# CAL/APT PROGRAM ─ COMPARISON OF CALTRANS AND AASHTO PAVEMENT DESIGN METHODS

Report Prepared for

**CALIFORNIA DEPARTMENT OF TRANSPORTATION**

by

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16. Abstract


This report compares the Caltrans and AASHTO pavement thickness design procedures. The design comparisons include pavement structures subjected to a range in traffic, as represented by Traffic Indexes of 7, 9, 11, 13, and 15, and a range in subgrade strengths, as measured by subgrade R-values of 5, 20, and 40.

This report illustrates that the AASHTO and Caltrans pavement thickness design procedures do not produce the same pavement structures for the same given inputs. The design procedures are based on different material properties determined in the laboratory: The Caltrans procedure uses the R-value test and AASHTO uses the resilient modulus test. Generally, the pavement structures designed by the Caltrans procedure are thicker than those designed by the AASHTO procedure. This increase in thickness results in improved fatigue performance for the pavement designed according to the Caltrans procedure.

The fatigue performance of the pavements is extremely sensitive to the asphalt concrete thickness. In this report, it is shown that due to the differences between the design procedures they should not be used interchangeably. It is also shown that for the subbase, the procedures are sensitive to the conversion from one type of laboratory test to another.


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## DISCLAIMER

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## FINANCIAL DISCLOSURE STATEMENT

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This report presents the results of an investigation comparing pavement structures designed with the Caltrans and AASHTO design methods, and performance predictions for both sets of designs. The report presents an analysis of the results and conclusions with implications for Caltrans pavement design practices. It also presents a plan for an improved pavement design method based on mechanistic-empirical procedures.

## IMPLEMENTATION STATEMENT

The results of the analyses presented in this report indicate that the Gravel Factors for asphalt concrete included in the current Caltrans flexible pavement design method should be re-evaluated. The Gravel Factors should probably be adjusted to provide thicker asphalt concrete layers, especially for pavements expected to carry large volumes of truck traffic. Implementation of new Gravel Factors that result in thicker asphalt concrete layers should significantly improve the performance of heavy-duty pavements and result in reduced maintenance costs and longer periods between required rehabilitations. Re-calculation of Gravel Factors requires a database containing pavement structure and performance data, which Caltrans currently doesn't have. This indicates either the need for a mechanistic-empirical design method, or waiting years to develop an adequate database.

The current Caltrans flexible pavement design method treats asphalt concrete as a generic material. Simulations of pavement performance reported herein indicate significant differences in predicted fatigue life depending upon asphalt concrete mix. Movement from the current empirical design method to a mechanistic-empirical method is recommended based on these and other analyses in this report. Implementation of a mechanistic-empirical design method will result in better estimation of pavement performance by taking into account materials selection, layer thickness, subgrade strength, drainage, and loading characteristics that cannot be included in the current method. Use of a mechanistic-empirical method should result in significant improvements in pavement design, including savings from selection of more economical combinations of materials and layer thicknesses, and improved programming of maintenance and rehabilitation funds from better estimates of pavement performance. A plan for implementation of a mechanistic-empirical design method is included in this report.

The current Caltrans design method assumes that inclusion of positive drainage systems, including treated permeable base, edge drains, and outlets, will improve pavement performance. Results presented in a separate CAL/APT report on inclusion of asphalt treated permeable base in flexible pavements indicate that this may not always be the case. The effects of drainage should be directly investigated by means of laboratory testing at different levels of saturation and included in the pavement design method. A mechanistic-empirical design method will facilitate inclusion of this drainage information in the design method. These results should be included in the design method software as well. Similarly, lifecycle cost calculations are required for most Caltrans pavement designs, and should be included in the design software. The current software (NEWCON90) only calculates initial construction cost. These changes will result in better calculations of pavement performance and lifecycle cost.

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## EXECUTIVE SUMMARY

This report compares the Caltrans and AASHTO pavement thickness design procedures. The design comparisons include pavement structures subjected to a range in traffic, as represented by Traffic Indexes of 7, 9, 11, 13, and 15, and a range in subgrade strengths, as measured by subgrade R-values of 5, 20, and 40.

This report has four objectives:

1. Quantify the differences in pavement thickness resulting from use of the two methods.
2. Examine differences in predicted pavement performance for pavement designs considered equal within the Caltrans method. Related to this objective is the examination of the Gravel Factors for aggregate base and asphalt concrete.
3. Evaluate the effect of assumed drainage conditions on the pavement structures designed using the AASHTO method and relate this effect to the Caltrans method.
4. Demonstrate the flexibility of the mechanistic-empirical design procedure developed as part of the CAL/APT program to quantitatively, systematically, and rationally permit pavement designers to evaluate the performance of different pavement structures and different materials.

This report illustrates that the AASHTO and Caltrans pavement thickness design procedures do not produce the same pavement structures for the same given inputs. The design procedures are based on different material properties determined in the laboratory: The Caltrans procedure uses the R-value test and AASHTO uses the resilient modulus test ($M_R$). Generally, the pavement structures designed by the Caltrans procedure are thicker than those designed by the AASHTO procedure. This increase in thickness results in improved fatigue performance for the pavement designed according to the Caltrans procedure. The fatigue performance of the pavements is extremely sensitive to the asphalt concrete thickness. In this report, it is shown that due to the differences between the design procedures they should not be used interchangeably. It is also shown that for the subbase, the procedures are sensitive to the conversion from one type of laboratory test to another.

The relative contribution of asphalt concrete, granular base, and subbase materials to a pavement structure's load carrying capability is different for the two design procedures. Results presented indicate that the structural contribution of the asphalt concrete to fatigue cracking resistance for thicker asphalt concrete layers is larger than indicated by the Caltrans Gravel Factors. It is therefore recommended that the Gravel Factors for asphalt concrete be re-evaluated.

The predicted fatigue performance of two Caltrans design options, "lowest cost" and "thinnest asphalt concrete layer allowed," are significantly different for most inputs of Traffic Index (TI) and subgrade R-value. It is therefore likely that pavements designed using the Caltrans procedure, which are supposed to have similar performance as measured by the Gravel Equivalent (GE), will exhibit different fatigue performance: Thicker asphalt concrete layers will exhibit better performance than thinner asphalt concrete layers.

Considerations of pavement drainage and lifecycle cost analyses are included in the AASHTO procedure but not in the Caltrans procedure. The Caltrans procedure assumes that pavement designs without special drainage priorities are adequate and that their inclusion makes Caltrans pavement designs conservative. Both drainage conditions and lifecycle cost analyses should be explicitly included in the Caltrans design method.

Predicted pavement performance is also sensitive to different asphalt concrete mixes. Two mixes were evaluated with AR-4000 binders, one from a California Valley source and the other from a California Coastal source. The California Coastal binder performed better in all the structures analyzed.

Using the University of California Berkeley fatigue analysis and design procedure, the fatigue performance predictions suggest that the Caltrans "lowest cost" pavements are adequate at a 90-percent reliability level for all Traffic Indices and subgrade R-values of 20 and 40. However, for a subgrade R-value of 5, the Caltrans "lowest cost" pavement designs may not be adequate. Moreover, the results indicate that all Caltrans "thinnest allowable asphalt concrete layer" designs are likely not adequate at the 90-percent reliability level. It would also appear that AASHTO pavement designs may not be adequate at this reliability level. Substitution of the Coastal asphalt mix for the Valley asphalt mix did not change these conclusions.

This report demonstrates the ability of the mechanistic-empirical pavement analysis and design procedure to quantitatively evaluate the effects of pavement structure, materials selection, and subgrade strength on a specific mode of pavement distress. It is recommended that Caltrans move towards implementation of a mechanistic-empirical fatigue analysis and design procedure for the design of asphalt concrete pavements. An implementation procedure is recommended.

## 1.0 INTRODUCTION

The majority of state highway agencies in the United States use flexible pavement design procedures that are essentially empirical, i.e., determination of the structural equivalencies of pavement materials and selection of thicknesses of the pavement components are based on observed performance of pavements. As conditions change, the procedures are modified to reflect these changes. Many state highway agencies use the design procedure originally developed from results of the American Association of State Highway Officials (AASHO) Road Test. (*1*) That procedure, expanded and updated at regular intervals, is now referred to as the American Association of State Highway and Transportation Officials (AASHTO) procedure following the name change of the organization. The California Department of Transportation (Caltrans) also uses an empirical pavement design method. Although both methods are empirically based, for the same inputs of materials and traffic they result in pavements with different thicknesses.

Because of the direct correlation of pavement thickness and traffic, it should be expected that empirical design procedures developed from different databases would produce different performances when variables that are not design inputs are different. These variables include, but are not limited to, different environmental conditions and different subgrade soils.

### 1.1 Objectives

The first objective of this report is to quantify the differences in pavement thickness resulting from use of the Caltrans and AASHTO methods. Bases for this objective include the following:

1. Illustration of the limitations of design procedures when used for a wide variety of variables affecting pavement performance. For example, if there are differences between structures designed with a procedure based on environmental conditions in Illinois (AASHTO) and a procedure based on California conditions, there should also be differences for the wide range of environments found within California. This is particularly important for flexible pavements, which rely on a temperature-sensitive material such as asphalt concrete.
2. The range of performance observations contained in the two design methods provides a reference against which mechanistic-empirical design procedures can be calibrated. It is therefore important to quantify differences between those sets of observations. Eventually, further calibration of the relationships between stresses and/or strains calculated from models of the pavement structure (the mechanistic part of mechanistic-empirical methods) and pavement performance should be made using accelerated pavement tests and long-term pavement performance data.
3. Some engineers who are not well experienced in the pavements area may consider the Caltrans and AASHTO empirical design procedures to be interchangeable and that they should produce similar design thicknesses. At times this can lead to disagreements as to a required design. For example, if a private toll road is designed by the AASHTO method and is to be accepted by a California public agency that typically uses Caltrans designs, that agency should expect differences in the performance from the two methods. It is also important to quantify differences in pavement designs produced by the two methods for different design conditions, considering that both methods, and particularly the AASHTO method, involve

> considerable extrapolation for traffic levels above a Traffic Index between about 11 and 12 (8,000,000 ESALs), and for more than a few subgrade soils types. (*1*)

The second objective is to examine differences in predicted pavement performance for pavement designs considered equal within the Caltrans method. This objective permits examination of the materials structural equivalencies, or Gravel Factors (GF), for the materials considered in the Caltrans procedure. The performance predictions are made using the mechanistic-empirical method for fatigue analysis and design developed by UCB for Caltrans. (*2*)

The third objective is to evaluate the effect of assumed drainage conditions on pavement structures designed using the AASHTO method. The Caltrans method assumes a worst case scenario including poor drainage and a saturated subgrade as represented by the conditions for the R-value test. Language within the Caltrans method indicates that pavement performance should be improved by inclusion of drainage features in the pavement. (*3*) However, the method provides no change in pavement thickness design when drainage features, such as edge drains and drainage layers (typically asphalt treated permeable base [ATPB]), are included in the design. The AASHTO method provides a rudimentary indication of the effect of inclusion of pavement drainage features on pavement thickness design.

The fourth objective is to demonstrate the flexibility of mechanistic-empirical design procedures to quantitatively, systematically, and rationally permit pavement designers to evaluate the performance of different pavement structures and different materials. In this report, two typical California asphalt mixes are included in the analyses as examples.

The ability of mechanistic-empirical procedures to permit Caltrans policy-makers to evaluate the effects of potential changes in factors affecting pavement performance and infrastructure investment needs is demonstrated. For example, although only one tire pressure and axle wheel type (single axle, dual wheels, 690 kPa [100 psi] contact pressure) is included in the analyses in this report, the same analyses to evaluate the effects of wide-base single wheels (super singles) or changes in typical tire pressures (e.g., to 800 kPa [115 psi]) could be performed in a short period of time. The introduction of new materials to Caltrans pavements can also be quickly evaluated with some laboratory testing and the mechanistic-empirical procedure used in this report.

Demonstrations of this approach have already been reported to Caltrans. These demonstrations include evaluation of the effects of construction variables, such as asphalt concrete compaction (*2*); use of ATPB as a structural material in flexible pavements under dry and wet conditions (*4*); and development of rational QC/QA pay factors. (*5*)

### 1.2 Overview of Report

Chapter 2 of this report provides a brief discussion of the Caltrans and AASHTO design methods, including fundamental concepts and some of the major assumptions. Chapter 3 discusses the thickness design matrix, including the assumed materials and the correlations between the different materials classifications used by the two design procedures. Chapter 4 presents thickness designs resulting from the two procedures and predicts fatigue performance for the sections using the thickness designs. Chapter 5 includes a summary and evaluation of the analyses and recommendations resulting from the evaluation of the information.

It should be noted that the non-metric version of the Caltrans design procedure was used for this study because the metric version was not yet available to UCB when this study was begun. It is assumed that the results from the metric version will be approximately the same.

## 2.0 SUMMARY OF DESIGN METHODOLOGIES

Both the AASHTO (*1*) and the Caltrans (*3*) design methods are based on a similar concept and are developed from empirical data. This section presents a brief overview of each design method, key assumptions, and important differences as well as an indication of similarities.

The concept on which both design procedures are based is that each layer (or material) is characterized by a value representing its structural equivalency relative to the other layers. The sum of the products of structural equivalencies and layer thicknesses gives a single number that is used as a check to ensure that the pavement can adequately withstand the traffic demand for the subgrade strength. AASHTO uses layer coefficients and the total pavement structural capacity is referred to as the "Structural Number" (*SN*). Caltrans uses "Gravel Factors" (*Gf*) to quantify structural equivalency and a "Gravel Equivalent" (*GE*) for the pavement structural capacity.

### 2.1 AASHTO Design Procedure

The AASHTO procedure is based on performance data obtained from the AASHO Road Test conducted in Ottawa, Illinois, from 1956 to 1960. Modifications have been made to improve the design guide based on research completed and experience gained since the initial implementation of the guide in 1972. Because the procedure was developed from one test track, an important limitation is that the database is based on one set of materials, limited pavement types, and limited loads and load types. Aging is limited and the environmental influences on the data are limited to that specific environment. Detailed discussion of these factors and their limitations is found in the AASHTO guide. (*1*)

The AASHTO method uses a Structural Number (*SN*) to represent the designed pavement structure, and each layer is characterized by a structural coefficient (*ai*). The layer coefficients were developed initially from performance information for the AASHO Road Test and then later related to moduli and the associated stresses and strains produced by multi-layered pavement analyses (similar to the UCB procedure).

