12	X	
Percent of ADT	X	
Annual % Growth	X	
Avg. Initial Truck Factor (ESALs/Truck)	X	
Annual % Growth in Truck Factor	X	
Calculated Accumulated ESALs over Performance Period	X	
* Choose either Simple or Rigorous		

Climate

This section is a summary of the Development Of The 2002 Guide for the Design of New and Rehabilitated Pavement Structures [Part 2 – Design Inputs; Chapter 3: Environmental Effects] for NCHRP Project 1-37A, and it discusses the environmental conditions that have a significant effect on the performance of both flexible and rigid pavements. Factors such as precipitation, temperature, freeze-thaw cycles, and depth to water table affect pavement and subgrade temperature and moisture content, which, in turn, directly affects the load-carrying capacity of the pavement.

Temperatures in the HMA pavement, and moisture content in the pavement and subgrade are modeled using the Enhanced Integrated Climatic Model (EICM) software, which is integrated into the . The EICM predicts the previously mentioned factors in order to obtain a set of layer stiffness values. These stiffness values can be used in mechanistic modeling to calculate pavement responses (i.e., stresses and strains) in the structure. So, essentially, the EICM generates material properties on a seasonal basis into the design. The EICM software is linked to the design guide software as an independent module through interfaces and design inputs. The user should only be concerned about the location of the site with relevance to given weather stations as well as the average water table height for the proposed construction area.

EICM

The Enhanced Integrated Climate Model is the model used to predict climatic effects on the pavement. This model is a one-dimensional coupled heat and moisture flow program developed for the Federal Highway Administration (FHWA). The model is intended to predict or simulate the changes in the behavior and characteristics of pavement and subgrade materials based on an estimation of climatic conditions over several years of operation. The EICM has three major components. They are the Climatic-Materials-Structural Model (CMS), the Frost Heave and Thaw Settlement Model (CRREL Model), and the Infiltration and Drainage Model (ID).

For flexible pavement analysis and design, the following tasks are needed to account for the environmental effects:

- ***Task 1***: Evaluation of the resilient modulus of all unbound layer materials, at a reference condition; which is usually at or near the optimum water content and maximum dry density.
- ***Task 2***: Evaluation of the expected changes in moisture content from the initial or reference condition as the natural water content reaches the equilibrium condition for each season (part of the EICM output).
- ***Task 3***: Evaluation of the effect on resilient modulus due to changes in soil moisture content regarding the reference condition.
- ***Task 4: E***valuation of the effect of freezing on the layer resilient modulus (M_R).

- ***Task 5***: Evaluation of the effect of thawing and recovery from the frozen resilient modulus conditions.
- ***Task 6***: Utilization of time – varying resilient modulus values in the calculation of critical pavement response parameters at various points within the pavement system.
- ***Task 7***: Evaluation of temperature changes as a function of time for all HMA and unbound layers.

Tasks 2 through 7 are either performed inside the EICM Module, or EICM output is used in the performance of these tasks.

One of the important outputs required from the EICM Module is a set of adjustment factors for the unbound material layers. These factors account for the effects of environmental parameters and conditions such as moisture content changes, freezing, thawing, and recovery from thawing. This factor, denoted as F_{env}, varies with position within the pavement structure and with the time throughout the analysis period. The F_{env} factor is a coefficient that is multiplied by the resilient modulus at optimum water content (M_{Ropt}) to obtain M_R as a function of position and time.

Three additional important EICM outputs are the in-situ temperatures at the midpoints of each bound sublayer, the temperature profiles within the HMA and/or PCC layer for every hour, and the average moisture content for each sublayer in the pavement structure. These outputs of the EICM Module can be described on two levels: internal and external, and are summarized in the following paragraphs.

For internal output, the computational engine of the EICM determines values of volumetric water content (θ_w), and temperature at each node over time (the term node is not clearly defined in reference literature). The values of the θ_w are divided by the saturated volumetric water contents (θ_{sat}) to get values of degree of saturation, S. Without oscillations in the general input groundwater table and no cracks in the HMA layer, values of S are essentially values at a state of equilibrium, unless freezing or thaw recovery is occurring.

The values of S at equilibrium, along with values of degree of saturation at optimum (S_{opt}) conditions are used to calculate an adjustment factor for unfrozen conditions at each node. The output temperatures are used to signal freezing at a node. Also, an adjustment factor for the frozen condition is computed at each freezing node. Thawing normally follows freezing, as signaled by the rise in temperature above the freezing point. During the recovery period, material/type properties are used to compute the recovery ratio (RR) at recovering nodes. These RR values, in conjunction with reduction factors (RF) due to thawing are used to compute an adjustment factor (F_R) for recovering conditions at each recovering node.

For external output, the following outputs are generated by EICM for use by other modules:

- Resilient Modulus (M_R) as functions of position and time – Values of composite environmental adjustment factors are computed for every sublayer from the values at each node. The sublayering is previously defined by the EICM, and it is a function of the frost penetration depth, amongst other factors. These factors are sent to the Finite Element Analysis Module or the Linear Elastic Analysis Module, where they are multiplied by the optimum resilient modulus (M_{Ropt}) to obtain M_R as functions of position and time.

- Temperatures at the surface and at the midpoint of each asphalt bound sublayer – these values are put through statistical characterization for every analysis period. The mean, standard deviation, and quintile points are sent forward for use in the Fatigue Module and the Permanent Deformation Module.
- Values of hourly temperature at the surface and at a set depth increment (every inch) within the bound layers. These values are used in the Thermal Cracking Module.
- Volumetric moisture content – an average value for each sublayer is generated. This average value is used in the Permanent Deformation Module for the unbound materials.

Materials Characterization

This section is a synopsis of the Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures [Part 2 – Design Inputs; Chapter 2: Material Characterization] for NCHRP Project1-37A. Material Characterization is an important collection of properties that are incorporated into the 2002 Design Guide. The two categories are bound and unbound materials (bound: Asphalt, PCC, and cementitiously stabilized materials; unbound: non-stabilized granular base/subbase, subgrade soils, and bedrock.). The 2002 Design Guide divides these two categories into six groups: Asphalt Materials, PCC Materials, Cementiously Stabilized Materials, Non-Stabilized Granular Base/Subbase, Subgrade Soils and Bedrock. Numerous organizations have developed categorical classifications systems for their own specific purposes. Since no convenient functional grouping of material categorical classification required for use in mechanistic-empirical (M-E) evaluation and analysis of pavement systems is available, a material category grouping was developed for use in the 2002 Design Guide (ERES Consultants, 2001). There are six major material groups in the Guides categorical system; these are shown in Table 3-5.

Asphalt Materials is one of the more complicated groups due to the response and behavior of these materials being significantly influenced by temperature, time rate of load, the mixing process, and the degree of damage of the material in the design process (new versus rehabilitated). The cementitiously stabilized materials category includes a broad range of pozzolanic (chemical) reactive materials. They range from materials that slightly modify the plasticity characteristics of the original material to materials that have significant gains in stiffness, strength, and other key engineering properties that might approach the behavior of PCC materials. Unbound material characteristics relate to the fact that moduli of these materials may be greatly influenced by the stress state (non-linear) and the in-situ moisture content. Generally, coarse-grained materials become stiffer as the confining state of stress is increased, and clayey materials behave in an opposite manner. The bedrock category is important since its presence near the pavement structure may require the designer to account for the high layer stiffness to obtain accurate predictions of the pavement stress, strain, and displacement (Milestones, 2002).

The 2002 guide uses a hierarchical approach for design inputs. This allows the engineer to consider the pavement design as a priority and to devote resources to material characterization accordingly.

Table 3-5. Material Categories (ERES Consultants, 2001)

Bound Materials	Unbound Materials
Asphalt Materials	**Non-Stabilized Granular Base/Subbase**
Hot Mix AC - Dense Graded	Granular Base/Subbase
Central Plant Produced	Sandy Subbase
In-Place Recycled	Cold Recycled Asphalt (used as aggregate)
Hot Mix AC - Open Graded Asphalt	RAP (includes millings)
Hot Mix AC - Sand Asphalt Mixtures	Pulverized In-Place
Cold Mix AC	Cold Recycled Asphalt Pavement (AC plus aggregate base/subbase)
Central Plant Processed	
In-Place Recycled	
	Subgrade Soils
PCC Materials	Gravelly Soils (A-1; A-2)
Intact Slabs	Sandy Soils
Fractured Slabs	Loose Sands (A-3)
Crack/Seat	Dense Sands (A-3)
Break/Seat	Silty Sands (A-2-4; A-2-5)
Rubblized	Clayey Sands (A-2-6; A-2-7)
	Clayey Soils
Cementitiously Stabilized Materials	Low Plasticity Clays (A-6)
Cement Stabilized Materials	Dry-Hard
Soil Cement	Moist Stiff
Lime Cement Fly Ash	Wet/Sat-Soft
Lime Fly Ash	High Plasticity Clays (A-7)
Lime Stabilized/Modified Soils	Dry-Hard
Open graded Cement Stabilized Materials	Moist Stiff
	Wet/Sat-Soft
	Bedrock
	Solid, Massive and Continuous
	Highly Fractured, Weathered

M-E Material Characterization

Mechanistic characterization allows the application of the principles of engineering mechanics to the pavement analysis problem. The stiffness (modulus) of each layer corresponds to how those layers respond to applied wheel loading in terms of fatigue cracking and rutting. Further, thermal properties relate to how pavement layers behave in response to temperature changes.

To provide a common basis for understanding material requirements, two M-E based subcategories were developed for the 2002 Guide: pavement material response properties and major distress/transfer functions. The pavement response properties are required for predicting states of stress, strain, and displacement within the pavement structure when subjected to an external wheel load. In the design guide, these properties are elastic modulus (E) and Poisson's ratio (μ). Major distress/transfer functions are equations that relate material properties to distress modes for a particular material type. For flexible pavements, the major modes of distress are load-related fatigue fracture, permanent deformation, and transverse fracture (ERES Consultants, 2001).

Material Factors

Material factors considered by the Guide include: Time-dependent properties, time-temperature effects, and non-linear behavior. Numerous combinations of material types and quality are used in flexible pavement systems.

Time-dependent Properties

Since some materials' properties undergo time-dependent changes, it is an important consideration in M-E analysis. Chemical and physical hardening of asphalt binders due to short and long-term aging of asphalt binder cause changes in long term, time-dependent behavior. This is beneficial in that it decreases the chance of permanent deformation in the pavement system by increasing the stiffness of the binder. However, this increase in stiffness also increases the probability of fracture occurring due to load and environmental effects (i.e. fatigue and block cracking) (ERES Consultants, 2001). Due to load-related fatigue distress, materials may also undergo degradation of properties with time and load repetitions. Micro-cracks can develop and lead to a reduction in stiffness or modulus. The reduction in the modulus will lead to an increase in stress states within the pavement, and over time, a greater chance for permanent rutting. The loss of stiffness is considered in the 2002 Design Guide (ERES Consultants, 2001).