The AASHTO procedure makes use of the empirically developed expression (Equation 2.1) to predict the amount of traffic that can be sustained before the pavement deteriorates to a specified terminal level of serviceability, which can be chosen by the designer.

$$\log(W_{18}) = Z_R \bullet S_0 + 9.36 \bullet \log(SN+1) - 0.20 + \frac{\log \frac{\Delta PSI}{4.2-1.5}}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \bullet \log(M_R) - 8.07 \qquad \textbf{(2.1)}$$

where $W_{18}$ = predicted number of 18-kip (80kN) equivalent single axle load applications (ESALs),

$Z_R$ = standard normal deviate,

$S_0$ = combined standard error of the traffic prediction and performance prediction,

$\Delta$ *PSI* = difference between the initial design serviceability index, p0, and the design terminal serviceability index, pt, and

$M_R$ = resilient modulus (psi) of subgrade.

In order to balance equation 2.1, a Structural Number (SN) calculated as a function of pavement layer thicknesses, must be selected as follows:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad \textbf{(2.2)}$$

where $a_{\mathrm{i}}$ = *i*th layer coefficient,

$D_{\mathrm{i}}$ = *T*h layer thickness (inches) and

$m_i$ = $i$th layer drainage coefficient.

Determination of the demand traffic ESALs from the expected traffic loading on the facility is obtained from tables of axle load equivalency factors which are a function of the axle configuration and loads, structural number, and terminal serviceability (1). In a number of instances, the equivalencies can be approximated by the expression:

$$W_{ESAL} = \left( \frac{W_{load}}{80} \right)^n \quad \textbf{(2.3)}$$

where: $W_{ESAL}$ = repetitions of an 80 kN single axle developing same pavement damage as one repetition of a single axle with a load = Wload (kN), and

$n$ = approximately 4 for a range in loads.

This exponent is actually a function of pavement type and therefore changes in the pavement type can have a significant effect on the pavement performance. Accordingly, it is more appropriate to use the tables in Reference (*1*) to deference the exponent.

The AASHTO design procedure also incorporates reliability to account for some of the risk factors involved in designing pavements., for example, the traffic prediction, material variability, and construction variability. The method also accounts for seasonal variation of the subgrade modulus.

The design process consists of selecting the layer thicknesses to obtain the Structural Number required to obtain the required load applications (demand traffic). This process is begun at the layer just above the subgrade and then repeated for each layer working towards the surface of the pavement.

Lifecycle costs are calculated in the procedure and are used as the basis for selection of a pavement structure from pavements with the same Structural Number.

## 2.2 Caltrans Design Procedure

The Caltrans design procedure (*3*) is based on theory, test track studies, experimental sections, materials research, and empirical evidence. (*6*) Data obtained from the AASHO Road Test were used to update the design method. (*7, 8*) The method references the structural equivalency of each layer, called the Gravel Factor ($G_f$), to the aggregate subbase. The Gravel Equivalent ($GE$) of the pavement is calculated using the relationship:

$$GE = GE_{AC} + d_{AB} \bullet G_{f_{AB}} + d_{SB} \tag{2.4}$$

where $d_{AB}$ and $d_{SB}$ are the thicknesses of the aggregate base and subbase layers (in feet) respectively, and $G_{f_{AB}}$ is the Gravel Factor for the aggregate base. $GE_{AC}$ is calculated using the following relationship:

$$GE_{AC} = \frac{5.67 d_{AC}}{TI^{0.5}} \text{ for } d_{AC} \le 0.50 \text{ feet} \tag{2.5}$$

$$GE_{AC} = \frac{7.00 (d_{AC})^{4/3}}{TI^{0.5}} \text{ for } d_{AC} \le 0.50 \text{ feet} \tag{2.6}$$

where $d_{AC}$ is the asphalt concrete layer thickness (feet) and TI is the design Traffic Index. The required $GE$ for the pavement is determined from the expected traffic during the design life and the R-value of the subgrade soil, as shown below:

$$GE = 0.0032 \bullet TI \bullet (100 - R) \tag{2.7}$$

In contrast to the AASHTO design procedure, the Caltrans design method first calculates the thickness of the top layer and then works down layer by layer. The effect of drainage is not directly incorporated, except that the design guide states that the inclusion of positive drainage (a drainage layer) will probably result in the service life exceeding the design life. In the R-value test, unbound soils and granular materials are tested in a near saturated condition. It is likely that many materials do not reach this weakened condition in California pavement sections.

The Caltrans guide states that the pavement selection should be based on the most economical design, considering the total lifecycle costs of the facility. The lifecycle costs should include the initial cost, maintenance cost, and anticipated rehabilitation costs during the selected lifecycle period. If the TI is greater than 10 (approximately 2,500,000 ESALs), then a total lifecycle cost analysis must be performed, as detailed in the design manual. However, the Caltrans design guide software program used to perform the pavement thickness calculations only calculates initial cost and does not calculate lifecycle costs. (*9*)

It would be relatively simple for lifecycle costs to be incorporated into the calculations in the Caltrans software. Doing so would enhance the use of the software and bring it into agreement with the stated design procedure and policy of Caltrans included in the text of the design guide.

Conversion of the traffic loading spectrum to ESALs is performed using the load equivalence factor (Equation 2.3) with the exponent $n$ equal to 4.2. The Caltrans exponent of 4.2 results in the Caltrans load equivalence being more sensitive than the AASHTO load equivalence factors..

## 3.0 THICKNESS DESIGN MATRIX

To compare the two design procedures, a thickness design matrix was established that incorporated a number of variables, including traffic, materials, reliability, and cost. In this section the variables used, the assumptions made, and the necessary conversions between the different specifications associated with each design method are discussed.

The matrix utilized is summarized in Table 3.1. The first block contains the combinations of traffic, materials, and drainage used to determine both pavement layer thicknesses and fatigue life predictions. The second block summarizes the cases used in the drainage study and the third block gives the cases used for determination of the effect of assumed subbase modulus on pavement thickness in the AASHTO method.

### 3.1 Traffic

Five design traffic levels, shown below, are used in this study and span the range from low to high levels of traffic on typical Caltrans highway pavements. The Caltrans design method requires that the design traffic be described in terms of a Traffic Index (TI) and the AASHTO design method requires equivalent standard axles (ESALs). The following equation from the Caltrans design procedure (3) is used to convert between TI and ESALs:

$$TI = 9.0 \bullet \left( \frac{ESAL}{10^6} \right)^{0.119} \qquad \textbf{(3.1)}$$

## Table 3.1 Thickness design matrix of pavement structures

| Traffic Index | ESALs | Subgrade R-value | Assumed Subgrade Modulus (MPa) | Subbase Modulus (MPa) | AASHTO Drainage factor (m) | Caltrans Lowest Cost | Caltrans Thinnest AC | AASHTO | Thickness Determined | Fatigue Life Predicted |
|---|---|---|---|---|---|---|---|---|---|---|
| **General Study** | | | | | | | | | | |
| 7 | 120,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | X |
| 7 | 120,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | X |
| 7 | 120,000 | 40 | 161 | 103 | 1.0 | X | X | X | X | X |
| 9 | 1,000,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | X |
| 9 | 1,000,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | X |
| 9 | 1,000,000 | 40 | 161 | 103 | 1.0 | X | X | X | X | X |
| 11 | 5,400,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | X |
| 11 | 5,400,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | X |
| 11 | 5,400,000 | 40 | 161 | 103 | 1.0 | X | X | X | X | X |
| 13 | 22,000,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | X |
| 13 | 22,000,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | X |
| 13 | 22,000,000 | 40 | 161 | 103 | 1.0 | X | X | X | X | X |
| 15 | 73,160,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | X |
| 15 | 73,160,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | X |
| 15 | 73,160,000 | 40 | 161 | 103 | 1.0 | X | X | X | X | X |
| **Drainage Study** | | | | | | | | | | |
| 9 | 1,000,000 | 5 | 27 | 103 | 0.6 | | | X | X | |
| 9 | 1,000,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | |
| 9 | 1,000,000 | 5 | 27 | 103 | 1.3 | | | X | X | |
| 9 | 1,000,000 | 20 | 84 | 103 | 0.6 | | | X | X | |
| 9 | 1,000,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | |
| 9 | 1,000,000 | 20 | 84 | 103 | 1.3 | | | X | X | |
| 13 | 22,000,000 | 5 | 27 | 103 | 0.6 | | | X | X | |
| 13 | 22,000,000 | 5 | 27 | 103 | 1.0 | X | X | X | X | |
| 13 | 22,000,000 | 5 | 27 | 103 | 1.3 | | | X | X | |
| 13 | 22,000,000 | 20 | 84 | 103 | 0.6 | | | X | X | |
| 13 | 22,000,000 | 20 | 84 | 103 | 1.0 | X | X | X | X | |
| 13 | 22,000,000 | 20 | 84 | 103 | 1.3 | | | X | X | |
| **Subbase Modulus Study** | | | | | | | | | | |
| 7 | 120,000 | 5 | 27 | 83 | 1.0 | | | X | X | |
| 7 | 120,000 | 5 | 27 | 103 | 1.0 | | | X | X | |
| 7 | 120,000 | 5 | 27 | 138 | 1.0 | | | X | X | |
| 9 | 1,000,000 | 5 | 27 | 83 | 1.0 | | | X | X | |
| 9 | 1,000,000 | 5 | 27 | 103 | 1.0 | | | X | X | |
| 9 | 1,000,000 | 5 | 27 | 138 | 1.0 | | | X | X | |
| 11 | 5,400,000 | 5 | 27 | 83 | 1.0 | | | X | X | |
| 11 | 5,400,000 | 5 | 27 | 103 | 1.0 | | | X | X | |
| 11 | 5,400,000 | 5 | 27 | 138 | 1.0 | | | X | X | |
| 13 | 22,000,000 | 5 | 27 | 83 | 1.0 | | | X | X | |
| 13 | 22,000,000 | 5 | 27 | 103 | 1.0 | | | X | X | |
| 13 | 22,000,000 | 5 | 27 | 138 | 1.0 | | | X | X | |
| 15 | 73,160,000 | 5 | 27 | 83 | 1.0 | | | X | X | |
| 15 | 73,160,000 | 5 | 27 | 103 | 1.0 | | | X | X | |
| 15 | 73,160,000 | 5 | 27 | 138 | 1.0 | | | X | X | |

The five TIs and corresponding ESALs included in the experiment are shown in Table 3.2.

## Table 3.2 Traffic Index Values and Corresponding ESALs

| Traffic Index (TI) | ESAL Range | Representative ESALs |
|---|---|---|
| 7 | 89,800 to 164,000 | 120,000 |
| 9 | 798,000 to 1,270,000 | 1,000,000 |
| 11 | 4,500,000 to 6,600,000 | 5,400,000 |
| 13 | 18,900,000 to 26,100,000 | 22,000,000 |
| 15 | 64,300,000 to 84,700,000 | 73,160,000 |

## 3.2 Pavement Materials

The pavement structures used in this analysis are assumed to consist of an asphalt concrete surface layer, an aggregate base, an aggregate subbase, and the subgrade. In some cases, the aggregate subbase is eliminated because it is as strong as the subgrade or it is less than 150 mm (6 inches) thick. The materials used in the analysis are assumed to meet standard Caltrans specifications. Because of different ways of defining materials response in the two methods, it is necessary to convert materials properties between them in terms of results from their respective reference tests: R-value for Caltrans and resilient modulus ($M_R$) for AASHTO.

### 3.2.1 Subgrade

Three subgrades were used in the analysis, with R-values of 5, 20, and 40. These R-values span a wide range in subgrade strength. Elastic moduli were estimated to be 27, 84, and 161 MPa (3,850, 12,200, and 23,400 psi) for R-values of 5, 20, and 40, respectively. (*10*) The conversion included in the AASHTO design guide gives elastic moduli very close in magnitude to the moduli assumed for this analysis.

### 3.2.2 Subbase

Only one subbase was used in this study, classified as a Class 2 aggregate subbase according to Caltrans specifications. A Class 2 subbase has a minimum R-value of 50. The Caltrans Gravel Factor for this material is 1.0. According to the AASHTO design guide, a subbase with an R-value of 50 corresponds to a modulus of approximately 83 MPa (12,000 psi). However, based on past experience, this modulus is considered too low for typical Caltrans subbases and was adjusted to 138 MPa (20,000 psi). The conversions from R-value to modulus in the AASHTO guide are averaged from correlations obtained in California, New Mexico, and Wyoming; it was therefore considered acceptable to refine, based on experience with the HVS test program, the moduli estimated from the AASHTO design guide to better reflect California conditions. However, to ensure an equitable comparison between the Caltrans and AASHTO thickness designs, three values of the subbase modulus were considered: 83, 103, and 138 MPa (12,000; 15,000; and 20,000 psi). In the mechanistic-empirical prediction of fatigue life presented in Chapter 5, a subbase modulus of 103 MPa (15,000 psi) is used.

The AASHTO method requires a structural coefficient, a function of modulus, as the input for the material layers. For granular subbases, the following equation is used (*1*) where $a_{SB}$ is the subbase structural coefficient and $E_{SB}$ is the subbase modulus (psi):

$$a_{SB} = 0.227(\log_{10} E_{SB}) - 0.839 \qquad \textbf{(3.2)}$$

For the subbase moduli of 83, 103, and 138 MPa (12,000; 15,000; and 20,000 psi), the structural coefficients are 0.09, 0.11, and 0.14, respectively.