Time-Temperature Effects

Asphalt binders are extremely sensitive to temperature and the time rate of load. They are viscoelastic-plastic materials, and their behavior can be approximated by using the Maxwell Model in Figure 3-11. The spring is used to model the elastic behavior and the dashpot is used to model the time-dependent viscous behavior. In this figure, a stress (load = P) is applied, and the spring instantly reacts to the load, yielding the vertical rise in the strain (elastic behavior). The dashpot begins to relax some time after the load is applied and continues to move as long as the load is applied (viscous behavior). The time-dependent response is modeled using the viscous stress-strain rate relationship (i.e. the strain rate, $\frac{d\varepsilon}{dt}$, is the slope of the diagonal portion on the graph). When the load is released, the spring immediately retracts, hence the vertical drop, but the dashpot stays deformed, hence the horizontal line, which depicts plastic behavior (Findley, Lai, & Onaran, 1989). For asphalt binders, the elastic limit (i.e., negligible viscous flow) is reached at either cold temperatures or very fast loads. The viscous limit is reached at warm temperature or slow load applications.

Since the properties of HMA are a function of both time and temperature the modulus of the asphalt bound layers can vary widely (Table 3-6). For example, the modulus of an HMA layer may approach that of an unbound granular material at high temperatures and long loading rates (i.e. slow vehicle speed). At cold temperatures and short loading rates, the asphalt materials tend to behave in a pure elastic mode and have high modulus values. The 2002 Guide takes into account the range of temperatures expected in the design period. Since the stiffness properties of HMA materials are known to be the function of temperature rate of loading, age, and mixture characteristics, the Guide uses a master curve to develop a relationship between the stiffness of the HMA (dynamic modulus) and the time rate of loading and mixture temperature (Milestones, 2002). Using the master curve accounts for the approximate speed of the vehicle being considered (ERES Consultants, 2001). The master curve along with an asphalt binder-aging model serves as the basis for selecting mixture modulus values at an incremental point in the design analysis period.

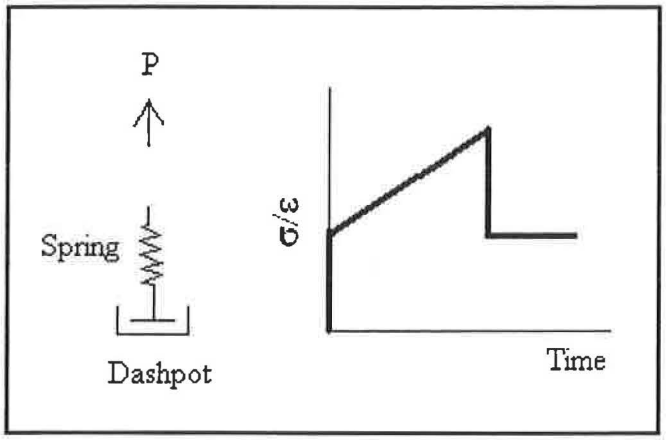


Figure 3-11. Maxwell model describing asphalt's viscoelastic-plastic behavior (Findley et al., 1989).

Table 3-6. Modulus Values for HMA Pavement Materials (Newcomb, D., Timm, D., & Mahoney, J., 2002)

Material	Modulus (psi)
HMA (0C)	2,000,000
HMA (21C)	500,000
HMA (49C)	20,000
Crushed Stone	20,000-40,000

Non-Linear Behavior

A material is non-linear when stress is not proportional to strain. The Guide only considers unbound base/subbase and subgrade materials as non-linear. For example coarse grained materials exhibit a stress hardening behavior, $M_R=k_1\theta^{k_2}$

where: M_R = Resilient Modulus

θ = Bulk Stress

k_1, k_2 = Constants

This factor is incorporated into design in the Guide through advanced M-E analysis (ERES Consultants, 2001).

CHAPTER IV. ALABAMA TRAFFIC STATISTICS

Introduction

Recent presentations and reviews of outputs from the beta versions of the new 2002 Pavement Design Guide software have concentrated on evaluations of how to use the software, discussions of load spectra, selection of material properties, and evaluation of pavement performance prediction outputs. Load spectra, while providing the most accurate approach for blending traffic variability, seasonal changes, and environmental considerations, is not information that is currently readily accessible for a wide range of roads.

Lack of easily obtainable local information can lead the pavement designer to accept the default values. In the case of traffic inputs, training information for using the 2002 Design Guide lists the needed traffic inputs and knowledge of parameters for each of the three hierarchical levels as (ERES, 2001):

- Level 1: segment specific data (AVC, WIM, vehicle counts) with a good knowledge of parameters
- Level 2: regional/statewide data (AVC, WIM, vehicle counts) with a fair knowledge of parameters
- Level 3: national data (AVC, WIM, vehicle counts), with an educated guess based on local experience and a knowledge of parameters listed as “poor”.

While default values used in the 2002 Design Guide were developed using actual traffic data in the Long Term Pavement Performance (LTPP) database, all of the defaults may not reflect typical local traffic conditions. This case study investigates the use of readily available Alabama traffic data to suggest when and where changes in traffic inputs may be needed. Adjustments for the Level 3 traffic inputs were investigated.

Objectives

The objectives for this chapter were to:

- Review the default traffic values used in the 2002 Design Guide with respect to readily available Alabama traffic data.
- Determine how to use easily available traffic statistics to customize the 2002 Design Guide Level 3 inputs.

Scope

Several state DOTs now make some of their traffic data available on the Internet or on a CD-ROM, providing easy access to widely used traffic information. The relatively new Alabama Traffic Statistics website (ALDOT, 2004) provides basic traffic information obtained from over 1,000 traffic counters all over the state. Traffic information that is available for each counter includes: annual average daily traffic (AADT) for each of 10 years, proportion of traffic traveling in the peak hour (K), directional distribution (D), percent trucks in the average daily traffic (TADT), percent of trucks that are heavy trucks

(3 axles or more), and percent trucks in the design hourly volume (TDHV). These data, either directly or after the development of correlation equations, were used to determine if adjustments to the Level 3 default 2002 Design Guide traffic volume factors are needed.

Traffic Data

The last 10 years of traffic data were obtained from the Alabama Traffic Statistics website for each traffic counter in 27 of Alabama's 67 counties. These data, collected from 75 sites, can be used to determine if, and how traffic changes with time. Key information that is needed for Level 3 default inputs are whether traffic growth is: no growth, linear growth, or compound growth. The percent of growth per year, which is also needed Level 3 input, can be calculated from this 10-year data set. The only hourly traffic distribution information is the design traffic factor, K, which defines the percent of the AADT that is contained in the design hourly volume (DHV):

$$K = \left(\frac{DHV}{AADT} \right) 100$$

The percent of the design hourly volume that is in the design direction is:

$$D = \left(\frac{DDHV}{DHV} \right) 100$$

Where the directional design hourly volume, DDHV, is:

$$DDHV = K(D)AADT$$

These statistics give some information about the maximum percent of traffic at the peak traffic time and the amount of traffic in the design direction. However, they do not provide further information about the distribution over the entire 24-hour period. This level of detail is typical of readily available traffic data from state DOTs.

The information from traffic counters in Alabama is divided into three vehicle classifications: heavy trucks (three axles or more), other trucks (two-axles), and everything else. The percent trucks in the design hourly volume (TDHV) therefore represents the percent of either two- or three or more axle trucks. The percent trucks in the AADT include both Alabama truck classifications. The highway functional classification for each site is also included.

Alabama Traffic Information

Traffic counter information from 27 of 67 Alabama counties was sampled. One counter for a state route and one for a US route were randomly selected from each county. One interstate route was also selected when this choice was available in the county.

Table 4-1 shows the range of traffic information obtained from the Alabama website. All of the FHWA functional classes are represented in this data base. The range of types of facilities can be seen in the variability in the AADT; the lowest AADT is 480 and the highest is over 50,000. The percent of the design hourly volume in the AADT (i.e., K) has limited variability, with a low of 10% and a high of 16%. The directional distribution varies somewhat more with a low of 55% to a high of 70%; 60% is the average value for D.

Table 4-1. Summary of Traffic Information Obtained From Alabama Web Site

Statistic	2002 AADT	K	D	TADT	TDHV	Heavy
Average	9,598	11.2	59.3	16.4	12.4	63.1
Std Dev.	11,852	1.0	3.3	8.8	6.6	18.2
Max	50,510	16	70	45	34	91
Min	480	10	55	3	2	20
Count	74	74	74	74	74	74

Average of three roads sampled per county (1 state route, 1 US route, and if applicable, 1 interstate)

The reported truck factors tend to have a much wider range than either the design hourly volumes or directional distribution. The AADTT varies between 3 to 45%. The percent of trucks in the design hourly traffic volume has a somewhat narrower range (2 to 34%). The percent of trucks that are "heavy trucks" (three or more axles) has the most variability, with the lowest value being 20% and the highest value of 91% heavy trucks.

2002 Design Guide Traffic Volume Adjustment Factors

The Level 3 input screens contain default settings for traffic volume factors needed for the design program that can either be used as suggested or altered by the user. There are four screens (windows) for entering traffic information:

- Monthly adjustment
- Vehicle class distribution
- Hourly distribution
- Truck growth factors

The information for calculating the vehicle class distribution can be accessed from two windows.

Traffic (Main Window)

Figure 4-1 shows where the annual average daily truck traffic can be calculated (one of two places). Clicking on the button with the three dots brings up the AADTT calculator window, which uses both the AADT and the TADT. This information can be obtained directly from the Alabama traffic counter website.