### 3.2.3 Aggregate Base.

The aggregate base is assumed to meet the requirements of a Caltrans Class 2 aggregate base. This base has a minimum R-value of 78. The Caltrans Gravel Factor is 1.1. Based on past experience, (*2*) the modulus of the base is assumed to depend in part on the asphalt concrete thickness as shown in Table 3.3. The AASHTO structural coefficient for the base is also shown in Table 3.3 and is determined using the following equation where $a_{AB}$ is the structural coefficient and $E_{AB}$ is the aggregate base modulus (psi). (*1*)

$$a_{AB} = 0.249(\log_{10} E_{AB}) - 0.997 \qquad \textbf{(3.3)}$$

**Table 3.3 Aggregate Moduli and Structural Coefficient**

| Asphalt Concrete Thickness | Class 2 Aggregate Base | |
|---|---|---|
| | Elastic modulus | AASHTO Structural Coefficient |
| 90 to 183 mm | 207 MPa (30,000 psi) | 0.14 |
| 195 to 305 mm | 172 MPa (25,000 psi) | 0.12 |
| 305 to 396 mm | 138 MPa (20,000 psi) | 0.09 |

3.2.4 Asphalt Concrete

Two asphalt concrete materials were included in the experiment with stiffnesses taken from laboratory flexural beam test measurements. Both mixes included Watsonville granite aggregate with a gradation between the Caltrans medium and coarse 19-mm specifications. (*2*) The two asphalts both met AR-4000 specifications and were refined from California Coastal and California Valley sources. For the two mixes, relationships were developed from laboratory testing to estimate the stiffness of the mix for different air-void and asphalt contents. (*2,11,12*) These relationships for a 20° Celsius test temperature and 10-Hz sinusoidal loading are:

$$Valley\,/\,Watsonville:\ \ln S_0 = 10.282 - 0.172\,AC - 0.076\,AV \tag{3.4}$$

$$Coastal\,/\,Watsonville:\ \ln S_0 = 8.5270 - 0.12224\,AV \tag{3.5}$$

where ln is the natural logarithm, $S_0$ = initial flexural stiffness (MPa), and $AC$ and $AV$ are the asphalt content and air-void content (percent), respectively. For the mix with the Coastal asphalt, asphalt content was found to be statistically insignificant. Using these equations and an assumed air-void content of 8 per cent (about 96 percent relative compaction in the Caltrans standard test method CTM 304), and an assumed asphalt content of 5 percent by mass of aggregate, the Valley mix has a stiffness of approximately 6,760 MPa and the Coastal mix a value of approximately 1,900 MPa. The 5-percent asphalt content is approximately that which meets the Hveem mix design requirements for a minimum stability of 37 or minimum air-void content of 4 percent for Caltrans standard laboratory compaction (standard test method CTM 367).

Using the guideline in the AASHTO design manual, the asphalt concrete stiffnesses of 1,900 MPa and 6,760 MPa correspond to structural coefficients of approximately 0.35 and 0.42, respectively. For this analysis, the pavements were designed using a structural coefficient of 0.42 for the asphalt concrete; however, the mechanistic-empirical analysis component of the study utilizes the actual stiffnesses.

### 3.3 Reliability

The AASHTO method permits explicit input of the desired design reliability in the thickness design of the pavements. For this analysis, a reliability of 90 percent was used. The Caltrans method does not permit input of the design reliability; however, it is certain that some level of reliability must be implicitly included in the thickness design procedure. (*13*)

### 3.4 Costs and Economic Selection of Thicknesses

Both the Caltrans and AASHTO design procedures require unit costs for the materials. The unit costs selected for use in both procedures were arbitrarily taken from the default costs of the Caltrans procedure and are as follows: asphalt concrete, $70 per cubic yard; aggregate base, $35 per cubic yard; and aggregate subbase, $25 per cubic yard. (*9*) All are in-place values.

For the Caltrans procedure, determination of the most economic pavement is made by choosing the pavement structure with the lowest initial cost, an output of the software program. The AASHTO design software presents only one pavement thickness, which is based on an economic choice. The AASHTO software also permits the incorporation of lifecycle costs for a specified design period. However, to facilitate a fair comparison between the Caltrans and AASHTO procedures for this analysis, the interest rate was input as zero for the AASHTO software in order to prevent the calculation of the full lifecycle costs. This was done because the full lifecycle costs are not calculated in the Caltrans method.

The AASHTO method automatically selects the most economical design. However, if the ratio of costs between the top two layers is smaller than the ratio between the structural coefficients of the top two layers times the drainage coefficient, then the design selected is that with the minimum base thickness as the optimum economic alternative.

## 4.0 THICKNESS DESIGN COMPARISON

Two software programs were used to determine the thicknesses: NEWCON90 (*9*), the July 1990 version of Chapter 600 of the Caltrans Highway Design Manual (*3*); and, DNPS86 (*14*), the AASHTO design of New Pavement Structures Program, Version 1C, September 1986 (*1*).

### 4.1 Design Selection Criteria for This Experiment

A total of 90 pavements were designed using both the Caltrans and AASHTO procedures. For the Caltrans procedure, two thickness designs were selected:

- for 30 pavements, the design with the Caltrans "thinnest allowable asphalt concrete layer" was chosen (referred to as "thinnest asphalt concrete"), and
- for another 30 pavements, the lowest default cost design based on the default materials costs mentioned in Section 3.4 was chosen (referred to as "lowest cost").

For some of the cases, the Caltrans "thinnest asphalt concrete" design and the Caltrans "lowest cost" design were identical. The thicknesses for the remaining 30 pavements were designed according to the AASHTO procedure, which selects the lowest cost design.

Several rules were used for thickness selection in this analysis. The subbase was eliminated if the thickness is less than 105 mm (4.2 inches). This is the default minimum subbase thickness in the NEWCON90 program. A minimum aggregate base thickness of 150 mm (6 inches) was assumed due to the difficulties in constructing a base layer thinner than 150 mm.

The specified thicknesses were rounded to the nearest 0.05 ft., the unit in which Caltrans specifies pavement layer thicknesses.

A shortcoming of the AASHTO software in this analysis is that if the program specifies an aggregate base thickness less than the minimum (150 mm) and the thickness is manually reset to 150 mm, neither the subbase nor the asphalt concrete thickness is decreased to compensate for the increased base thickness. This occurred in a few of the cases.

Water infiltration in a pavement can severely affect performance by causing stiffness reduction in the pavement layers, particularly the unbound layers. Accelerated damage results from the reduced stiffness of these components; drainage, on the other hand, can reduce the potential for damage. Therefore, drainage has an important influence on pavement performance and should be taken into account by the design method. The AASHTO design procedure allows the consideration of drainage whereas the Caltrans procedure does not. However, the R-value test, a soaked test, is used to evaluate subgrade and other unbound soils materials and therefore the materials are conservatively characterized.

The criteria that are considered in the AASHTO procedure for drainage are the time within which the water will be removed from the pavement structure and the percent time the pavement structure is exposed to moisture levels approaching saturation. These drainage considerations are incorporated into the calculation of the Structural Number (*SN*) by applying a drainage coefficient (*m*), as shown in the following relationship:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{4.1}$$

where $SN$ = Structural Number,

$a_i$ = structural coefficients, and

$D_i$ = layer thicknesses.

The drainage coefficient is determined from the AASHTO design guide.

For most of the analyses included in this study, the drainage coefficient was assumed to be 1.0 for all layers. To investigate the effect of incorporating the drainage considerations into the thickness designs of the AASHTO pavements, four of the pavement structures were evaluated by varying the drainage inputs. The cases used for this small analysis are the TI 9 and 13 and R-value 5 and 20 cases, shown in Table 3.1. For both the base and the subbase, two drainage coefficients were used. In the first, the pavement drainage was assumed to be good, which means the water drains in one day and the pavement is exposed to moisture levels approaching saturation less than 1 percent of the time. The drainage coefficient for this scenario is 1.30. The second scenario assumed the pavement drainage was poor; for these conditions, the water is assumed to take 1 month to drain and the pavement is exposed to moisture levels approaching saturation more than 25 per cent of the time. The drainage coefficient for this scenario is 0.60.

The cases used for this small analysis are the TI 9 and 13 and R-value 5 and 20 cases, given in Table 3.1.

## 4.2 Thickness Designs, Results, and Analysis

The results from the thickness designs are presented and discussed in this section. Firstly, the effect of the selected subbase modulus on the thickness of the pavements designed by the AASHTO method is discussed; secondly, the pavement thicknesses from the AASHTO and Caltrans design procedures are compared; and thirdly, the structural coefficients (AASHTO) and Gravel Equivalents (*GE*) of the pavements are compared.

#### 4.2.1 Effect of Subbase Modulus on Thickness and Structural Equivalency of AASHTO Designed Pavements

Experience has indicated that Caltrans Class 2 subbase materials are typically stiffer than indicated by the AASHTO material conversions. For this reason, the effect of using stronger subbases in the thickness design had to be quantified to ensure that the Caltrans and AASHTO design procedures were equivalently compared in the main part of this study.

The thicknesses selected from the AASHTO design procedure using three subbase moduli, 83, 103, and 138 MPa (12,000; 15,000; and 20,000 psi), were compared by investigating the change in thickness of each layer and by comparing both the Structural Number and Gravel Equivalent of several pavements.

The pavement thicknesses for the pavements designed using the three moduli and the resulting increase or decrease in the layer thicknesses are shown in Table 4.1. In the table, the changes in the pavement thicknesses are all relative to the thickness of the pavement with a subbase modulus of 103 MPa (15,000 psi).

**Table 4.1 Effect of aggregate subbase modulus on layer thickness and gravel equivalent (AASHTO procedure)**

| Traffic Index | R-value | Subbase Modulus (MPa) | Layer Thickness (mm) AC | AB | ASB | Structural Number | Gravel Equivalent | Change* in Layer Thickness (mm) Asphalt Concrete | Aggregate Base | Aggregate Subbase | Change* in Gravel Equivalent |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7 | 5 | 83 | 80 | 152 | 240 | 3.03 | 1.70 | - | - | 44 | 0.14 |
| | | 103 | 80 | 152 | 196 | 3.03 | 1.56 | | | | |
| | | 138 | 80 | 152 | 155 | 3.03 | 1.43 | - | - | -41 | -0.13 |
| 9 | 5 | 83 | 117 | 152 | 414 | 2.80 | 2.45 | - | - | 75 | 0.25 |
| | | 103 | 117 | 152 | 339 | 2.80 | 2.20 | | | | |
| | | 138 | 117 | 152 | 265 | 2.76 | 1.96 | - | - | -74 | -0.24 |
| 11 | 5 | 83 | 155 | 196 | 501 | 5.39 | 3.06 | - | 44 | 39 | 0.29 |
| | | 103 | 155 | 152 | 462 | 5.39 | 2.78 | | | | |
| | | 138 | 155 | 152 | 366 | 5.39 | 2.46 | - | - | -96 | -0.31 |
| 13 | 5 | 83 | 210 | 226 | 562 | 6.54 | 3.63 | - | 73 | 23 | 0.34 |
| | | 103 | 210 | 153 | 539 | 6.54 | 3.29 | | | | |
| | | 138 | 195 | 152 | 466 | 6.54 | 2.98 | -15 | - | -73 | -0.31 |
| 15 | 5 | 83 | 255 | 251 | 623 | 7.60 | 4.12 | - | 80 | 26 | 0.37 |
| | | 103 | 255 | 171 | 597 | 7.60 | 3.75 | | | | |
| | | 138 | 240 | 152 | 533 | 7.64 | 3.40 | -15 | -19 | -64 | -0.34 |

* change relative to modulus = 103 MPa case; positive indicates increase, negative indicates decrease

For the thinner pavements (TI = 7, 9) the effect of changing the subbase modulus is to change only the thickness of the subbase. Typically this change is approximately 20 percent, either a 20 percent increase for the decreased modulus or a 20 percent decrease for the increased modulus.

As TI increases, and the base thickness exceeds the minimum (150 mm); the base thickness also changes. For TIs of 13 and 15, the effect of reducing the subbase modulus is to increase the base and subbase thicknesses by approximately 47 and 4 percent, respectively.

At a TI of 13 and a TI of 15, the asphalt concrete thickness decreases for the 138 MPa (20,000 psi) subbase modulus. This decrease is due to the reduction of the base thickness to the minimum, which necessitates adjustment of the asphalt concrete to obtain the required Structural Number. In both of these cases, the subbase thickness is also reduced.

In summary, the effect of the assumed subbase modulus for the AASHTO method using the three values of modulus included here (83, 103, and 138 MPa) was to change the thickness of some or all of the pavement layers. The changes are most likely significant and large enough to influence the mechanistic-empirical fatigue analysis. For the purpose of this report, it is sufficient to state that if a large modulus is assumed (e.g., the 138 MPa (20,000 psi) modulus), longer fatigue lives would be experienced. The modulus of 103 MPa (15,000 psi) was considered reasonable to use in the further analysis because 1) a modulus of 83 MPa (12,000 psi) is not representative of the Caltrans conditions, and 2) a modulus of 138 MPa (20,000 psi) is too large relative to the recommended AASHTO conversions and procedure, and will therefore not facilitate a reasonable comparison between the Caltrans and AASHTO design methods.

4.2.2 Pavement Thicknesses and Structural Equivalency.

The pavement thicknesses for both the Caltrans and AASHTO design procedures and their Structural Numbers and Gravel Equivalents are shown in Table 4.2. Typically an aggregate subbase is included in the pavements with subgrade R-values of 5 and 20. The cases, shown in Table 4.2, are also plotted against Structural Number and Gravel Equivalent in Figures 4.1 and 4.2, respectively.

The asphalt concrete layer of the AASHTO pavements is always thinner than the Caltrans "lowest cost" pavements. The asphalt concrete layer of the AASHTO pavements is also thinner than the Caltrans "thinnest AC" design asphalt concrete pavements for 9 of the 15 cases shown in Table 4.2; those designed for lower traffic volumes. If the AASHTO and Caltrans design methods are interchangeable, as is sometimes considered, then these three pavement designs should also be interchangeable, and should provide the same performance. The differences in layer thickness have a large influence on performance predicted by the UCB mechanistic-empirical procedure with the assumptions mentioned, as will be shown later in the report.

In Figure 4.1 it can be seen that the Structural Numbers of the Caltrans "thinnest AC" designs are smaller than those of the Caltrans "lowest cost" designs (Figure 4.2). The Gravel Equivalents of the two Caltrans designs are the same. This apparent inconsistency is due to the different structural coefficients used to determine the Structural Number and Gravel Equivalent. The structural coefficient for asphalt concrete ($a_{AC}$) in the AASHTO method is 0.42, and the Gravel Factor ($G_f$) of asphalt concrete in the Caltrans method varies from 1.46 to 2.54. The $a_{AB}$ and $G_f$ for the aggregate base layer are 0.12 or 0.14 (depending on asphalt concrete thickness) and

**Table 4.2 Summary of pavement layer thicknesses for Caltrans "lowest cost," Caltrans "thinnest asphalt concrete," and AASHTO designs**

| | | | Thickness (mm) | | | Structural Number | | Gravel Equivalent | |
|---|---|---|---|---|---|---|---|---|---|
| **TI** | **R value** | **Design** | **AC** | **AB** | **ASB** | **Required *** | **Actual** | **Required **** | **Actual** |
| 7 | 5 | Caltrans Lowest Cost | 122 | 152 | 213 | 3.01 | 3.78 | 2.13 | 2.11 |
| | | Caltrans Thinnest AC | 91 | 183 | 259 | 3.01 | 3.64 | 2.13 | 2.15 |
| | | AASHTO | 80 | 152 | 196 | 3.01 | 3.01 | 2.13 | 1.76 |
| | 20 | Caltrans Lowest Cost | 107 | 152 | 152 | 2.16 | 3.27 | 1.79 | 1.80 |
| | | Caltrans Thinnest AC | 91 | 183 | 152 | 2.16 | 3.17 | 1.79 | 1.80 |
| | | AASHTO | 80 | 152 | | 2.16 | 2.16 | 1.79 | 1.11 |
| | 40 | Caltrans Lowest Cost | 107 | 168 | | 2.16 | 2.70 | 1.34 | 1.36 |
| | | Caltrans Thinnest AC | 91 | 198 | | 2.16 | 2.60 | 1.34 | 1.35 |
| | | AASHTO | 80 | 152 | | 2.16 | 2.16 | 1.34 | 1.11 |
| 9 | 5 | Caltrans Lowest Cost | 137 | 213 | 335 | 4.24 | 4.89 | 2.74 | 2.72 |
| | | Caltrans Thinnest AC | 137 | 213 | 335 | 4.24 | 4.89 | 2.74 | 2.72 |
| | | AASHTO | 117 | 152 | 339 | 4.24 | 4.24 | 2.74 | 2.39 |
| | 20 | Caltrans Lowest Cost | 183 | 152 | 168 | 2.78 | 4.47 | 2.30 | 2.28 |
| | | Caltrans Thinnest AC | 137 | 213 | 213 | 2.78 | 4.36 | 2.30 | 2.32 |
| | | AASHTO | 117 | 152 | | 2.78 | 2.78 | 2.30 | 1.28 |
| | 40 | Caltrans Lowest Cost | 183 | 152 | | 2.78 | 3.74 | 1.73 | 1.73 |
| | | Caltrans Thinnest AC | 137 | 244 | | 2.78 | 3.61 | 1.73 | 1.73 |
| | | AASHTO | 117 | 152 | | 2.78 | 2.78 | 1.73 | 1.28 |
| 11 | 5 | Caltrans Lowest Cost | 229 | 152 | 411 | 5.41 | 6.28 | 3.34 | 3.34 |
| | | Caltrans Thinnest AC | 168 | 274 | 427 | 5.41 | 6.14 | 3.34 | 3.34 |
| | | AASHTO | 155 | 152 | 462 | 5.41 | 5.41 | 3.34 | 2.92 |
| | 20 | Caltrans Lowest Cost | 259 | 152 | 168 | 3.63 | 5.73 | 2.82 | 2.80 |
| | | Caltrans Thinnest AC | 168 | 274 | 259 | 3.63 | 5.41 | 2.82 | 2.79 |
| | | AASHTO | 155 | 192 | | 3.63 | 3.63 | 2.82 | 1.55 |
| | 40 | Caltrans Lowest Cost | 244 | 152 | | 3.41 | 4.75 | 2.11 | 2.12 |
| | | Caltrans Thinnest AC | 168 | 320 | | 3.41 | 4.54 | 2.11 | 2.11 |
| | | AASHTO | 155 | 152 | | 3.41 | 3.41 | 2.11 | 1.41 |
| 13 | 5 | Caltrans Lowest Cost | 274 | 168 | 503 | 6.54 | 7.50 | 3.95 | 3.94 |
| | | Caltrans Thinnest AC | 198 | 335 | 503 | 6.54 | 7.04 | 3.95 | 3.95 |
| | | AASHTO | 210 | 153 | 539 | 6.54 | 6.54 | 3.95 | 3.51 |
| | 20 | Caltrans Lowest Cost | 290 | 152 | 290 | 4.52 | 6.77 | 3.33 | 3.32 |
| | | Caltrans Thinnest AC | 198 | 335 | 305 | 4.52 | 6.18 | 3.33 | 3.30 |
| | | AASHTO | 210 | 221 | | 4.52 | 4.52 | 3.33 | 1.98 |
| | 40 | Caltrans Lowest Cost | 305 | 152 | | 4.20 | 5.76 | 2.50 | 2.49 |
| | | Caltrans Thinnest AC | 198 | 396 | | 4.20 | 5.14 | 2.50 | 2.52 |
| | | AASHTO | 210 | 153 | | 4.20 | 4.20 | 2.50 | 1.74 |
| 15 | 5 | Caltrans Lowest Cost | 335 | 152 | 594 | 7.61 | 8.83 | 4.56 | 4.55 |
| | | Caltrans Thinnest AC | 229 | 381 | 594 | 7.61 | 8.16 | 4.56 | 4.56 |
| | | AASHTO | 255 | 171 | 597 | 7.61 | 7.61 | 4.56 | 4.00 |
| | 20 | Caltrans Lowest Cost | 351 | 152 | 335 | 5.37 | 7.97 | 3.84 | 3.83 |
| | | Caltrans Thinnest AC | 229 | 381 | 381 | 5.37 | 7.24 | 3.84 | 3.86 |
| | | AASHTO | 255 | 246 | | 5.37 | 5.37 | 3.84 | 2.31 |
| | 40 | Caltrans Lowest Cost | 366 | 152 | | 5.02 | 6.77 | 2.88 | 2.86 |
| | | Caltrans Thinnest AC | 229 | 457 | | 5.02 | 5.95 | 2.88 | 2.88 |
| | | AASHTO | 255 | 171 | | 5.02 | 5.02 | 2.88 | 2.04 |

* Required Structural Number by AASHTO

** Required Gravel Equivalent by Caltrans

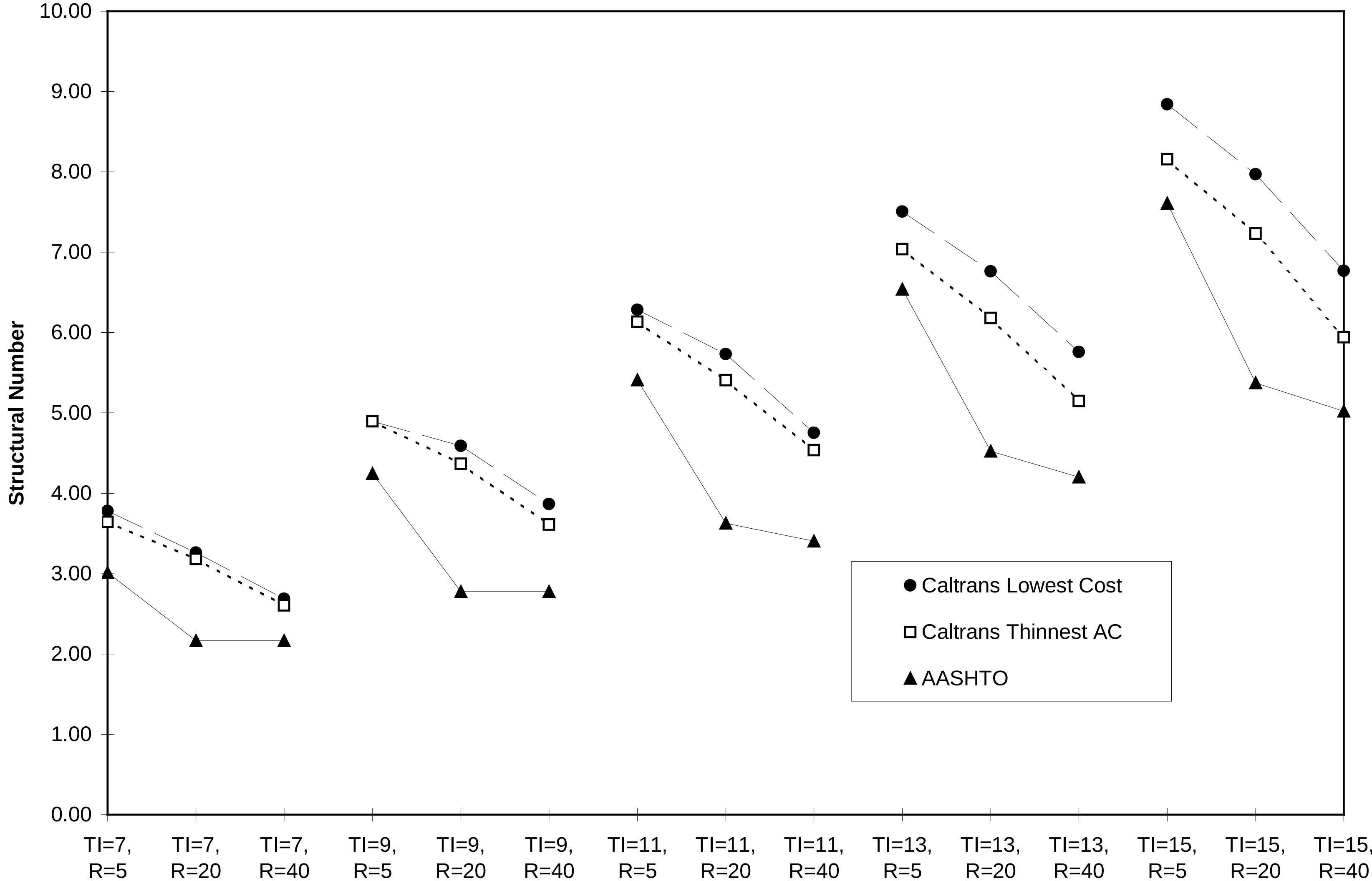


**Figure 4.1 Structural numbers (SN) for pavements designed by Caltrans and AASHTO procedures.**

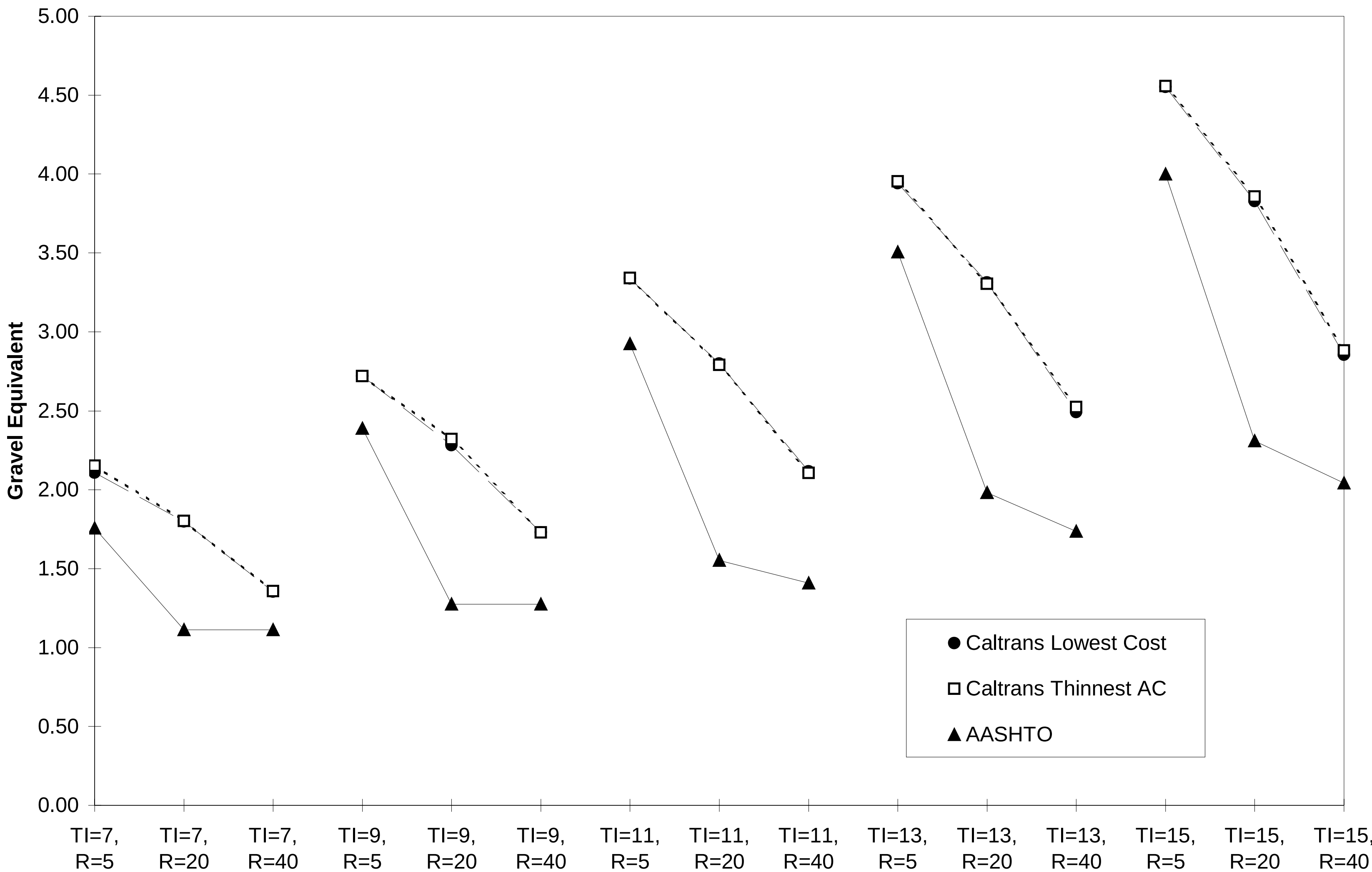


**Figure 4.2 Gravel equivalents (GE) for pavements designed by Caltrans and AASHTO procedures.**

1.1, respectively. The ratio of the asphalt concrete and aggregate base structural equivalencies is approximately 3.23 for the AASHTO procedure and 1.33 to 2.24 for the Caltrans procedure. These ratios show that the AASHTO method considers the asphalt concrete to add much more structural capacity than the aggregate base, whereas the Caltrans procedure assigns less structural capacity to the asphalt concrete layer.

The Gravel Equivalent is almost the same for the Caltrans "thinnest asphalt concrete" and "lowest cost" designs, which implies that these pavements will provide the same performance. As discussed in the next section, performance predictions indicate that these Caltrans "thinnest AC" pavements have shorter fatigue lives than the Caltrans "lowest cost" pavements, which suggests that the Gravel Factors may not be adequately proportioning the structural capacity of the pavement to each layer and material type.

The Structural Number for the AASHTO pavements is considerably smaller than the Caltrans pavements, indicating that the AASHTO designs are thinner and thus should withstand less traffic. This is shown by the Gravel Equivalents of the pavements, as reported in Chapter 5.