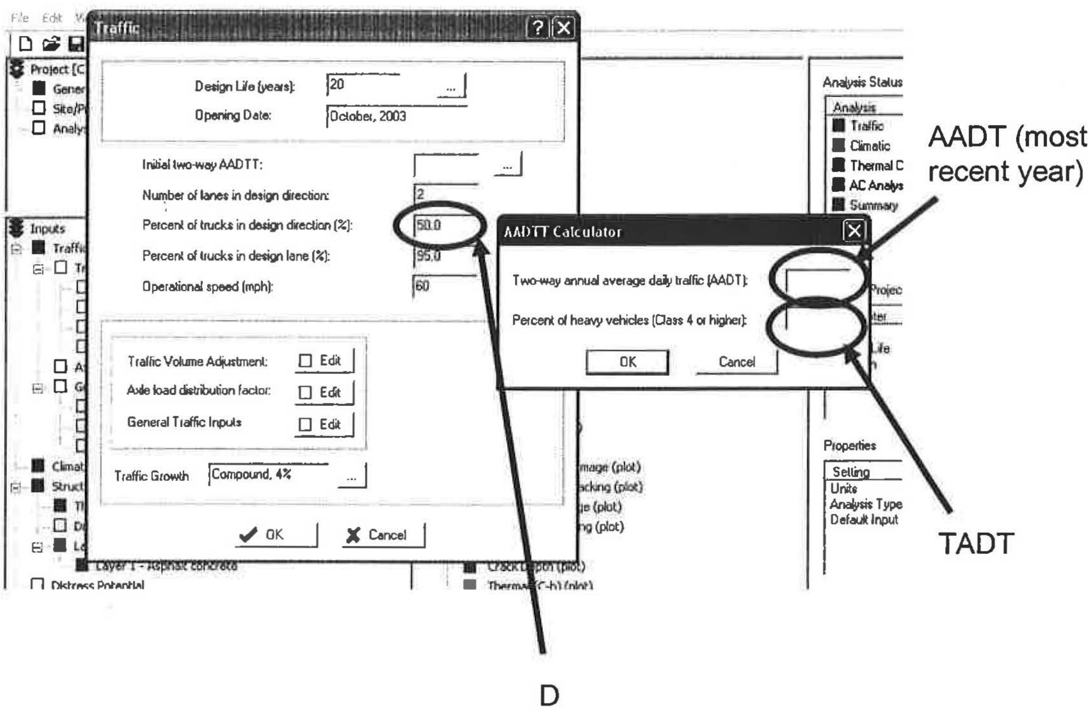


Figure 4-1. Main traffic data input screen.

The percent of trucks in the design direction is the next entry that can be made from the main window. The default directional distribution of traffic is 50%. Both Table 4-1 and Figure 4-2 show the directional distribution of traffic in the design lane obtained from the Alabama traffic statistics website is fairly consistent with a narrow range.

Based on this information, 60% would be a good, conservative standard value for Alabama. It is assumed that this value is for the peak traffic hour.

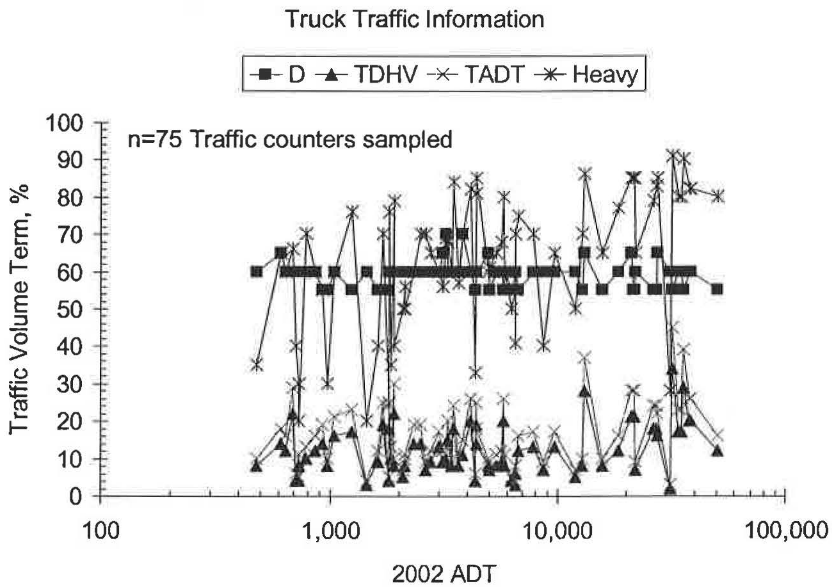


Figure 4-2. Range of truck traffic information obtained from the Alabama Traffic Statistics website.

Monthly Adjustment Factors

Figure 4-3 shows the window that is used to enter the monthly adjustment factors. The default value is 1.0 for each month and each FHWA vehicle classification. No information is available for this screen from the Alabama traffic statistics web site. While some state DOTs provide monthly adjustment factors in their traffic data packages; these factors are typically for the entire traffic stream rather than separated by vehicle classification. Given the lack of local information, an even distribution of traffic over the year and vehicle classifications seems reasonable.

Monthly Truck Distribution

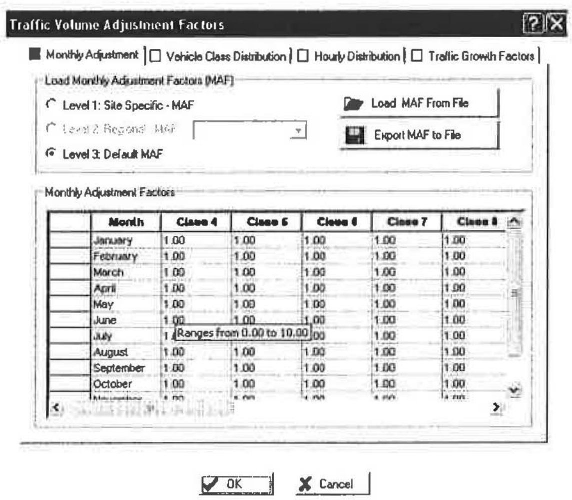


Figure 4-3. Monthly truck distribution factors (ERES, 2001).

Vehicle Class Distribution

Figure 4-4 shows the input screen for the distribution of the AADTT. The Alabama classification of heavy vehicles (three or more axles) covers the FHWA vehicle classifications 6 through 13. The sum of the default percent trucks in these classifications is 73.7%. This leaves 26.3% that would likely be classified in the Alabama statistics as "other trucks".

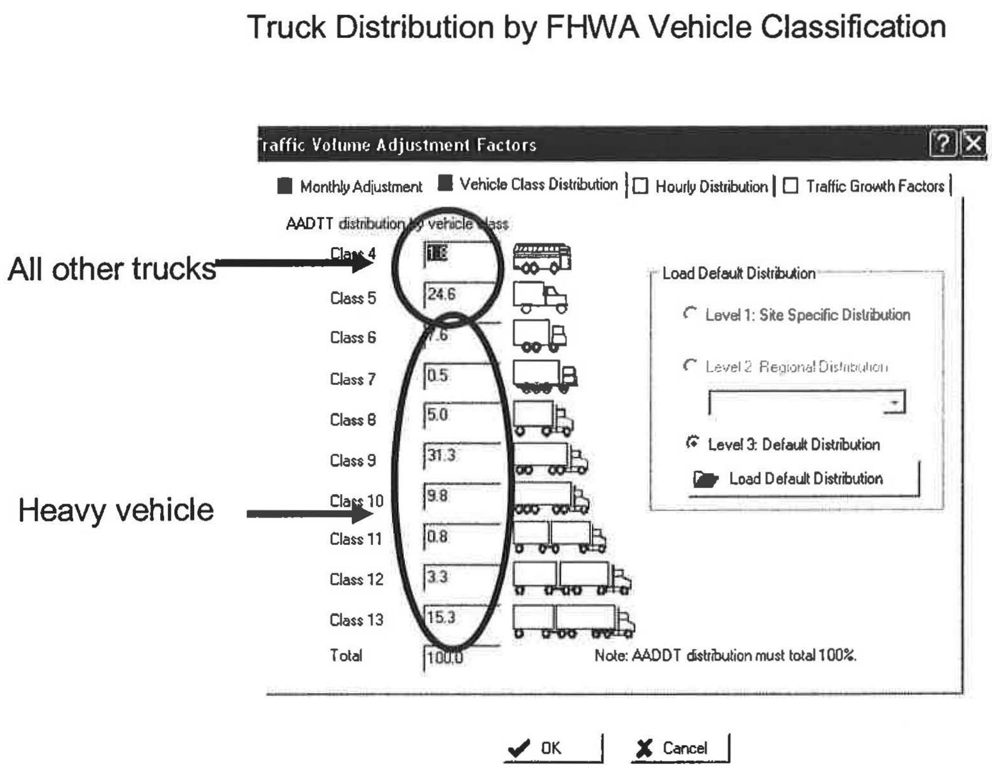


Figure 4-4. Vehicle class distribution screen (*1*).

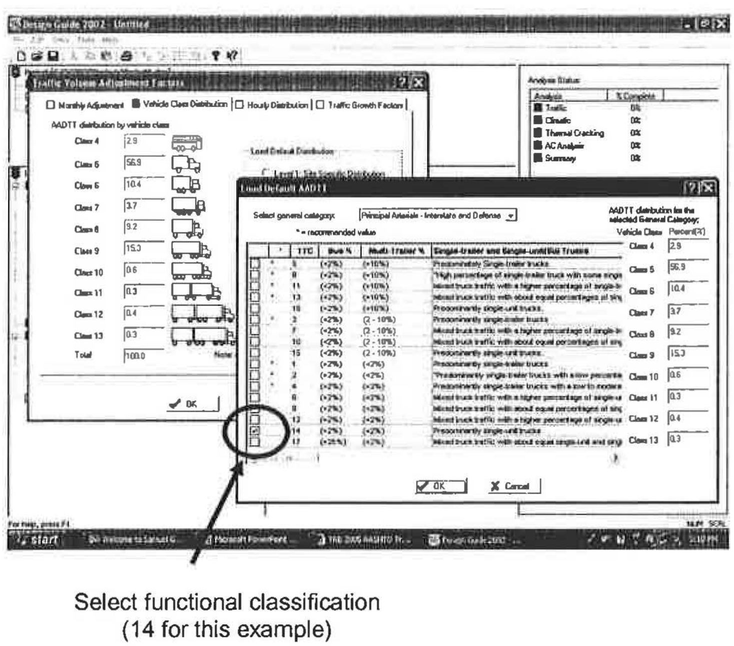


Figure 4.5. Vehicle class distribution using functional classification designation.

Figure 4-6 shows that the percentage of heavy trucks can be related to the log of the percent of all trucks in the ADT. As the percent of all trucks in the ADT increases, there is an increasingly higher percent of the trucks that are "heavy". The Level 3 default percentage of heavy trucks (73.7%) corresponds to around 20% of trucks in the ADT. Since the percent of heavy trucks in the ADT varies more at the lower TADT levels, it is suggested that the Level 3 defaults be tailored for local conditions when the TADT is lower than 20%.

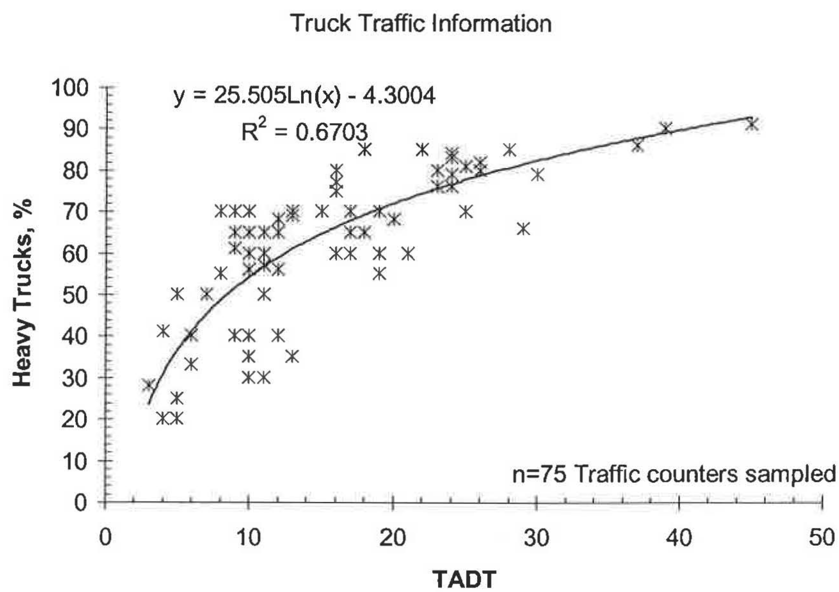


Figure 4-6. Relationship Between Trucks in the ADT and the Percent of Trucks that are Heavy Vehicles.