As the TI increases, the difference in Structural Number between the Caltrans "lowest cost" and Caltrans "thinnest AC" designs also increases. An increase in TI also causes an increase in the Structural Number for the AASHTO pavements (Figures 4.1 and 4.2). For the AASHTO pavements there appears to be a large difference between the Structural Number for the subgrade R-values of 5 and 20. This suggests that the AASHTO design procedure may be more sensitive to very low subgrade strengths than the Caltrans method.

4.2.3 Consideration of Drainage in AASHTO Procedure

As already mentioned, the AASHTO design procedure allows consideration of drainage in the pavement design process. The effect that drainage will have on thickness designs can be substantial, as shown in Table 4.3 for a few cases. Thicknesses of the pavement layers were reduced when the drainage conditions were good, simulated here by assuming a drainage coefficient of 1.30, and increased when the conditions are poor, simulated here by a drainage coefficient of 0.60. As can be seen in Table 4.3, subbase thickness is primarily affected, while base thickness is affected to a lesser degree. It is apparent from Table 4.3 that the AASHTO method increases base and subbase thicknesses for poor drainage conditions and does not increase the thickness of the asphalt concrete. Thicknesses for the pavements with a subgrade R-value of 5 change more than do those for which the subgrade R-value is 20. For the case of poor

**Table 4.3 Effect of drainage coefficient on thickness, structural number, and gravel equivalent (AASHTO procedure)**

| | Traffic Index | Subgrade R-value | Drainage Factor | Thickness (millimeters) Asphalt Concrete | Aggregate Base | Aggregate Subbase | AASHTO Structural Number | Caltrans Gravel Equivalent |
|---|---|---|---|---|---|---|---|---|
| AASHTO | 9 | 5 | 0.6 | 117 | 182 | 657 | 5.78 | 3.54 |
| AASHTO | 9 | 5 | 1.0 | 117 | 152 | 339 | 4.24 | 2.39 |
| AASHTO | 9 | 5 | 1.3 | 117 | 152 | 216 | 3.71 | 1.98 |
| Caltrans Lowest Cost | 9 | 5 | - | 137 | 213 | 335 | 4.89 | 2.72 |
| Caltrans Thinnest AC | 9 | 5 | - | 137 | 213 | 335 | 4.89 | 2.72 |
| AASHTO | 9 | 20 | 0.6 | 117 | 182 | | 2.94 | 1.38 |
| AASHTO | 9 | 20 | 1.0 | 117 | 152 | | 2.78 | 1.28 |
| AASHTO | 9 | 20 | 1.3 | 117 | 152 | | 2.78 | 1.28 |
| Caltrans Lowest Cost | 9 | 20 | - | 183 | 152 | 168 | 4.47 | 2.28 |
| Caltrans Thinnest AC | 9 | 20 | - | 137 | 213 | 213 | 4.36 | 2.32 |
| AASHTO | 13 | 5 | 0.6 | 210 | 255 | 899 | 8.58 | 5.05 |
| AASHTO | 13 | 5 | 1.0 | 210 | 153 | 539 | 6.54 | 3.51 |
| AASHTO | 13 | 5 | 1.3 | 210 | 152 | 377 | 5.83 | 2.97 |
| Caltrans Lowest Cost | 13 | 5 | - | 274 | 168 | 503 | 7.50 | 3.94 |
| Caltrans Thinnest AC | 13 | 5 | - | 198 | 335 | 503 | 7.04 | 3.95 |
| AASHTO | 13 | 20 | 0.6 | 210 | 255 | 123 | 5.22 | 2.51 |
| AASHTO | 13 | 20 | 1.0 | 210 | 221 | | 4.52 | 1.98 |
| AASHTO | 13 | 20 | 1.3 | 217 | 152 | | 4.30 | 1.78 |
| Caltrans Lowest Cost | 13 | 20 | - | 290 | 152 | 290 | 6.77 | 3.32 |
| Caltrans Thinnest AC | 13 | 20 | - | 198 | 335 | 305 | 6.18 | 3.30 |

* Ratio given relative to Drainage factor (m) = 1.0 case

drainage, TI of 13, and R-value of 5, the AASHTO design results in more than 1.15 m (3.77 ft.) of granular material.

For comparison, the pavement designs for the equivalent cases of the Caltrans "thinnest asphalt concrete" and Caltrans "lowest cost" pavements are also shown in Table 4.3. For the cases where the subgrade R-value is 5, the low drainage coefficient results in a pavement structure that is thicker than both the Caltrans "thinnest asphalt concrete" and Caltrans "lowest cost" pavements. This is not the case for the pavements with an R-value of 20. These observations suggest that the Caltrans design method may be conservative with respect to drainage and that if good drainage is present, the thickness design might be reduced, resulting in cost savings.

While the current Caltrans design method does not explicitly consider drainage, an alternative procedure is available to Caltrans in which such effects can be incorporated, i.e. with the mechanistic-empirical fatigue analysis and design method utilized in Chapter 5. (*15*) In this methodology, the effects of water content (or suction) on materials stiffness for use in pavement analyses can be determined by laboratory tests or non-destructive measurements on existing pavements. Such considerations may have a significant influence on pavement thickness, for example, as illustrated in the current AASHTO method (Table 4.3). Thus, to improve the overall performance of pavements in California and at the same time ensure that cost effective designs are obtained, a more systematic approach should be adopted to consider the effects of water (and drainage) on pavement response.

# 5.0 FATIGUE LIFE PREDICTIONS

## 5.1 UCB Design and Analysis Method

The fatigue analysis and design system used herein was originally developed as a part of the Strategic Highway Research Program as a performance based procedure for designing asphalt mixes to resist fatigue cracking. (*16*) Recently upgraded through the CAL/APT program, the system considers not only fundamental mix properties but also the level of design traffic, the temperature environment at the site, the pavement structural section, laboratory testing and construction variabilities, and an acceptable level of risk. (*2*)

For the purposes of this study, the fatigue analysis and design system is used primarily to estimate the number of ESALs that can be sustained for a selected design. The estimation is based on the following equation:

$$ESALs = \frac{N \bullet SF}{TCF \bullet M} \qquad \textbf{(5.1)}$$

in which $ESALs$ = the number of equivalent, 80-kN (9,000-lb.) single axle loads that can be sustained in situ before failure, $N$ = the number of laboratory load repetitions to failure under the anticipated in situ strain level, $SF$ = a shift factor necessary to reconcile the difference between fatigue in situ and that in the laboratory, $TCF$ = a temperature conversion factor which converts loading effects under the expected range of temperatures in the site-specific temperature environment to those under the single temperature typically used in laboratory testing for conventional asphalts (20°C), and $M$ = a reliability multiplier based on the level acceptable risk and variabilities associated with computing $N$ and with estimating actual traffic loading. Figure 5.1 contains a flow diagram of ESALs determination embodied in this methodology.

The estimate of N is based on laboratory testing, which measures the stiffness and fatigue life of the asphalt mix, and on elastic multilayer analysis, which determines the initial critical strain ($\varepsilon_t$) expected at the underside of the asphalt layer in situ under the standard 80-kN axle load. It is simply fatigue life that would result from the repetitive application of the strain expected in situ. The computer code, CIRCLY (*17*), was used for the elastic multilayer analyses reported herein.

As mentioned previously, two asphalt concrete mixes were included in this study: one containing a Coastal AR-4000 asphalt and the other a Valley AR-4000 asphalt. Both mixes use the same aggregate and the same aggregate gradation. The relationships of fatigue versus tensile strain are shown below for the Valley and Coastal mixes, respectively.

$$\ln N = -22.001 + 0.57520\,AC - 0.16457\,AV - 3.7176\ln\varepsilon_t \qquad \textit{(Valley asphalt)} \qquad \textbf{(5.2a)}$$

$$\ln N = -24.362 + 0.83988\,AC - 0.19193\,AV - 4.3606\ln\varepsilon_t \qquad \textit{(Coastal asphalt)} \qquad \textbf{(5.2b)}$$

where ln is the natural logarithm, $N$ = laboratory fatigue life, $AC$ and $AV$ = asphalt and air-void contents (percent), respectively, and $\varepsilon_t$ = tensile strain at the bottom of the asphalt concrete layer. Air-void and asphalt contents of 8 and 5 percent were assumed for both mixes.

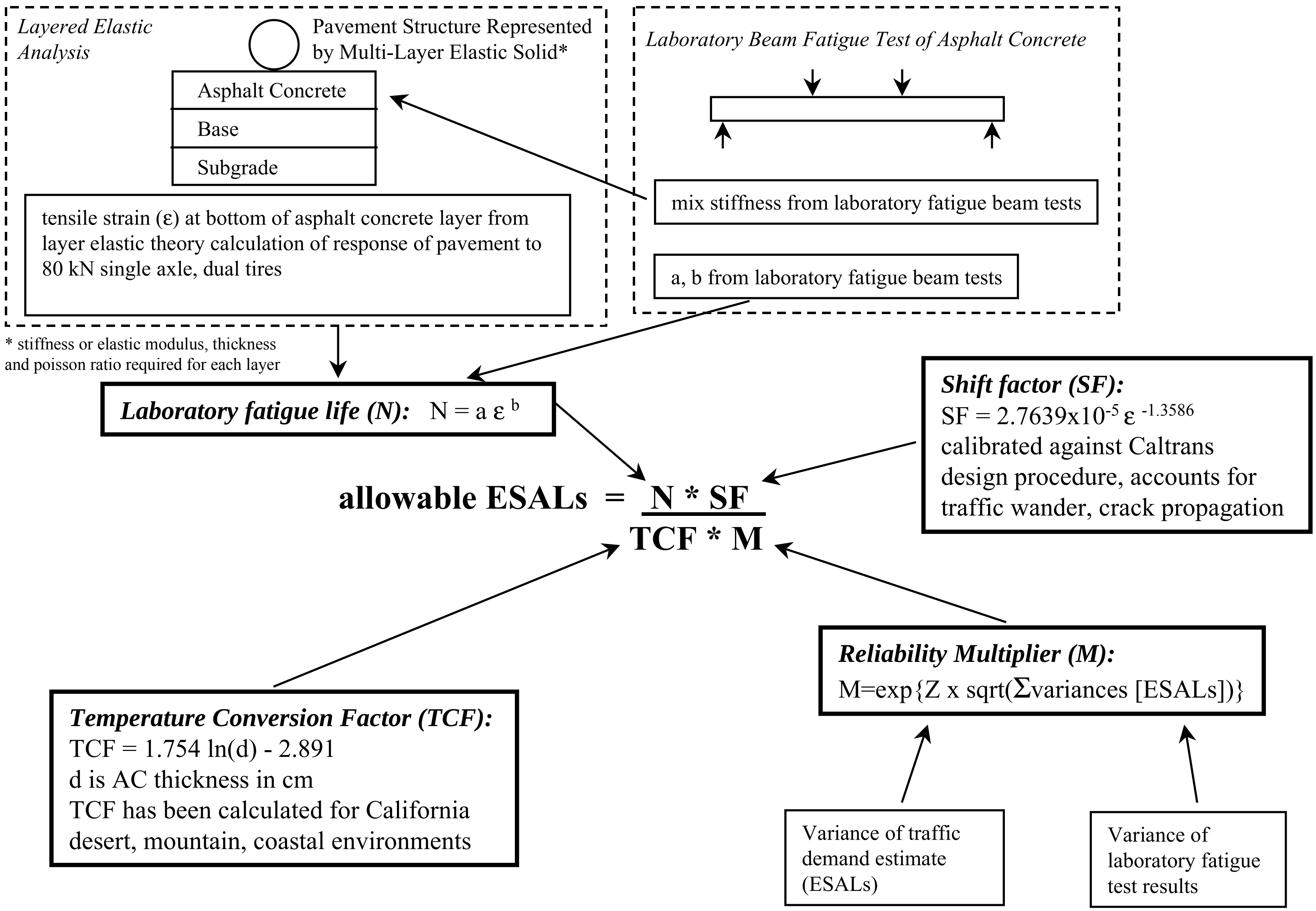


**Figure 5.1 Summary of elements of UC Berkeley fatigue analysis and design procedure**

The critical strain is also used in determining the shift factor1, SF, as follows:

$$SF = 2.7639 \bullet 10^{-5} \varepsilon_t^{-1.3586} \quad \textbf{(5.3)}$$

The shift factor accounts for differences between laboratory and field conditions including but not limited to traffic wander, crack propagation, and rest periods. The shift factor relation is based on calibration of laboratory results against the Caltrans pavement thickness design procedure. This shift factor was calibrated against pavements designed according to the Caltrans procedure using the lowest initial cost, based on default costs, and only the mix containing the Valley asphalt mix. (*2*) The shift factor may, therefore, be on the somewhat conservative side.

The temperature conversion factor, TCF, has been previously determined for three representative California environments including those in coastal, desert, and mountain regions, an asphalt-aggregate mix containing a crushed gravel, and an AR-4000 asphalt refined from California Coastal sources (the same asphalt included in this study). For this study, in-situ performance simulations were based on the TCF for the coastal environment using the following expression:

$$TCF = 1.754\ln(d) - 2.891 \quad \textbf{(5.4)}$$

in which $d$ = the asphalt concrete thickness in centimeters.

The reliability multiplier, M, is calculated as follows:

$$M = e^{Z\sqrt{\mathrm{var}(\ln N) + \mathrm{var}(\ln ESALs)}} \quad \textbf{(5.5)}$$

---

[1] The shift factor and reliability multiplier used for this report is taken from (*2*). Later analyses that include the effects of construction variance and use Monte Carlo simulations to determine the reliability multiplier (*M*) use the following shift factor (*11, 14*): SF = $3.1833 \cdot 10^{-5}\ \varepsilon^{-1.3759}$

in which $e$ = the base of natural logarithms, $Z$ = a factor for the quantile of the standard normal distribution depending solely on the design reliability, $var(\ln N)$ = the variance of the logarithm of the laboratory fatigue life estimated at the in-situ strain level, and $var(\ln ESALs)$ = the variance of the estimate of the logarithm of the design ESALs (i.e., the variance associated with uncertainty in the traffic estimate). For this study, a 90-percent reliability level was selected for which the $Z$ value is 1.28. The $var(\ln N)$ was determined using the equation shown above and a Monte Carlo simulation procedure. This procedure accounts for the inherent variability in fatigue measurements, the nature of the laboratory testing program (principally the number of test specimens and the strain levels), and the extent of extrapolation necessary for estimating fatigue life at the design in-situ strain level. An assumed variance of the estimate of the design traffic $var(\ln ESALs)$ of 0.3 has also been used in these calculations.