Hourly Truck Distribution

Figure 4-7 shows the third screen, which contains inputs for the hourly distribution of truck traffic. Note that while a percent of truck traffic per hour over a 24 hour period can be entered from this window, the default values are really divided into 5 time periods:

- 2.3% from midnight to 6 am
- 5% from 6 am to 10 am
- 5.9% from 10am to 4 pm
- 4.6% from 4 pm to 8 pm
- 3.1% from 8 pm to midnight

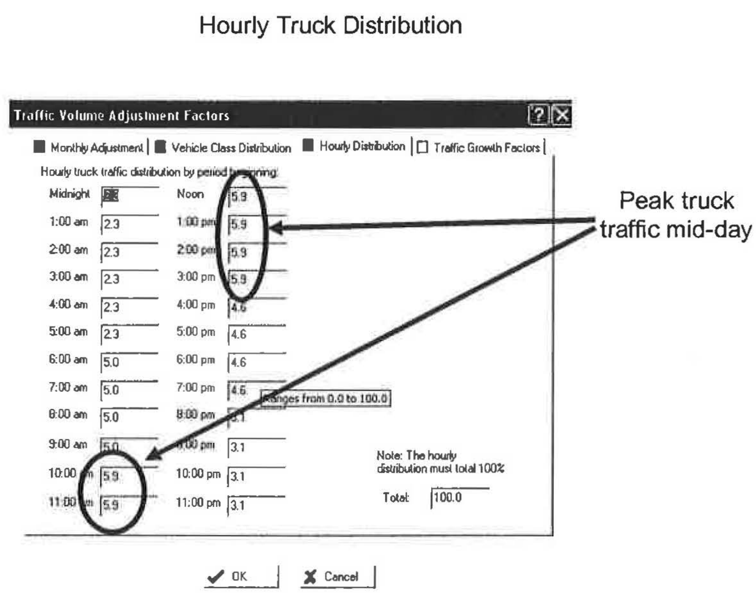


Figure 4-7. Traffic Input Screen for Vehicle Distribution in 24 Hour Period.

The higher concentration of trucks during the mid-day hours does not seem a reasonable pattern for long-haul heavy trucks. Mid-day will be times of warmer daily temperatures, and higher traffic volumes that will likely lead to greater traffic congestion and increased travel time (slower loadings). Using this hourly distribution with actual heavy truck data will produce a design that represents a worst case for flexible pavement performance when rutting is the predominate distress (slow loads, warm temperatures). It should also cover the time of maximum warp in concrete pavements, which would result in a maximum potential for mid-slab cracking.

The change in the percent of trucks in the design hourly volume generally follows the increases and decreases seen in the percent heavy trucks (Figure 4-2). Either the 2002 Design Guide defaults, or simply an equal distribution of trucks over a 24 hour period, seem reasonable in the absence of site-specific data.

The next information needed is the percent of traffic in the design lane. Since this type of information is not typically included in readily available traffic data, in the absence of local information, use of the default values is the only option.

The section on growth factors in the window shown in Figure 4-8 requires the user to select the growth relationship (no growth, linear growth, or compound growth). Figure 4-9 shows the AADT obtained from the web site plotted for a sample of 15 traffic counter stations; in all cases a linear growth model better fit the data than did a model based on no growth or on compound growth. Since the percent of traffic comprised of trucks is based on a percentage of the AADT, if the change in AADT is linear, then the truck growth factor will be linear.

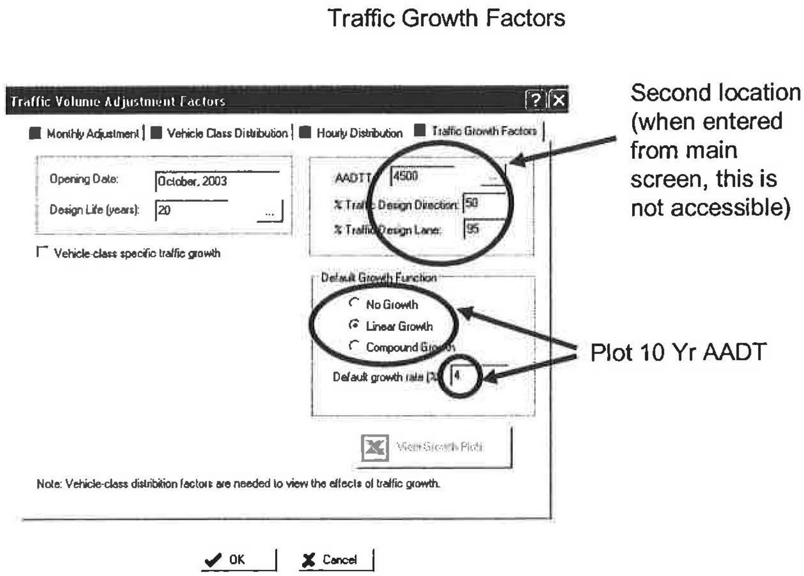


Figure 4-8. Traffic Growth Factor Input Screen.

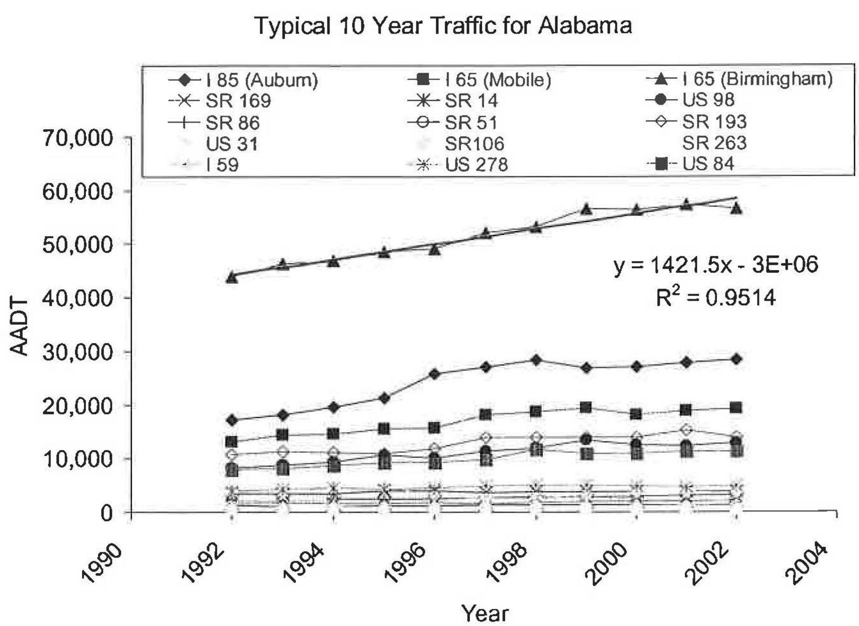


Figure 4-9 Typical Traffic Growth for Alabama Roadways.

Figure 4-10 shows the relationship between the slope of the traffic with time (years) for all 75 sites (i.e., the slopes from the relationships shown in Figure 4-9). There is a good linear relationship, with more data scatter with higher levels of AADT. The slope is 0.0337, or 3.37% growth per year. The default value of 4% will tend to result in a conservative design due to very high estimation of traffic of design life traffic.

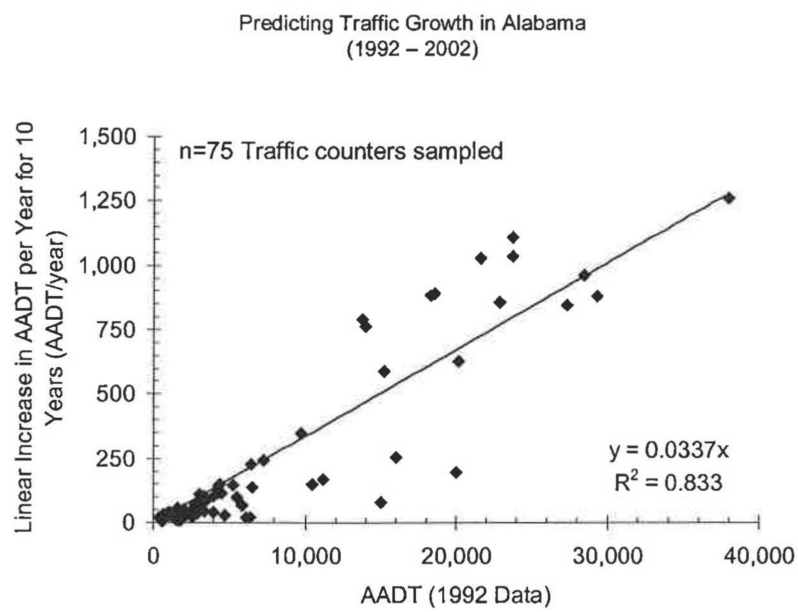


Figure 4-10. Relationship Between AADT and the Linear Traffic Growth.

Since the 2002 Design Guide requires the percent growth, the linear change in traffic with time (slope) is converted to a percent of the starting 1992 AADT. Figure 4-11 shows when there are less than 20,000 vehicles per day, the percent linear growth ranges from less than 1 to slightly over 4%. Because of the wide range in the actual percent growth, traffic information for local conditions should be used to estimate the percent growth for facilities with less than 20,000 vehicles per day. The 2002 Design

Guide default percent growth of 4% seems to be more reasonable for facilities with AADT greater than 20,000 vehicles per day.

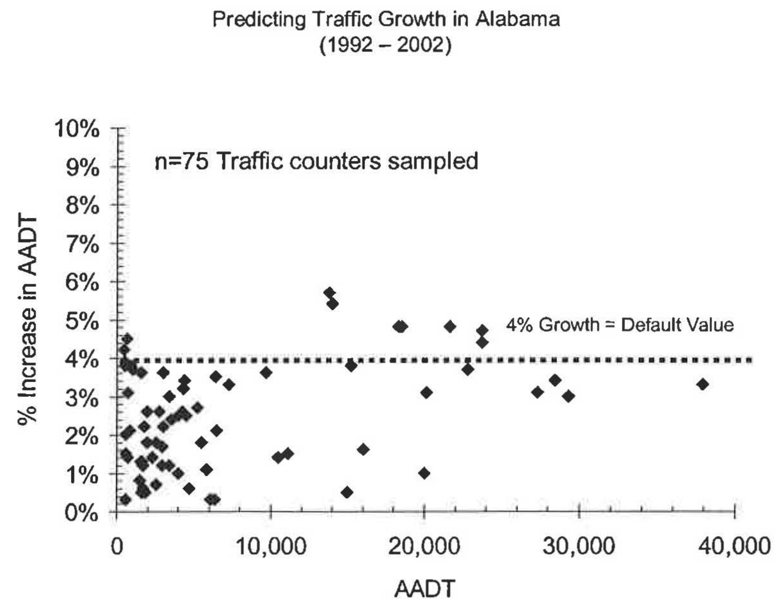


Figure 4-11. Percent Increase in AADT as a Function of the AADT for Alabama.

Recommendations for Alabama Traffic Inputs

The following conclusions can be drawn from this evaluation of the suitability of readily available Alabama traffic statistics for pavement design:

1. No information is available from the Alabama website that can be used to adjust the monthly distribution of traffic. The default values provide for evenly distributed traffic over the year for each class of vehicle. Since no other information is currently available, it is recommended that the default values be used in this case.

2. If information about the AADT, TADT, percent heavy trucks, and the functional classification of the roadway is entered, the default distribution among these classifications seems to be reasonable. This information is all readily available on the Alabama Traffic Statistics website.
3. Adjustment of the default percent trucks should be considered critical when the AADTT is less than 20%. Use of the default values without adjustment for the traffic statistics noted above could result in pavements that are substantially over-designed (as with SR 193) or under-designed (as with I-85).
4. The hourly distribution of truck traffic is not readily available; however, use of the default values is questionable since these values run contrary to observed patterns at some locations.
5. The directional distribution in the design hourly volume ranges from 55 to 70%, with the average being 60% for sites studies in Alabama. An increase in the default directional distribution from 50 to 60% should be considered to avoid possibly designing inadequate pavement structures.
6. A linear relationship for traffic growth in Alabama was seen for all of the data evaluated. This type of relationship is recommended as the default for Alabama.
7. The linear change in traffic with time, over a 10-year period, was well correlated with the AADT. The default Level 3 percent growth of 4% appears to be high for facilities with less than 20,000 ADT. If at all possible, historical traffic data for local facilities should be used for estimating the percent growth to avoid the possibility of over-design.

CHAPTER V. EVALUATION OF PUBLICLY RELEASED VERSION OF 2002 DESIGN GUIDE

Beta Version 2002 Design Guide Software

To evaluate output from the 2002 Design Guide, sample designs were run in the Beta Version of the Guide (received in August 2002). For one set of designs, the typical inputs included a 12 % binder content, AC thickness from 5-10 inches, a constant granular base (GB) thickness of 10 inches, and traffic levels of 150-3750 AADTT, which yielded 1.11E+06 to 2.75E+07 ESALs. Despite changes in thickness and traffic levels, there was a minimum change in the output from the Guide, which implies that the output data are inaccurate. A further discussion of the output files is below.

The output files generated by the Design Guide are easy to navigate. Additional effort is required to assess the adequacy of the design given the data on the summary sheets. The accuracy of data, nonetheless, is questionable. The IRI seemed to be nonsense; it increased 1 unit per month. Rutting appeared high, and failure (0.5 in) due to rutting usually occurred within the first 5 years. The data generated for the maximum surface down and the maximum bottom up cracking appeared suspicious. The maximum surface down cracking was usually a constant 4.95 ft/500ft and is shown in Figure 3-12. The bottom up cracking is initially very small, but then increases and becomes constant (100ft^2/500ft), Figure 3-13. The thermal cracking data seemed to be a placeholder, and will be discussed below.

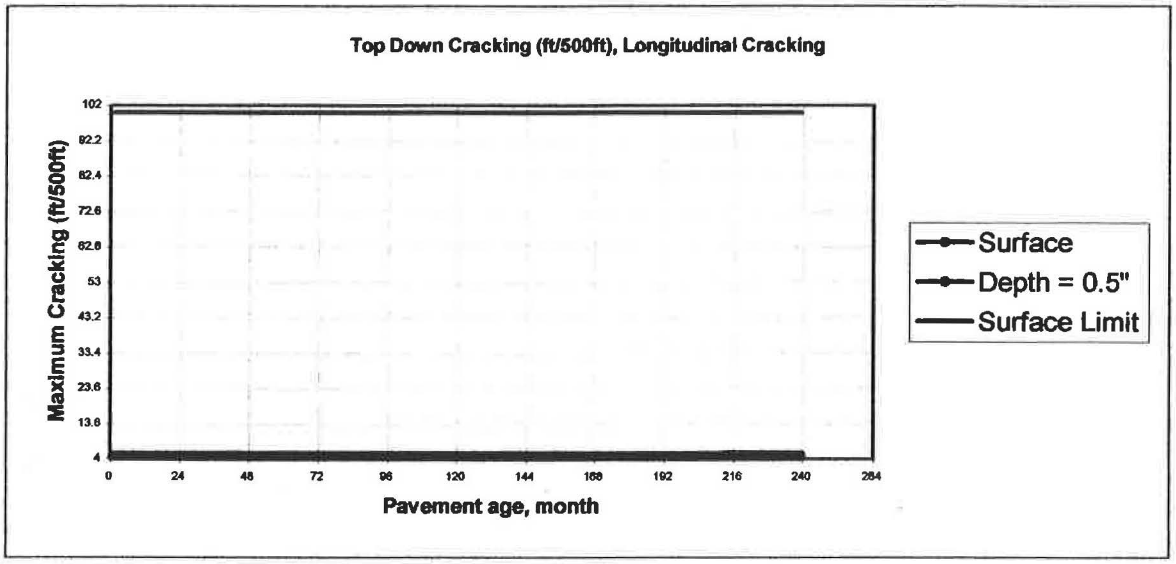


Figure 5-1. Maximum Surface Down Cracking

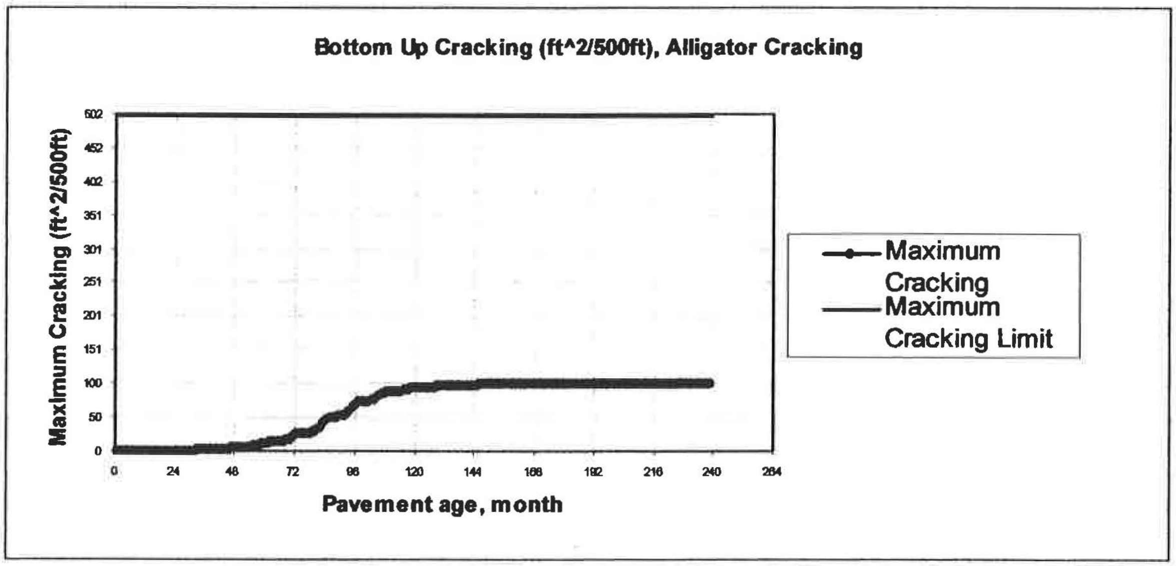


Figure 5-2 Maximum Bottom Up Cracking

There have been numerous subsequent upgrades in preparation for the release of the Design Guide for public use, however there were progressively more bugs/problems with the pre-release upgrades. These bugs occurred even when using the default values.

With one version of the Guide (received August 2002), a 20-year design would take up to 1-½ hours to run, and possibly generate no output files. The current version (received in December 2002) runs for about 30 minutes then becomes idle and displays '9999' for the remaining AC analysis time. According to ERES Consultants, this bug is due to a problem within the thermal cracking model. Per an email conversation with the ERES planning team, deselecting the 'AC surface down cracking and the AC Thermal Fracture options' in the Analysis parameter screen would allow the program to run. However, even with the options deselected, the program still fails to generate data. The response to this problem was that work is being conducted on the bugs along with finalizing calibration factors, and that hopefully a better performing version of the Guide will soon be available.

The Guide has a help menu, but most of the items including traffic, climate, and structure are for the most part placeholders (i.e. contains little to no information). The help menu currently contains information on the introduction and the history of the Guide, what's new, and how to get started (i.e. installing, running, and uninstalling the Guide). The help menu also contains general project information.

Initial and extensive work was done with the beta version 2002 Design Guide available during the original contract timeline (Robinson, 2003). A no-cost time extension was granted so that subsequent, but limited analyses could be completed with newer version(s) of the Design Guide software. The remainder of this report will present the findings from this subsequent evaluation. The additional time was also used to more thoroughly investigate easily obtainable, Alabama-specific traffic information that can be used in the Post 2002 Design Guide.

Publicly Released 2002 Design Guide Software

Once the 2002 Design Guide was publicly released, this new version was evaluated to determine if the pavement performance prediction results were more reasonable. Table 5-1 shows the experimental design for evaluating the newer version of the program. A single ADT (10,000) value was used and pavement performance distress predictions were extracted from each data base for 5 and 20 year results. A consistent 3% traffic linear growth, per the analysis in the previous chapter, was used. The load distribution was varied by selecting three different functional classes of roadway.

The influence of load distribution as a function of roadway functional class can be expressed as a percent of Class 6 or higher vehicles where:

- Functional class 1 has 90% of vehicles in the class 6 or higher categories
- Functional class 6 has 60% of vehicles in the class 6 or higher categories
- Functional class 14 has 40% of vehicles in the class 6 or higher categories

Table 5-1. Abbreviated Experimental Design for Post DG2002 Evaluations.