## 5.2 Fatigue Life Predictions

Fatigue life predictions presented in this section include a number of assumptions, the reasonableness of which has been demonstrated in part by the Goal 1 CAL/APT test program. Accordingly, they permit a uniform and quantitative means to compare the thickness designs from the Caltrans and AASHTO methods.

The critical tensile strain at the bottom of the asphalt concrete layer for the pavement thicknesses given in Table 4.2 under a 40-kN dual wheel load with center to center distance between the wheels of 304 mm (12 inches), and a tire pressure of 690 kPa (100 psi) load was calculated using elastic layer theory and the program CIRCLY. (*17*) This load is the "equivalent single axle load" or "ESAL" to which all traffic loads are converted. Using the laboratory fatigue life prediction equations, discussed in Section 5.1, and the assumed temperature conversion factor, reliability factor, and shift factor, the predicted field fatigue life is determined. The values used for the temperature conversion factor, the shift factor, and the reliability factor are shown in Table 5.1. As already discussed, the temperature conversion factor is only dependent on the thickness of the asphalt concrete layer. The shift factor, on the other hand, is dependent on the tensile strain, which is a function of both the stiffness and thickness of the asphalt concrete layer. The reliability factor is assigned and is therefore neither a function of thickness nor stiffness. The results are shown in Table 5.2.

Figures 5.2 to 5.6 show plots of the fatigue life predictions versus subgrade R-value for each Traffic Index. Figures 5.7 and 5.8 show the same data separated for each asphalt type on a logarithmic plot. It can be seen in these figures that the pavements designed according to the Caltrans "lowest cost" method typically give the longest fatigue life predictions for all the cases. This result contrasts with the Gravel Equivalent results, which indicate that the Caltrans "thinnest asphalt concrete" pavements have about the same Gravel Equivalents and therefore should withstand traffic equally. The Caltrans "lowest cost" pavements have thicker asphalt concrete layers than do the Caltrans "thinnest asphalt concrete" pavements, resulting in longer predicted fatigue lives for the "lowest cost" designs. In the Caltrans "thinnest asphalt concrete" pavements, the thinner asphalt concrete layer is compensated for by thicker base and subbase layers. The longer predicted fatigue lives for the pavements with thicker asphalt concrete layers suggest that

**Table 5.1 Temperature conversion factors and shift factors**

| Traffic Index | Subgrade R-Value | Caltrans Lowest Cost | Caltrans Thinnest AC | AASHTO | Caltrans Lowest Cost | Caltrans Thinnest AC | AASHTO | Caltrans Lowest Cost | Caltrans Thinnest AC | AASHTO | All Design Procedures |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Temperature Conversion Factor | | | Shift Factor - Valley Asphalt Mix | | | Shift Factor - Coastal Asphalt Mix | | | Reliability |
| 7 | 5 | 1.495 | 0.991 | 0.757 | 3.511 | 2.552 | 2.086 | 1.433 | 1.139 | 0.996 | 2.517 |
| 7 | 20 | 1.261 | 0.991 | 0.757 | 3.244 | 2.695 | 2.193 | 1.293 | 1.138 | 0.977 | 2.517 |
| 7 | 40 | 1.261 | 0.991 | 0.757 | 3.643 | 2.957 | 2.543 | 1.392 | 1.191 | 1.061 | 2.517 |
| 9 | 5 | 1.702 | 1.702 | 1.425 | 4.649 | 4.649 | 3.486 | 1.827 | 1.827 | 1.422 | 2.517 |
| 9 | 20 | 2.207 | 1.702 | 1.425 | 7.544 | 4.959 | 3.594 | 2.625 | 1.853 | 1.386 | 2.517 |
| 9 | 40 | 2.207 | 1.702 | 1.425 | 8.519 | 5.443 | 4.143 | 2.899 | 1.959 | 1.542 | 2.517 |
| 11 | 5 | 2.598 | 2.054 | 1.919 | 10.096 | 6.635 | 5.449 | 3.334 | 2.518 | 2.103 | 2.517 |
| 11 | 20 | 2.817 | 2.054 | 1.919 | 14.281 | 6.979 | 5.742 | 4.372 | 2.540 | 2.127 | 2.517 |
| 11 | 40 | 2.711 | 2.054 | 1.919 | 14.259 | 7.566 | 6.466 | 4.364 | 2.665 | 2.315 | 2.517 |
| 13 | 5 | 2.918 | 2.347 | 2.452 | 14.804 | 8.547 | 8.857 | 4.726 | 2.933 | 2.973 | 2.517 |
| 13 | 20 | 3.013 | 2.347 | 2.452 | 18.309 | 8.953 | 9.464 | 5.514 | 2.961 | 3.052 | 2.517 |
| 13 | 40 | 3.103 | 2.347 | 2.452 | 22.947 | 9.587 | 10.604 | 6.786 | 3.078 | 3.344 | 2.517 |
| 15 | 5 | 3.270 | 2.598 | 2.788 | 22.869 | 11.393 | 13.060 | 7.060 | 3.772 | 4.217 | 2.517 |
| 15 | 20 | 3.348 | 2.598 | 2.788 | 28.079 | 11.895 | 14.005 | 8.241 | 3.806 | 4.344 | 2.517 |
| 15 | 40 | 3.422 | 2.598 | 2.788 | 34.789 | 12.675 | 15.627 | 10.089 | 3.945 | 4.748 | 2.517 |

**Table 5.2 Predicted pavement fatigue lives for Caltrans "lowest cost," Caltrans "thinnest asphalt concrete" and AASHTO designs, assuming 90-percent reliability**

| Traffic Index | TI Caltrans ESALs | Subgrade R-Value | Caltrans Lowest Cost | Caltrans Thinnest AC | AASHTO | Caltrans Lowest Cost | Caltrans Thinnest AC | AASHTO |
|---|---|---|---|---|---|---|---|---|
| | | | Valley Asphalt Mix | | | Coastal Asphalt Mix | | |
| 7 | 120,000 | 5 | 116,000 | 53,000 | 33,000 | 195,000 | 112,000 | 83,000 |
| 7 | 120,000 | 20 | 102,000 | 65,000 | 39,000 | 150,000 | 112,000 | 77,000 |
| 7 | 120,000 | 40 | 158,000 | 92,000 | 68,000 | 205,000 | 135,000 | 109,000 |
| 9 | 1,000,000 | 5 | 291,000 | 291,000 | 118,000 | 476,000 | 476,000 | 198,000 |
| 9 | 1,000,000 | 20 | 1,375,000 | 370,000 | 133,000 | 1,687,000 | 505,000 | 178,000 |
| 9 | 1,000,000 | 40 | 2,169,000 | 525,000 | 226,000 | 2,564,000 | 639,000 | 279,000 |
| 11 | 5,400,000 | 5 | 3,480,000 | 914,000 | 468,000 | 3,920,000 | 1,523,000 | 764,000 |
| 11 | 5,400,000 | 20 | 11,761,000 | 1,104,000 | 569,000 | 11,315,000 | 1,579,000 | 801,000 |
| 11 | 5,400,000 | 40 | 12,153,000 | 1,494,000 | 888,000 | 11,669,000 | 1,932,000 | 1,144,000 |
| 13 | 22,000,000 | 5 | 12,996,000 | 2,065,000 | 2,258,000 | 15,164,000 | 2,530,000 | 2,565,000 |
| 13 | 22,000,000 | 20 | 27,898,000 | 2,456,000 | 2,894,000 | 28,113,000 | 2,634,000 | 2,866,000 |
| 13 | 22,000,000 | 40 | 63,107,000 | 3,174,000 | 4,433,000 | 65,422,000 | 3,102,000 | 4,207,000 |
| 15 | 73,160,000 | 5 | 59,124,000 | 5,472,000 | 8,504,000 | 73,350,000 | 6,592,000 | 9,827,000 |
| 15 | 73,160,000 | 20 | 124,553,000 | 6,431,000 | 11,049,000 | 137,333,000 | 6,847,000 | 11,132,000 |
| 15 | 73,160,000 | 40 | 271,848,000 | 8,160,000 | 16,656,000 | 314,871,000 | 7,964,000 | 16,196,000 |