HMA	Base	Soil	Functional Class		
			1	6	14
5	6	A-1	X	X	X
		A-6	X	X	X
	12	A-1	X	X	X
		A-6	X	X	X
10	6	A-1	X	X	X
		A-6	X	X	X
	12	A-1	X	X	X
		A-6	X	X	X
15	6	A-1	X	X	X
		A-6	X	X	X
	12	A-1	X	X	X
		A-6	X	X	X

Table 5-2 and Figure 5-3 show when the functional class number increases, the percentage of class 6 vehicles decreases and the percentage of class 2 vehicles increases. This means that for a given AADT, functional class 1 roadways will have substantially more heavier trucks than functional class 14.

Table 5-2. DG02 Default Vehicle Distribution Per Roadway Functional Class.

Vehicle Class	Percent of Vehicle Class in Each Functional Class		
	1	6	14
Class 4	1.3%	2.8%	2.9%
Class 5	8.5%	31.0%	56.9%
Class 6	2.8%	7.3%	10.4%
Class 7	0.3%	0.8%	3.7%
Class 8	7.6%	9.3%	9.2%
Class 9	74.0%	44.8%	15.3%
Class 10	1.2%	2.3%	0.6%
Class 11	3.4%	1.0%	0.3%
Class 12	0.6%	0.4%	0.4%
Class 13	0.3%	0.3%	0.3%

Shaded rows indicate major changes in percent vehicles

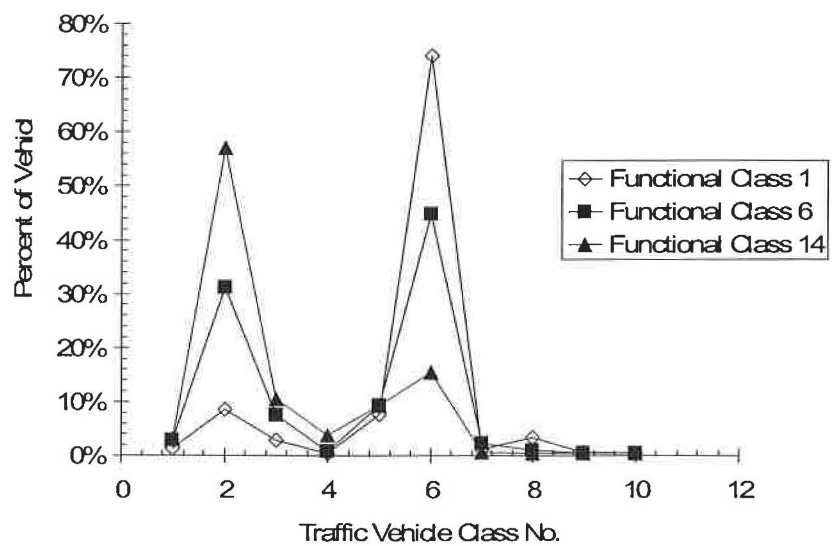


Figure 5-3. Default Distribution of Vehicles for Three Functional Classes of Roadways.

Table 5-3. Flexible Pavement Distresses from the 2002 Design Guide after 5 Years of Traffic (9.2M Cumulative Heavy Trucks).

HMA Thick. in	Base Thick. in	AASHTO Soil Type	Funct. Class.	Age mo.	Long. Crack. %	Alligator Cracking %	AC Rutting in	Total Rutting in	IRI in/mi
5	6	A1	1	60	91.6	59.5	0.42	0.76	254
10	6	A1	1	60	36.4	2.0	0.31	0.52	221
15	6	A1	1	60	0.2	0.1	0.16	0.27	84
5	12	A1	1	60	96.6	65.1	0.42	0.81	0
10	12	A1	1	60	45.4	2.6	0.29	0.55	221
15	12	A1	1	60	0.1	0.2	0.15	0.28	84
5	6	A6	1	60	97.5	75.5	0.42	0.80	0
10	6	A6	1	60	13.6	3.7	0.28	0.50	0
15	6	A6	1	60	0.0	0.3	0.15	0.31	223
5	12	A6	1	60					
10	12	A6	1	60	13.5	3.7	0.28	0.52	87
15	12	A6	1	60	0.0	0.3	0.15	0.32	86
5	6	A1	6	60	86.1	51.3	0.37	0.70	246
10	6	A1	6	60	24.9	1.5	0.27	0.47	221
15	6	A1	6	60	0.2	0.1	0.14	0.25	220
5	12	A1	6	60	94.4	57.4	0.36	0.75	252
10	12	A1	6	60	31.1	1.9	0.26	0.50	221
15	12	A1	6	60	0.1	0.1	0.13	0.26	220
5	6	A6	6	60	95.6	69.2	0.36	0.73	272
10	6	A6	6	60	8.6	2.9	0.25	0.46	224
15	6	A6	6	60	0.0	0.2	0.13	0.28	223
5	12	A6	6	60	96.6	69.3	0.36	0.79	272
10	12	A6	6	60	8.9	3.0	0.24	0.48	224
15	12	A6	6	60	0.0	0.3	0.13	0.30	223
5	6	A1	14	60	66.1	37.6	0.30	0.63	236
10	6	A1	14	60	12.0	1.0	0.22	0.42	84
15	6	A1	14	60	0.1	0.1	0.10	0.21	83
5	12	A1	14	60	86.9	46.1	0.30	0.68	105
10	12	A1	14	60	15.6	1.3	0.21	0.45	84
15	12	A1	14	60	0.1	0.1	0.11	0.24	84
5	6	A6	14	60	89.0	57.0	0.30	0.65	117
10	6	A6	14	60	4.3	2.1	0.20	0.41	87
15	6	A6	14	60	0.0	0.2	0.11	0.25	86
5	12	A6	14	60	91.6	59.1	0.30	0.71	120
10	12	A6	14	60	4.4	2.2	0.20	0.43	87
15	12	A6	14	60	0.0	0.2	0.11	0.27	86

Shaded cells indicate data that does not appear to be reasonable

Table 5-4. Flexible Pavement Distresses from the 2002 Design Guide after 20 Years of Traffic (44.5 M Cumulative Heavy Trucks).

HMA Thick. in.	Base Thick. in	AASHTO Soil Type	Funct. Class.	Age mo.	Long. Crack. %	Alligator Cracking %	AC Rutting in	Total Rutting in	IRI in/mi
5	6	A1	1	240	99.4	89.6	0.88	1.28	472
10	6	A1	1	240	85.7	9.3	0.63	0.88	318
15	6	A1	1	240	2.3	0.6	0.32	0.46	131
5	12	A1	1	240	99.4	91.5	0.87	1.34	0
10	12	A1	1	240	89.7	12.1	0.60	0.90	318
15	12	A1	1	240	1.0	0.8	0.32	0.47	131
5	6	A6	1	240	99.4	94.6	0.86	1.32	0
10	6	A6	1	240					0
15	6	A6	1	240	0.0	1.5	0.31	0.50	330
5	12	A6	1	240					
10	12	A6	1	240	59.8	16.3	0.58	0.87	151
15	12	A6	1	240	0.0	1.6	0.31	0.52	146
5	6	A1	6	240	98.5	86.0	0.77	1.16	433
10	6	A1	6	240	77.6	7.2	0.55	0.79	317
15	6	A1	6	240	1.6	0.5	0.28	0.41	315
5	12	A1	6	240	99.4	88.6	0.76	1.22	459
10	12	A1	6	240	82.4	8.9	0.53	0.82	317
15	12	A1	6	240	0.7	0.6	0.28	0.43	315
5	6	A6	6	240	99.4	92.8	0.75	1.20	553
10	6	A6	6	240	47.6	13.0	0.51	0.77	334
15	6	A6	6	240	0.0	1.2	0.27	0.46	330
5	12	A6	6	240	99.4	92.8	0.75	1.20	553
10	12	A6	6	240	48.2	13.6	0.50	0.79	334
15	12	A6	6	240	0.0	1.3	0.27	0.47	330
5	6	A1	14	240	95.6	77.8	0.63	1.01	387
10	6	A1	14	240	58.7	5.1	0.45	0.69	132
15	6	A1	14	240	1.1	0.4	0.23	0.36	131
5	12	A1	14	240	98.5	83.2	0.62	1.08	228
10	12	A1	14	240	65.7	6.4	0.43	0.72	132
15	12	A1	14	240	0.5	0.5	0.23	0.38	131
5	6	A6	14	240	98.5	88.3	0.61	1.05	287
10	6	A6	14	240	30.2	9.8	0.41	0.67	149
15	6	A6	14	240	0.0	0.9	0.22	0.40	146
5	12	A6	14	240	99.4	89.3	0.61	1.12	298
10	12	A6	14	240	30.8	10.3	0.41	0.69	149
15	12	A6	14	240	0.0	1.0	0.22	0.42	146

Shaded cells indicate data that does not appear to be reasonable

Tables 5-3 and 5-4 show the individual flexible pavement distresses predicted from this version of the 2002 DG software after 5 years and 20 years of traffic, respectively. The total run time for the software was still about 45 minutes. All of the pavement distress values were reported by the program for each run. However, there were three instances where the IRI calculations reported values of 0; these are marked as shaded cells in the tables. Individual flexible pavement distresses predicted from the 2002 Design Guide were reported by the program for each run.

The following trends can be seen in Table 5-3 for the predicted levels of pavement distresses after 5 years of traffic:

- HMA layer thicknesses of 5 inches, regardless of base thickness and subgrade type, have begun to fail (longitudinal cracking consistently greater than 85% in the wheel paths).
- Alligator cracking is higher for flexible pavement constructed over an A-6 soil.
- Rutting in the HMA layers decreases with increasing HMA thickness.
- Total rutting in the pavement is about twice that in the HMA layer, regardless of base thickness and subgrade type.

At 20 years of traffic (Table 5-4):

- All of the 5 inch HMA pavements have failed.
- The functional class 1 percent vehicle distribution is predicted to fail 10 inches of HMA, regardless of base thickness or subgrade type. The 10 inches of HMA performs well when the percent heavy vehicles (class 6) are 60% or lower.

- Rutting the HMA layer is dependent on the HMA thickness; this rutting accounts for at least 60% of the total rutting.

5-Year Performance Results

A Pearson's correlation matrix (Table 5-5) was developed for the 5-year data to identify any viable single-variable statistically significant comparisons between the distresses, individual layer thickness, or subgrade type. The results show that each type of pavement distress presented in this evaluation is well correlated with another and with the HMA thickness. None of the distresses are well correlated with base thickness or subgrade type.

Table 5-5. Pearson's Correlation Matrix (5 yr; 9.5M Cumulative Heavy Trucks).

	HMA Thickness	Base Thickness	Soil Type	Func. Class	Long. Cracking, %	Alligator Cracking %	Subtotal AC Rutting in	Total Rutting in	IRI in/mi
HMA Thickness	1.00								
Base Thickness	-0.04	1.00							
Soil Type	0.05	-0.04	1.00						
Func. Class	-0.05	0.04	0.22	1.00					
Long. Cracking, %	-0.92	0.08	-0.09	-0.08	1.00				
Alligator Cracking (%)	-0.86	0.05	0.04	-0.04	0.96	1.00			
Subtotal AC Rutting (in)	-0.93	0.01	-0.16	-0.28	0.91	0.82	1.00		
Total Rutting (in)	-0.97	0.11	-0.05	-0.13	0.96	0.89	0.97	1.00	
IRI (in/mi)	-0.22	-0.18	0.07	-0.45	0.24	0.19	0.33	0.27	1.00

Shaded cells indicate good and very good single variable correlations.