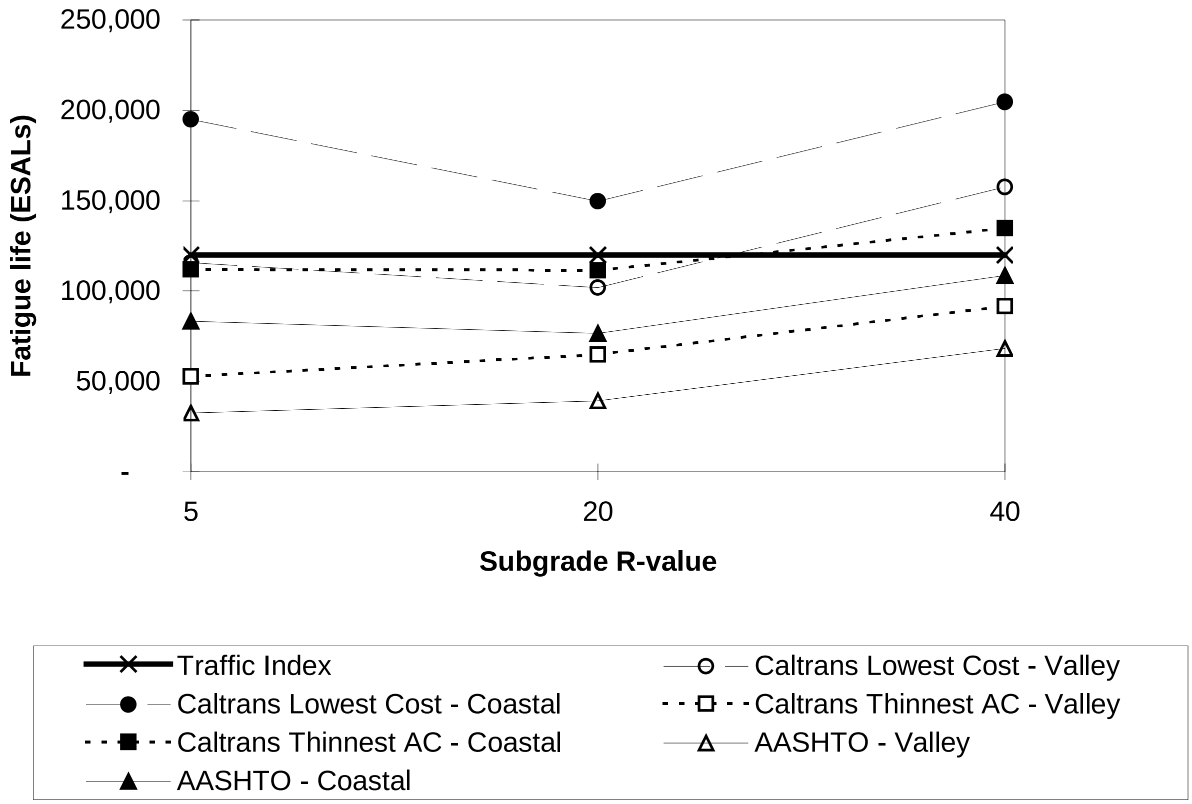


**Figure 5.2 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, TI = 7 (linear plot)**

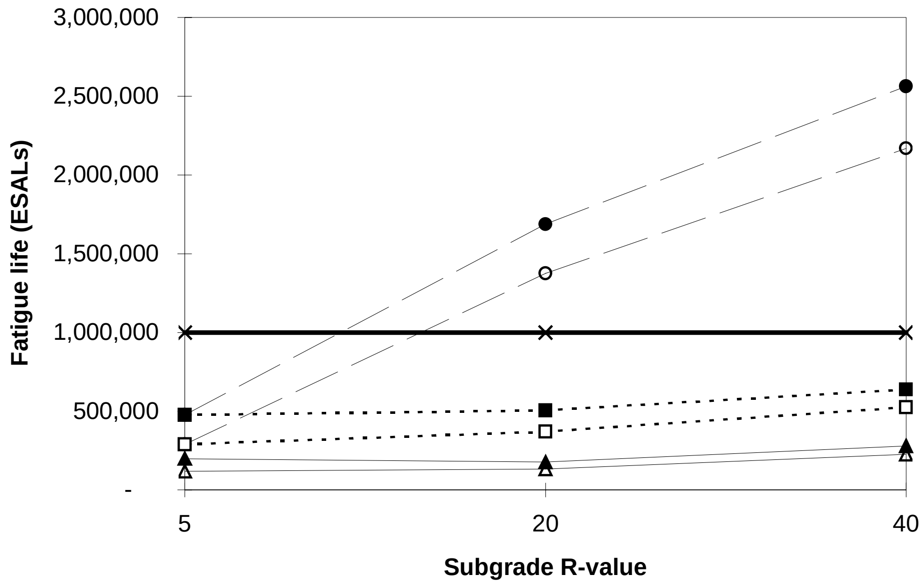


**Figure 5.3 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, TI = 9 (linear plot)**

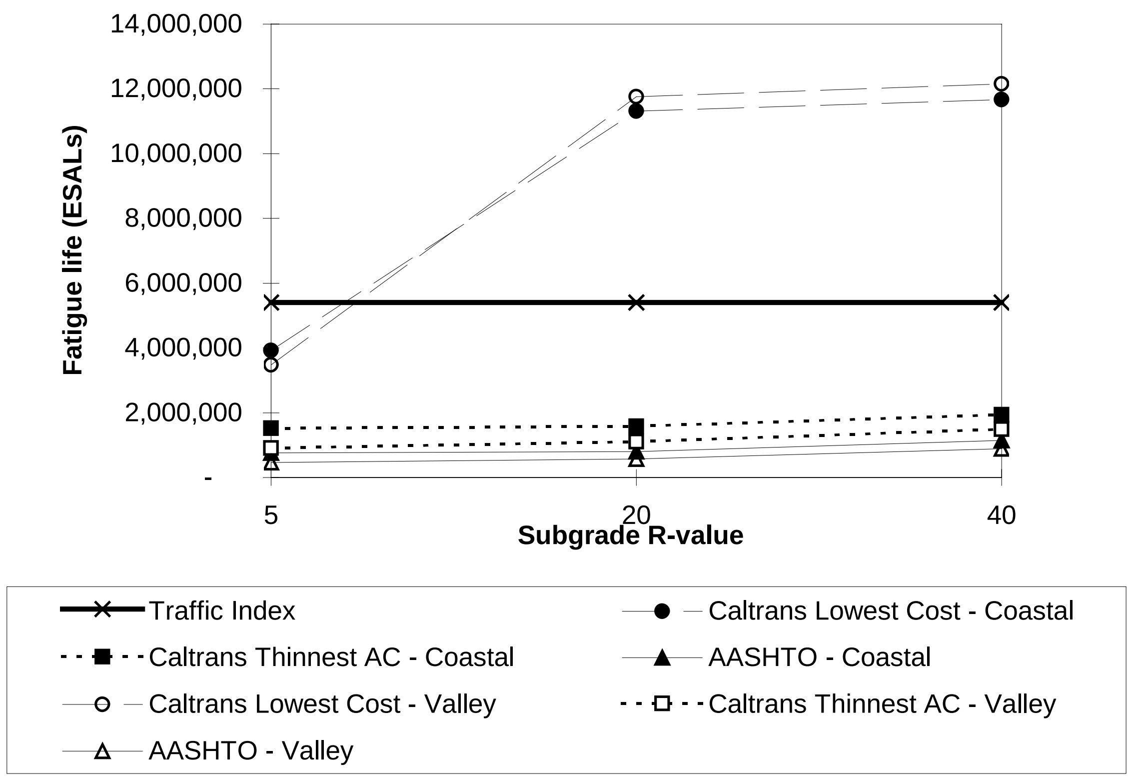


**Figure 5.4 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, TI = 11 (linear plot)**

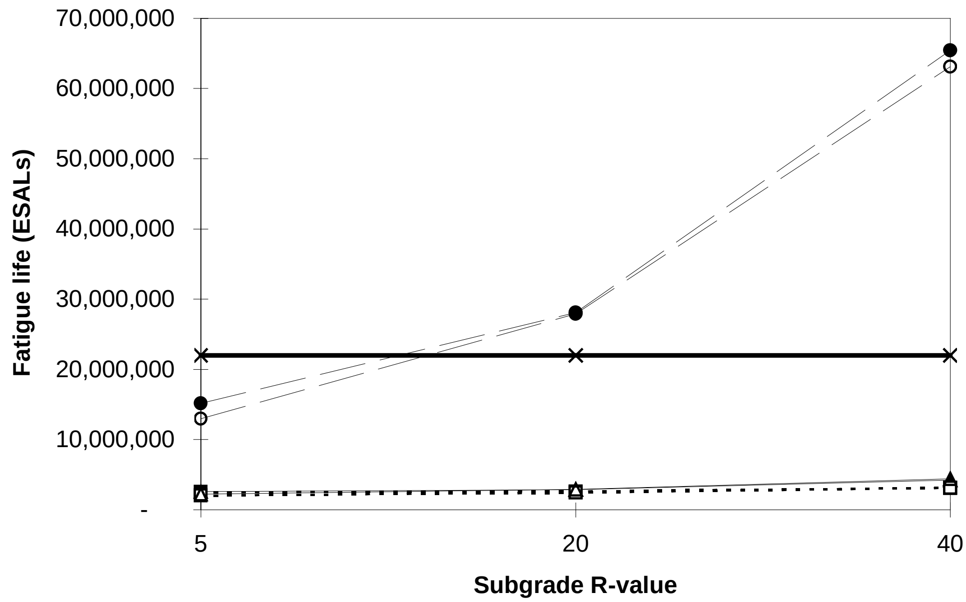


**Figure 5.5 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, TI = 13 (linear plot)**

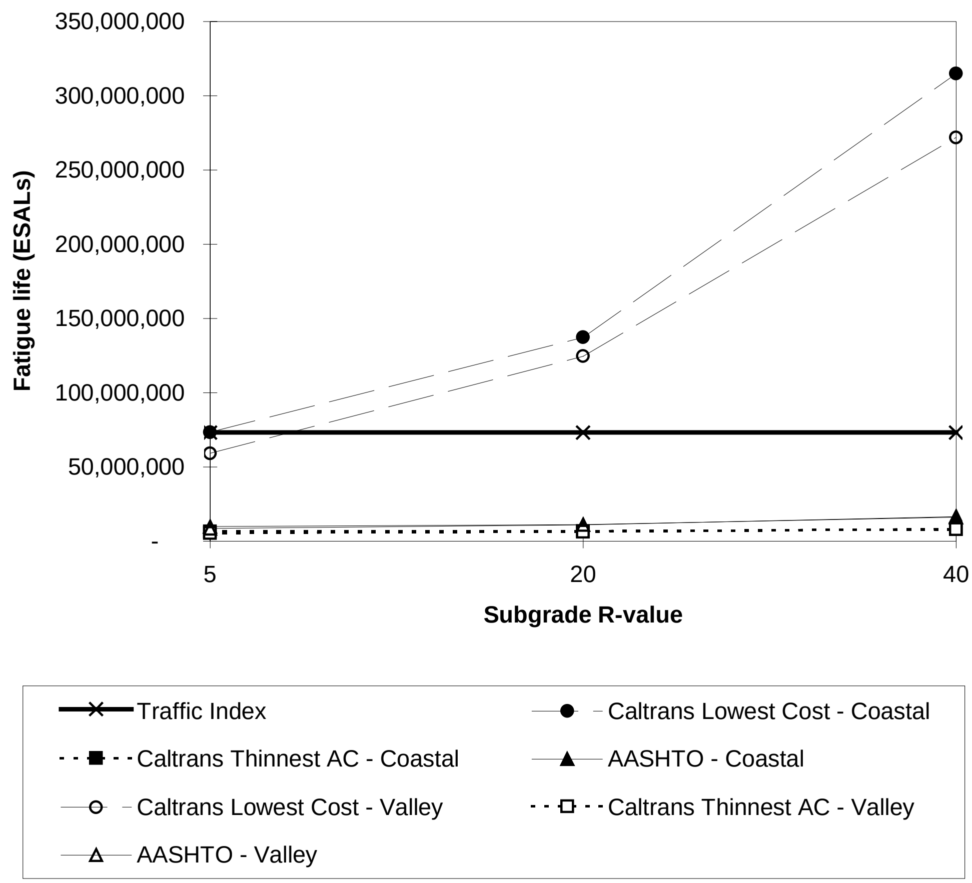


**Figure 5.6 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, TI = 15 (linear plot)**

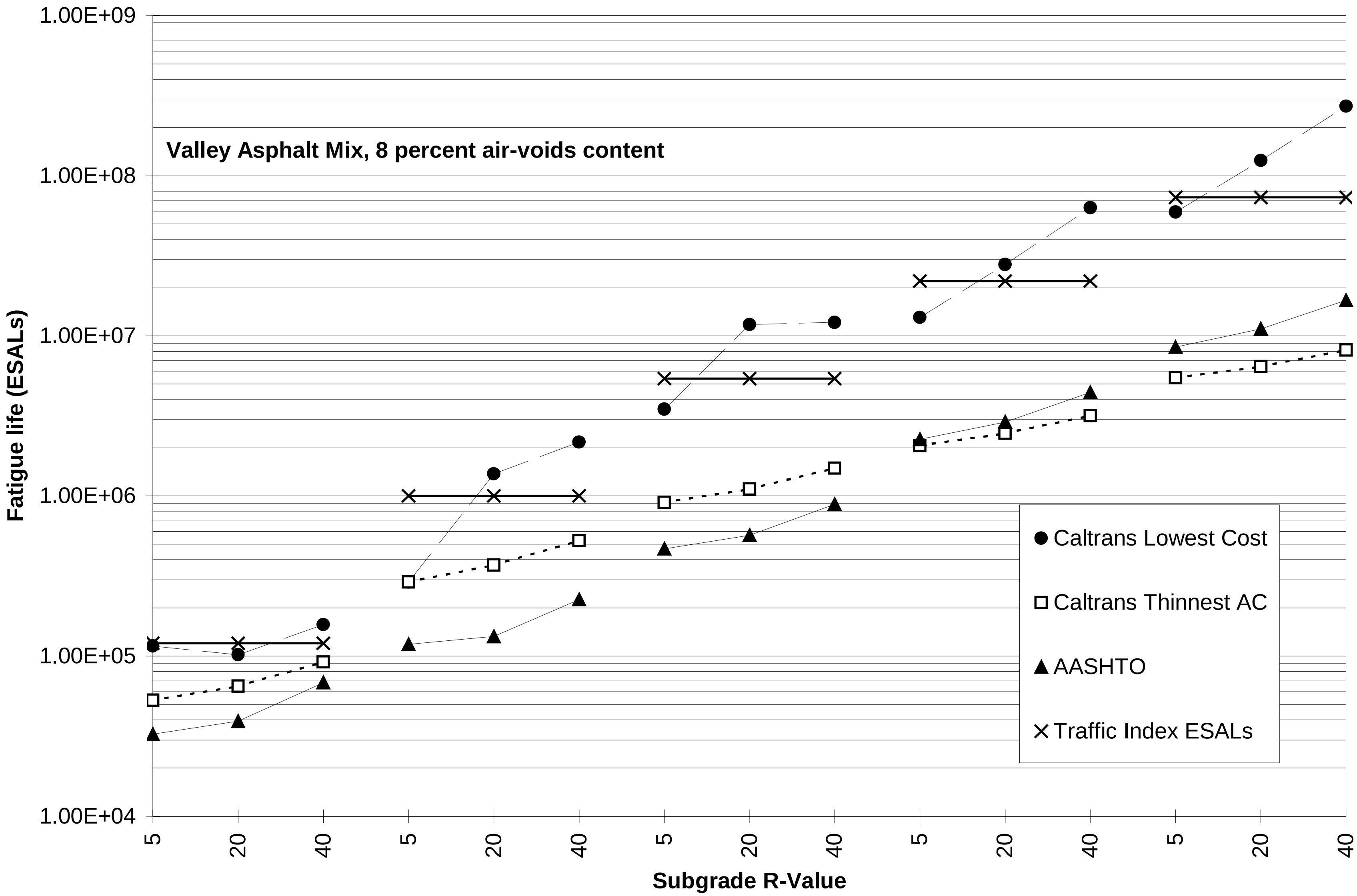


**Figure 5.7 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, Valley asphalt mixes (logarithmic plot)**

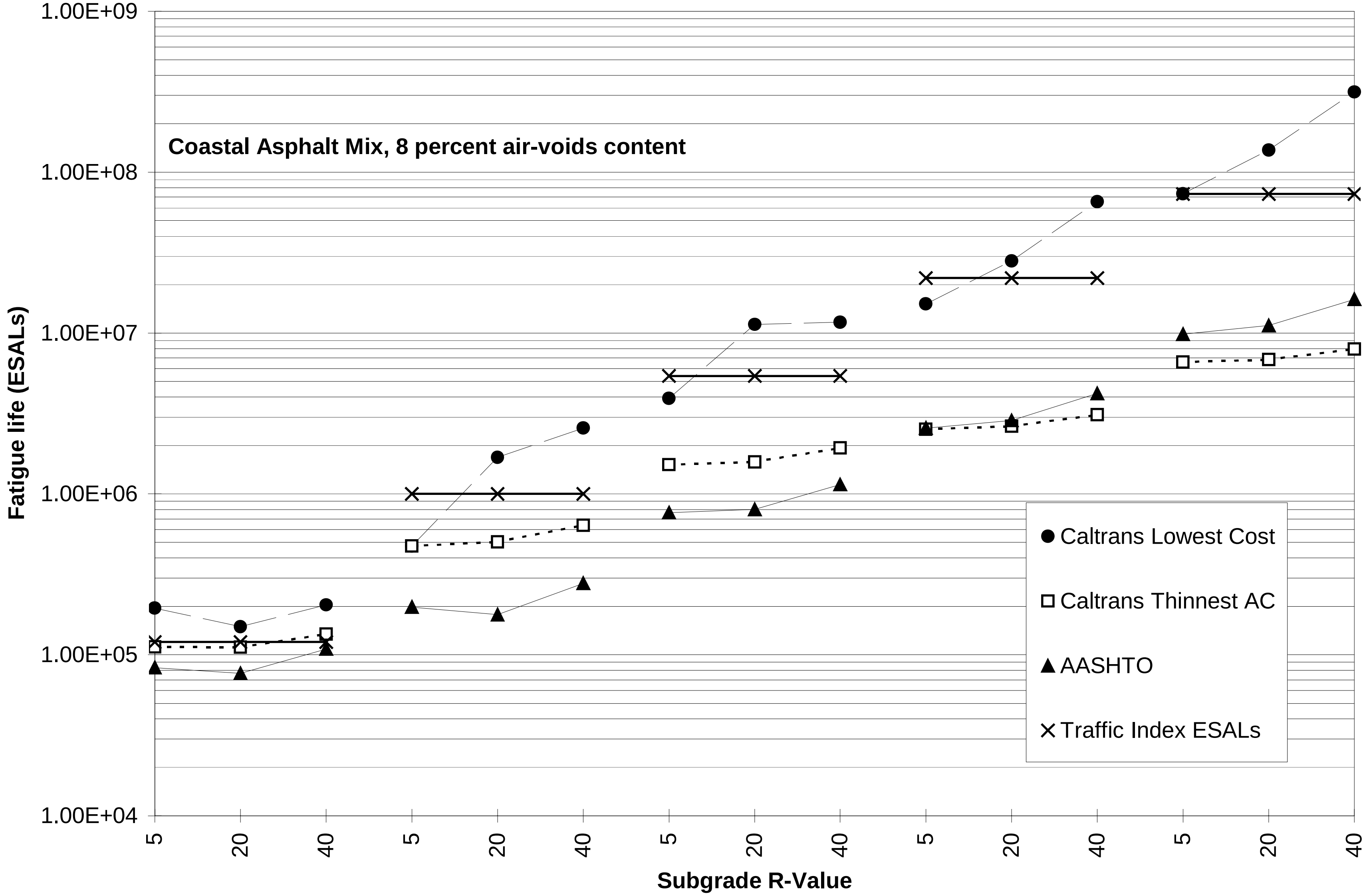


**Figure 5.8 Comparison of predicted fatigue performance for pavements designed by Caltrans and AASHTO procedure, Coastal asphalt mixes (logarithmic plot)**

the Gravel Factors for asphalt concrete should be larger relative to those of the aggregate base and subbase than currently in use. An increase in the asphalt concrete Gravel Factors relative to those of the aggregate base would be more consistent with the structural coefficients of these materials in the AASHTO method as well.