Table 5-6 combines the results for the various HMA thicknesses so that more subtle trends due to changes in base thickness, subgrade type, and functional class (load distribution), can be seen more easily. Since a single AADT (10,000) was used for this analysis and the effect of the HMA thickness was eliminated by averaging, the SN changes only because of the change in the base layer thickness.

Table5-6. Average Flexible Pavement Distress for Each Traffic Classification and Subgrade Soil Classification for Each of Two Base Thicknesses (5 yr; 9.5M Cumulative Heavy Trucks).

Description of Structural Sections and Functional Classification	SN	Longitudinal Cracking Based on 2-Wheel Path Length, %	Alligator Cracking %	Subtotal AC Rutting in	Total Rutting in	IRI in/mi
FC 1, 6" Base, A-1 Soil	5.42	42.7	20.5	0.29	0.51	186
FC 6, 6' Base, A-1 Soil	5.42	37.1	17.6	0.26	0.47	229
FC 14, 6" Base, A-1 Soil	5.42	26.1	12.9	0.21	0.42	134
FC 1,12" Base, A-1 Soil	6.44	47.4	22.6	0.29	0.55	102
FC 6 12" Base, A-1 Soil	6.44	41.8	19.8	0.25	0.51	231
FC 14, 12" Base, A-1 Soil	6.44	34.2	15.8	0.21	0.46	91
FC 1, 6" Base, A-6 Soil	5.42	37.1	26.5	0.28	0.53	74
FC 6, 6"Base, A-6 Soil	5.42	34.8	24.1	0.25	0.49	239
FC 14, 6" Base, A-6 Soil	5.42	31.1	19.8	0.20	0.44	97
FC 1, 12" Base, A-6 Soil	6.44	37.0	27.3	0.24	0.45	87
FC 6, 12" Base, A-6 Soil	6.44	35.2	24.2	0.25	0.52	239
FC 14, 12" Base, A-6 Soil	6.44	32.0	20.5	0.20	0.47	98

Table 5-6 estimates that after 9.5 million cumulative heavy trucks, the pavement exhibits up to 47% of longitudinal cracking in the wheel paths and 23% alligator cracking in the wheel paths for HMA pavements over crushed gravel base and an A-1 subgrade soil. The estimated longitudinal and alligator cracking for HMA over crushed gravel base and A-6 soil are 37% and 27%, respectively. Longitudinal cracking in the wheel path is typically the precursor of alligator cracking, which is the result of a tensile crack that started at the bottom of the HMA layer propagates up to the surface. Once the longitudinal cracking in the wheel path starts, it quickly evolves into alligator cracking. The shift in longitudinal to alligator cracking in the wheel paths is most likely the reason for the shift in values between the two subgrade types. That is, the pavement on the A-6 (poor) subgrade is showing more advanced load related distress (higher alligator cracking).

Figure 5-4 show that the predicted longitudinal and alligator cracking decreases with the decrease in heavily loaded vehicles. As expected, the percent of distress decreases with the percent of heavy vehicles. As indicated by the Pearson's correlations, there is little evidence that changing the base thickness from 6 inches to 12 inches has any influence on the quantity of distress. It appears that there may be some indication that the prediction models indicate slightly lower quantities of distresses on the A-6 (poor subgrade) than on the A-1 subgrade.

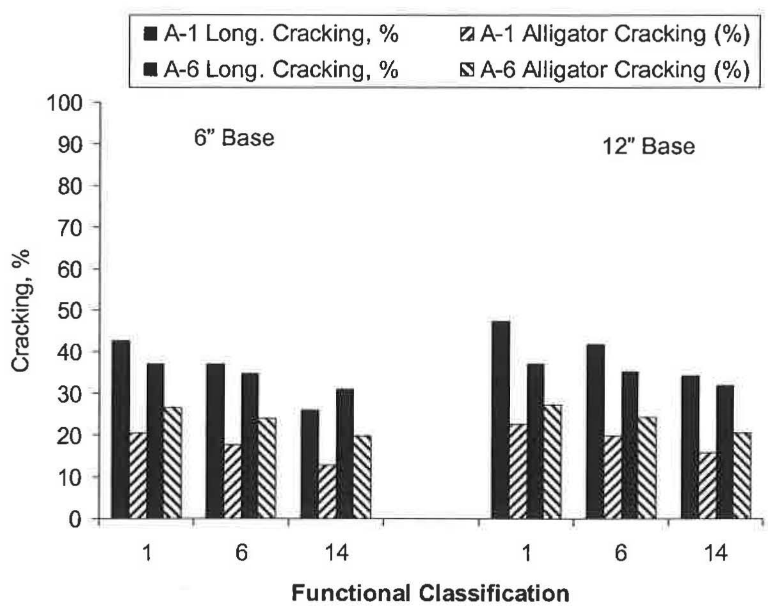


Figure 5-4. Influence of subgrade type, base thickness, and functional class on wheel path cracking.

Figure 5-5 indicates that the newer version of the 2002 Design Guide rutting prediction models account for changes in the percentages of heavy vehicles in the traffic volume. Rutting decreases, both in the HMA and for all layers (total), as the percent of heavy vehicles decreases. There is little indication that base layer thickness will influence the prediction of this distress. There is also no indication that the type of subgrade will have any influence on this distress.

The HMA rutting decreases slightly with decreases percentages of heavy vehicles. There is no apparent trend in the total rutting as a function of base thickness or type of subgrade. There is a slight, but not statistically significant, trend in decreased rutting with decreased heavy vehicle loading due to changes in base thickness or subgrade type.

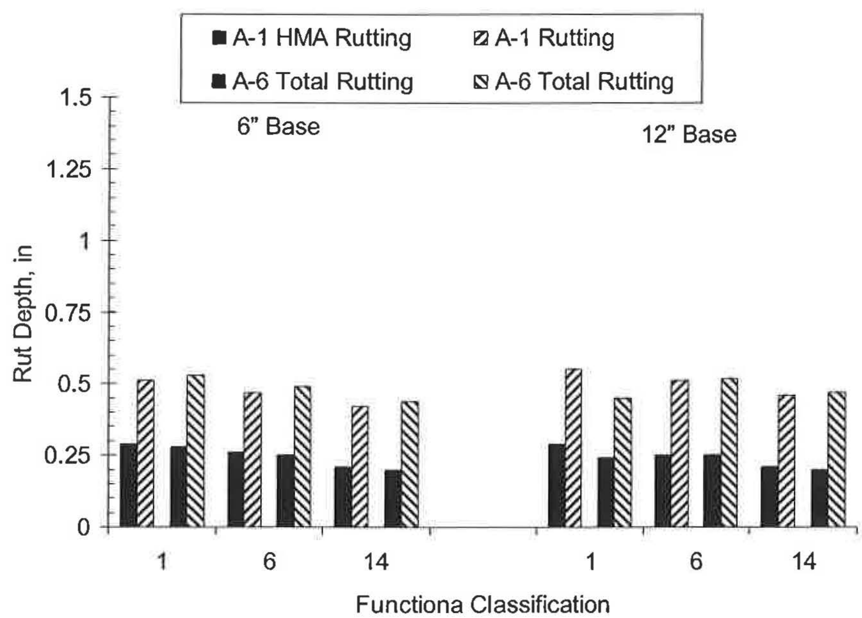


Figure 5-5. Influence of subgrade type, base thickness, and functional class on rutting.

The predicted IRI values are somewhat unusual in that there is a peak in the increase at the intermediate functional class. It is unclear whether this pavement performance prediction equation is fully developed at this time and this program output should be considered suspect.

20 Year Performance Results

A Pearson's correlation matrix (Table 5-7) was developed for the 20 year distress predictions. The correlations between each distress and between distresses and HMA thickness are similar to those seen for the 5 year results.

Table 5-8 combines the results for the various HMA thicknesses so that a comparison between functional class (load distribution), base thickness and subgrade type can be more easily made. The levels of distresses are higher than at 5 years, as expected.

Table 5-7. Pearson's Correlation Matrix (20 yrs; 44.5M cumulative heavy trucks).

	HMA	BASE	SOIL	SN	LC%	AC%	AC RUT	TOTAL RUT	IRI
HMA	1.00								
BASE	0.00	1.00							
SOIL	0.00	0.00	1.00						
SN	0.96	0.27	0.00	1.00					
LC%	-0.92	-0.04	-0.12	-0.90	1.00				
AC%	-0.84	-0.10	-0.02	-0.84	0.81	1.00			
AC RUT	-0.89	-0.09	-0.29	-0.88	0.93	0.86	1.00		
TOTAL RUT	-0.91	-0.04	-0.18	-0.89	0.94	0.91	0.99	1.00	
IRI	-0.29	-0.09	0.09	-0.31	0.28	0.26	0.22	0.21	1.00

Shaded cells indicate good and very good single variable correlations.

Table 5-8. Average Flexible Pavement Distress for Each Traffic Classification and Subgrade Soil Classification for Each of Two Base Thicknesses (20 yrs; 44.5M cumulative heavy trucks).

Description of Structural Sections and Functional Classification	SN	Longi. Crack. Based on 2-Wheel Path Length (%)	Alligator Cracking (%)	Subtotal AC Rutting (in)	Total Rutting (in)	IRI (in/mi)
FC 1, 6" Base, A-1 Soil	5.42	62.5	33.2	0.6	0.9	306.6
FC 6, 6' Base, A-1 Soil	5.42	59.2	31.2	0.5	0.8	354.7
FC 14, 6" Base, A-1 Soil	5.42	51.8	27.8	0.4	0.7	216.5
FC 1,12" Base, A-1 Soil	6.44	63.4	34.8	0.6	0.9	224.6
FC 6 12" Base, A-1 Soil	6.44	60.8	32.7	0.5	0.8	363.8
FC 14, 12" Base, A-1 Soil	6.44	54.9	30.0	0.4	0.7	163.6
FC 1, 6" Base, A-6 Soil	5.42	66.3	33.6	0.7	1.1	330.4
FC 6, 6"Base, A-6 Soil	5.42	49.0	35.7	0.5	0.8	405.8
FC 14, 6" Base, A-6 Soil	5.42	42.9	33.0	0.4	0.7	193.8
FC 1, 12" Base, A-6 Soil	6.44	64.3	37.5	0.5	0.7	149.3
FC 6, 12" Base, A-6 Soil	6.44	49.2	35.9	0.5	0.8	405.8
FC 14, 12" Base, A-6 Soil	6.44	43.4	33.5	0.4	0.7	197.8

CHAPTER VI. CONCLUSIONS AND RECOMMENDATIONS

The 2002 Design Guide user interface (input screens) is easy to use and leads the user through the information needed to run the analysis. The 2002 Design Guide software requires information be entered for:

- Project information
- Analysis Parameters
- Traffic
- Climate
- Materials Characterization
- Output

The ***Project Information*** screens require the same information as is currently logged for all Alabama Department of Transportation projects, so no change in ALDOT practices are needed for this section.

The selection of the ***Analysis Parameters*** requires the user define the threshold values for individual pavement distresses. It is recommended that ALDOT review their pavement management data base in order to develop rational levels of distresses for specific time periods (e.g, 5, 10, 15, and 20 years).

The Alabama ***Traffic*** Statistic website is a very good, very easy to use source for most of the traffic parameters needed for the analysis. Information that is available and needed include:

- AADT
- TADT
- Percent heavy trucks
- Functional classification

Load distribution (spectra) can be accounted for in the analysis by selecting the appropriate roadway functional class. Care needs to be taken to make sure that this is done as the screen for this selection is one of the very few that are not easily found.

Adjustment of the default percent trucks should be considered critical when the AADTT is less than 20%. Using the default values without adjustment for actual traffic conditions could result in pavements that are substantially over- or under-designed. The directional distribution in the design hourly volume ranges from 55 to 70%, with the average being 60% for sites studies in Alabama. An increase in the default directional distribution from 50 to 60% should be considered to avoid possibly designing inadequate pavement structures.

A linear relationship for traffic growth in Alabama was seen for all of the data evaluated. This type of relationship is recommended as the default for Alabama. The linear change in traffic with time, over a 10-year period, was well correlated with the AADT. The default Level 3 percent growth of 4% appears to be high for facilities with less than 20,000 ADT. If at all possible, historical traffic data for local facilities should be used for estimating the percent growth to avoid the possibility of over-design.

The inputs for ***Climate*** can be obtained from a number of files included in the 2002 Design Guide software. While the weather data may not be specifically developed for the project site, there seems to be a substantial amount of information available for cities throughout the state.

The ***Material Characterization*** screens require the most information that, at this time, is the least well-defined for Alabama materials. Development of a catalogue or GIS presentation of various material properties for Alabama will require the most effort on the part of the DOT. It is strongly suggested that ALDOT focus on developing guidelines for material property inputs for pavement design.

The ***Output*** for the 2002 Design Guide (publicly released version) indicate that the results consider the percent of heavy vehicles and the thickness of the HMA layer in the prediction of pavement distresses. However, the current models are insensitive to the base thickness and the type of subgrade. These limitations need to be explored very thoroughly before the 2002 Design Guide is used by ALDOT.

When using the 2002 Design Guide software for flexible pavement design:

1. Estimates of pavement distresses are strongly tied to the thickness of the HMA layer.
2. The choice of roadway functional class is important in the prediction of repetitive load-related distresses (longitudinal and alligator cracking). This is because the default distributions of heavy vehicles changes with functional class.
3. Estimates of rutting in the HMA layer are dependent primarily on the properties of the HMA layer. Estimates of total rutting seem to be a function of both the percent of heavy vehicles and a combination of base thickness and subgrade type. However, no statistically significant differences were evident.
4. The estimate of changes in ride quality (i.e., IRI) over time was erratic. It appears that this prediction model needs to be refined.

Recommendations

Recommendations from this work are:

- **Traffic information**: Expand the Alabama Traffic Statistic website to include information on the percent of each vehicle class in the AADT for each of the 10 years included in the current database.
- **Analysis parameters:** The ALDOT pavement management system information should be evaluated to establish typical limits for each distress type for each roadway functional classification.

- **Material characteristics:** ALDOT should use the historical data for material properties to define typical material property values for those parameters that are not easily obtainable by the design engineers (e.g., soil resilient modulus, typical CBR values, and subgrade types).
- **Validation of 2002 Design Guide:** A set of Alabama roadways that are between 5 and 10 years should be identified for validation of the 2002 Design Guide. Construction, material, traffic and performance information for these in-service projects should be collected. If possible, current percent truck distributions should be obtained, and then used to validate the predicted progression of distresses.

CHAPTER VII. REFERENCES

Alabama Traffic Statistics. Alabama Department of Transportation http://205.172.52.51/website/production/TrafficVolumn/Public/viewer.htm Accessed July 12, 2004.

Ali, H. A. & Tayabji S. D. (1998). Evaluation of Mechanistic-Empirical Performance Prediction Models for Flexible Pavements. Transportation Research Record, 1629, 169-180.

American Association of State Highway and Transportation Officials [AASHTO], (1986). AASHTO Guide for Design of Pavement Structures. Volume 2. Washington D. C.: AASHTO.

American Association of State Highway and Transportation Officials [AASHTO], (1993). AASHTO Guide for Design of Pavement Structures. Washington D. C.: AASHTO.

Brown, E. R., Kandhal, P. S., & Zhang, J. (2001). Performance Testing For Hot Mix Asphalt. NCAT Report No 01-05. Auburn University: National Center for Asphalt Technology.

ERES Consultants. (2002). 2002 Design Guide [Beta Version of Computer Software]. Illinois: Author.

ERES Consultants. The 2002 Pavement Design Guide – Mechanistic-Empirical Design of New and Rehabilitated Pavements. ERES Consultants, a Division of ARA, Inc, Arizona State University, Fugro-BRE, Inc. 2002 Design Guide NCHRP 1-37A training materials. June, 2004.

Federal Highway Administration (1997). SUPERPAVE for the Generalist Engineer and

Project Staff (Publication No. FHWAHI 97-031). United States: National Highway Institute.

Hajek, J. J. (1995). General Axle Load Equivalency Factors. Transportation Research Record, 1482, 67-78.

Hilton, DR. & Roque, R. (1994). A Mechanistics-Based Prediction Model for Thermal Cracking of Asphaltic Concrete Pavements (with Discussion). Journal of the Association of Asphalt Paving Technologists, 63, 81-117.

Mahoney, J.P., & Pierce, L.M. (1996). Examination of Washington State Department of Transportation Transfer Functions for Mechanistic-Empirical Asphalt Concrete Overlay Design. Transportation Research Record, 1539, 25-32.

Milestones 2002 Guide (Winter 2001). www.2002designguide.com.

Mohamed, E. H. H. (1994). Criteria for evaluation of rutting potential based on repetitive uniaxial compression test. Transportation Research Record, 1454, 74-81.

Monismith, C. & Epps J. (1969). Asphalt Mixture Behavior in Repeated Flexure. Transportation and Traffic Engineering. Berkeley, CA: University of California.

Monismith, C. (1992). Analytically Based Asphalt Pavement Design and Rehabilitation: Theory to Practice, 1962-1992. Transportation Research Record, 1354, 5-26.

National Asphalt Pavement Association [NAPA]. (N. D.) A Guide for Hot Mix Asphalt Pavement (TAS-30). [Computer Software]. Lanham, MD: Author.

National Cooperative Highway Research Program [NCHRP]. (2002, February). Experimental Plan for Calibration and Validation of Hot-Mix Asphalt Performance Models for Mix and Structural Design. Synthesis of Pavement

Performance Models for Structural and Mixture Design, Construction Specifications and Asset Management. Interim Task Activity Report 1-1/1. Project 9-30. [Draft]. NCHRP Project 9-30. United States: Author.

National Cooperative Highway Research Program [NCHRP]. (10/2001). Development Of The 2002 Guide For The Design Of New And Rehabilitated Pavement Structures [Part 2 – Design Inputs; Chapter 2: Material Characterization]. NCHRP Project 1-37A. United States.

NCHRP 2002 Design Guide [Computer software]. (2002). Springfield, IL: ERES.

National Highway Institute [NHI]. (1997). SUPERPAVE for the Generalist Engineer And Project Staff [Course No. 13152]. Arlington: NHI.

NHI. (1999). Introduction to Mechanistic Design for New and Rehabilitated Pavements [Course Document]. Arlington: NHI.

Newcomb, D. & Timm, D. (2001). Mechanistic Pavement Design: The Next Wave. Hot Mix Asphalt Technology, Vol. 6, Num. 5, 49-51.

Pierce, L. M., Jackson, N. C., & Mahoney J. P. (1993). Development and Implementation of a Mechanistic, Empirically Based Overlay Design Procedure For Flexible Pavements. Transportation Research Record, 1388, 120-128.

Roberts, F. L., Kandhal, P. S., Brown, E. R., Lee, D. Y., & Kennedy, T. W. (1991). Hot Mix Asphalt Materials, Mixture Design, and Construction (1st ed.). Maryland: NAPA Education Foundation.

Robinson, Casey W. An Evaluation of the AASHTO 2002 Mechanistic-Empirical Design Guide for Flexible Pavements. Masters Thesis. Civil Engineering Department, Auburn University. August, 2003.

Sayers, M. W., & Karamihas S. M. (1998). The Little Book of Profiling: Basic Information About Measuring and Interpreting Road Profiles. Michigan: University of Michigan.

Shahin, M. Y. (2002). Pavement Management for Airports, Roads, and Parking Lots. Boston: Kluwer Academic Publishers.

Shen, W., & Kirkner, D. J. (2001). Thermal Cracking of Viscoelastic Asphalt-Concrete Pavement. Journal of Engineering Mechanics, 700-709.

Texas Department of Transportation (TxDOT) (09/2001). Traffic Data and Analysis Manual (pp. 1-55 - 1-58). Texas: TxDOT.

Theyse, H. L., De Beer, M., & Rust, F. C. (1996). Overview of South African Mechanistic Pavement Design Method. Transportation Research Record, 1539, 6-17.

Thompson, M. R. (1987). ILLI-PAVE Based Full-Depth Asphalt Concrete Pavement Design Procedure. Proceedings of the Sixth International Conference on Structural Design of Asphalt Pavements (pp. 13-22). Ann Arbor, MI: The University of Michigan.

Timm, D. H. (1996). Investigation of Wheel Wander Effects on ESAL Calculations (University of Minnesota, 1996)

Timm, D. H., Newcomb, D. E., & Birgisson, B. (1999). Mechanistic-Empirical Flexible Pavement Thickness Design – The Minnesota Method. Minneapolis: Minnesota Department of Transportation.

Timm, D. & Newcomb, D. (2002). Lies, Damned Lies, and Traffic Forecasting. Hot

Mix Asphalt Technology, 7(5), 14-21.

Yoder, E. J., & Witczak, M. W. (1975). Principles of Pavement Design (2^{nd} ed.) New York: John Wiley & Sons, Inc.

Zhang, Z., Leidy, J., Kawa, I., & Hudson, W. R. (2000). Impact of Changing Traffic Characteristics and Environmental Conditions on Flexible Pavements. Transportation Research Board.