The ratios of predicted fatigue life for same subgrade R-value and TI and the three sets of pavement designs are shown in Table 5.3. It can be seen that for both asphalts, as the required pavement thicknesses increase, the fatigue lives of the Caltrans "lowest cost" pavements become much larger than those of the Caltrans "thinnest asphalt concrete" pavements. For thin pavements designed for small TIs (7 to 9) in which the asphalt concrete is relatively thin and does not play such a large role in determining fatigue life, the ratio of the fatigue lives of the "lowest cost" pavements to those of the "thinnest asphalt concrete" pavements is on the order of 1:1 to 4:1. For TIs of 11 to 15, the Caltrans "lowest cost" pavements have predicted fatigue lives that are 3 to 40 times greater than those of the Caltrans "thinnest asphalt concrete" pavements. In these thicker structures, the asphalt concrete plays a greater role in determining fatigue life. This effect is included to some degree in the Caltrans design method by the increase in Gravel Factor for asphalt concrete from 1.46 to about 2.5 with increase asphalt concrete layer thickness. However, increasing asphalt concrete Gravel Factors without adjusting the design equations would result in thinner asphalt concrete layers, which would be counter productive.

Predicted fatigue lives for the pavements designed by the AASHTO procedure are always less than those of the Caltrans "lowest cost" pavements. The difference is about 3:1 for pavements with a TI = 7 (Table 5.3). For other TIs and subgrade R-values of 20 and 40, the

Table 5.3 Ratios of predicted pavement fatigue lives for Coastal and Valley asphalt mixes.

| Traffic Index | TI Caltrans ESALs | Subgrade R-Value | Valley Asphalt Mix | | | Coastal Asphalt Mix | | |
|---|---|---|---|---|---|---|---|---|
| | | | CT lowest cost/ CT thinnest AC | CT lowest cost/ AASHTO | CT thinnest AC/ AASHTO | CT lowest cost/ CT thinnest AC | CT lowest cost/ AASHTO | CT thinnest AC/ AASHTO |
| 7 | 120,000 | 5 | 2.2 | 3.5 | 1.6 | 1.7 | 2.3 | 1.3 |
| 7 | 120,000 | 20 | 1.6 | 2.6 | 1.7 | 1.3 | 1.9 | 1.5 |
| 7 | 120,000 | 40 | 1.7 | 2.3 | 1.4 | 1.5 | 1.9 | 1.2 |
| 9 | 1,000,000 | 5 | 1.0 | 2.5 | 2.5 | 1.0 | 2.4 | 2.4 |
| 9 | 1,000,000 | 20 | 3.7 | 10.3 | 2.8 | 3.3 | 9.5 | 2.8 |
| 9 | 1,000,000 | 40 | 4.1 | 9.6 | 2.3 | 4.0 | 9.2 | 2.3 |
| 11 | 5,400,000 | 5 | 3.8 | 7.4 | 2.0 | 2.6 | 5.1 | 2.0 |
| 11 | 5,400,000 | 20 | 10.7 | 20.7 | 1.9 | 7.2 | 14.1 | 2.0 |
| 11 | 5,400,000 | 40 | 8.1 | 13.7 | 1.7 | 6.0 | 10.2 | 1.7 |
| 13 | 22,000,000 | 5 | 6.3 | 5.8 | 0.9 | 6.0 | 5.9 | 1.0 |
| 13 | 22,000,000 | 20 | 11.4 | 9.6 | 0.8 | 10.7 | 9.8 | 0.9 |
| 13 | 22,000,000 | 40 | 19.9 | 14.2 | 0.7 | 21.1 | 15.6 | 0.7 |
| 15 | 73,160,000 | 5 | 10.8 | 7.0 | 0.6 | 11.1 | 7.5 | 0.7 |
| 15 | 73,160,000 | 20 | 19.4 | 11.3 | 0.6 | 20.1 | 12.3 | 0.6 |
| 15 | 73,160,000 | 40 | 33.3 | 16.3 | 0.5 | 39.5 | 19.4 | 0.5 |

Caltrans "lowest cost" pavements have fatigue lives about 10 to 20 times greater than those of the AASHTO pavements. Lives of the Caltrans "lowest cost" pavements with a subgrade R-value of 5 are only about 2 to 8 times longer than the AASHTO designs, indicating that these designs are not as conservative when the two designs are compared for weaker subgrades.

Comparison of the Caltrans "thinnest asphalt concrete" pavements with the AASHTO pavements indicates that the "thinnest asphalt concrete" pavements have fatigue lives 1.2 to 3 times longer than the AASHTO pavements for TIs of 7, 9, and 11. This difference is due to thicker asphalt concrete and base layers in the Caltrans "thinnest asphalt concrete" pavements than in the AASHTO pavements for these conditions. For TIs of 13 and 15, the Caltrans "thinnest asphalt concrete" pavements have asphalt concrete layers 12 to 26 mm thinner and base and subbase layers that are up to 2.5 times thicker than those of the AASHTO pavements (Table 4.2). For these designs, the predicted fatigue lives for the Caltrans "thinnest asphalt concrete" pavements are at the most equal to and maybe as little as half those of the AASHTO pavements (Table 5.3).

These results also imply that control of asphalt concrete thickness during construction is important. This implication supports the study reported in Reference (*5*) for pay factor associated with deviations in the thickness during construction.

For the comparisons shown in Table 5.3, the AASHTO pavements were designed based on fair to good drainage conditions, which assumes the pavement structure is to be exposed to water contents approaching saturation about 5 to 25 percent of the time. The Caltrans design method utilizes results of the R-value test, in which the soil is tested in a near-saturated condition.[2] By considering various drainage conditions in the AASHTO procedure, it was shown that the AASHTO procedure produces structures with thicker base and subbase layers when poor drainage conditions are anticipated (Chapter 4). Assumption of poor drainage conditions in the AASHTO procedures, and the resultant thicker structures, would result in less difference in predicted pavement fatigue life between the Caltrans "thinnest asphalt concrete" and AASHTO structures. Such results suggest that explicit consideration of drainage in the Caltrans procedure may lead to considerably different pavement structures depending upon site-specific conditions for expected rainfall, soil type, and potential drainage features included in the pavement.

Analytically-based methodology such as that utilized herein permits a direct consideration of drainage effects on rehabilitation design if material properties are back-calculated from Falling Weight Deflectometer (FWD) data collected at the wettest and driest times of the year.

Comparison of the predicted fatigue lives for the same pavement structures containing the Valley and Coastal asphalts indicates that the mix with the Coastal asphalt provides better performance for nearly every structure included in this study (Table 5.2, Figures 5.2 to 5.6). Fatigue lives of these pavements are from 1.0 to 2.5 times greater than those for pavements containing the mix with the Valley asphalt (Table 5.4). Calculations indicate that the Coastal asphalt mix produces longer service lives as compared to the Valley asphalt mix for pavements with Traffic Indices of 7, 9, and 11 and pavements with thinner asphalt concrete layers, which include the Caltrans "thinnest asphalt concrete" and AASHTO designs. The Coastal asphalt mix

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[2] In the early work of Hveem, it was suggested that only about one third of the pavement subgrades become saturated. However, it was difficult to predict those which would develop these conditions. Hence, all are tested this way to be on the conservative side. (*16*)

**Table 5.4 Ratios of predicted pavement fatigue lives for Caltrans "lowest cost," Caltrans "thinnest asphalt concrete" and AASHTO designs**

| | | | Predicted Fatigue Life (ESALs) of Coastal Asphalt Mix Relative to Valley Asphalt Mix for Each Structure | | |
|---|---|---|---|---|---|
| Traffic Index | TI Caltrans ESALs | Subgrade R-Value | Caltrans lowest cost | Caltrans thinnest AC | AASHTO |
| 7 | 120,000 | 5 | 1.7 | 2.1 | 2.5 |
| 7 | 120,000 | 20 | 1.5 | 1.7 | 2.0 |
| 7 | 120,000 | 40 | 1.3 | 1.5 | 1.6 |
| 9 | 1,000,000 | 5 | 1.6 | 1.6 | 1.7 |
| 9 | 1,000,000 | 20 | 1.2 | 1.4 | 1.3 |
| 9 | 1,000,000 | 40 | 1.2 | 1.2 | 1.2 |
| 11 | 5,400,000 | 5 | 1.1 | 1.7 | 1.6 |
| 11 | 5,400,000 | 20 | 1.0 | 1.4 | 1.4 |
| 11 | 5,400,000 | 40 | 1.0 | 1.3 | 1.3 |
| 13 | 22,000,000 | 5 | 1.2 | 1.2 | 1.1 |
| 13 | 22,000,000 | 20 | 1.0 | 1.1 | 1.0 |
| 13 | 22,000,000 | 40 | 1.0 | 1.0 | 0.9 |
| 15 | 73,160,000 | 5 | 1.2 | 1.2 | 1.2 |
| 15 | 73,160,000 | 20 | 1.1 | 1.1 | 1.0 |
| 15 | 73,160,000 | 40 | 1.2 | 1.0 | 1.0 |

also performs well relative to the Valley mix when the subgrade R-value is 5, as compared to pavements with subgrade R-values of 20 and 40.

The Coastal asphalt mix has a lower stiffness than the Valley asphalt mix; lower mix stiffnesses result in larger tensile strains for a given structure and load. The Coastal asphalt mix also exhibits a larger number of load applications to failure for a given tensile strain than does the Valley asphalt mix. It is therefore reasonable to expect that the Coastal asphalt mix will perform particularly well relative to the Valley asphalt mix when the larger strains are expected, e.g., for poor support conditions and for comparatively thin layers of asphalt concrete.

These results comparing the performance of the two mixes, in conjunction with other results produced by the CAL/APT program illustrate the usefulness of the mechanistic-empirical approach to integrate materials response characteristics and pavement design. (*2, 5, 13, 15, 18*)

The results also demonstrate that in the Caltrans and AASHTO design procedures, the assumption that all asphalt concrete mixes have similar performance characteristics is likely not valid. The results also demonstrate that both the pavement structure and associated layer material characteristics determine fatigue performance and that designs associated with specified reliability levels can be produced by the mechanistic analysis and design procedure described herein (e.g., the 90 percent level illustrated in Figures 5.2 through 5.8 and Table 5.5).

It must be kept in mind that the shift factor included in the UCB procedure was calibrated using the Caltrans "lowest cost" pavement design and the Valley asphalt mix with 8-percent air voids. However, if that shift factor is assumed to be correct, the results would indicate that the Caltrans "thinnest asphalt concrete" and the AASHTO pavement structures are generally not adequate. In particular, the Caltrans "thinnest asphalt concrete" pavement structures for high traffic levels (i.e., Traffic Indexes of 13 and 15) can only withstand about 10 to 40 percent of the design ESALs with 90 percent reliability.

It is interesting to note that although the shift factor used herein was calibrated using the Valley asphalt mix, the ratios of predicted fatigue life to design fatigue life appear to be similar between the Valley and Coastal asphalt mixes. This similarity suggests that use of the Valley asphalt mix for calibration does not produce unreasonable results for the other mix.

# 6.0 CONCLUSIONS AND RECOMMENDATIONS

## 6.1 Summary

This report has been prepared to compare the Caltrans and AASHTO pavement thickness design procedures. The design comparisons included pavement structures subjected to a range in traffic, as represented by Traffic Indexes of 7, 9, 11, 13, and 15, and a range in subgrade strengths, as measured by subgrade R-values of 5, 20, and 40. Fatigue performance of the pavement structures designed by the two procedures were predicted using the mechanistic-empirical procedure developed as a part of the CAL/APT program. The various comparisons presented provide the basis for recommendations for consideration by Caltrans relative to pavement thickness design.

## 6.2 Conclusions

The following conclusions can be drawn from results presented in this report:

- The AASHTO and Caltrans pavement thickness design procedures do not produce the same pavement structures for a given set of design inputs of subgrade strength, design traffic, and pavement materials. Accordingly, these thickness design procedures should not be used interchangeably. This conclusion is based on results that include assumptions regarding comparable subgrade and pavement materials properties for a given set of materials within the laboratory testing systems used for each design procedure: R-value for the Caltrans procedure and resilient modulus ($M_R$) for the AASHTO procedure. Pavement structures produced by the two design procedures are sensitive to the conversion from one type of laboratory test to the other, as was demonstrated for the subbase layer in this report.

- Generally, pavement structures designed by the Caltrans procedure are thicker than those designed by the AASHTO procedure.
- The relative contribution of asphalt concrete, granular base, and subbase materials to a pavement structure's load carrying capability is different for the two design procedures. The ratio of the Gravel Factors of asphalt concrete to granular base is between 1.33 to 2.54 in the Caltrans design procedure, and about 3.2 in the AASHTO design procedure. Results presented in this report indicate that the structural contribution of the asphalt concrete to fatigue cracking resistance for thicker asphalt concrete layers is larger than is indicated by the Caltrans Gravel Factors.
- The predicted fatigue performance for the two Caltrans design options, "lowest cost" (the layer thickness combination with the lowest construction cost in NEWCON90) and "thinnest asphalt concrete layer allowed" is significantly different for most inputs of Traffic Index and subgrade R-value. The "lowest cost" pavements, which have thicker asphalt concrete layers, have predicted fatigue lives that are as much as 40 times longer at a Traffic Index of 15 than the pavements designed using the Caltrans "thinnest asphalt concrete layer" design. Differences in predicted fatigue lives between the Caltrans "lowest cost" pavements and the Caltrans "thinnest asphalt concrete" pavements diminish for smaller Traffic Indices. It is therefore likely that using the Caltrans procedure, alternative designs that are supposed to have similar performance characteristics as measured by gravel equivalent (GE) will exhibit

different fatigue performance, with pavements using thinner asphalt concrete layers not performing as well as pavements with thicker asphalt concrete layers.

- For Traffic Indices of 7, 9, and 11, the predicted fatigue lives of the Caltrans "thinnest allowable asphalt concrete layer" pavements are about 1.5 to 2.8 times greater than those of pavements designed using the AASHTO method with an assumed drainage coefficient of 1.0. For more heavily trafficked pavements under the same drainage condition, the AASHTO pavements typically have somewhat longer predicted fatigue lives, approaching a factor of 2 at a Traffic Index of 15. Pavements designed by the Caltrans methodology using the "lowest cost" option always have predicted fatigue lives greater than those of pavements designed using the AASHTO method.
- Predicted pavement fatigue life is extremely sensitive to asphalt concrete thickness, emphasizing an earlier recommendation that asphalt concrete thickness be included as a pay factor variable in the Caltrans QC/QA procedures for asphalt concrete construction. (*5*)
- Pavement drainage and lifecycle cost analyses are included explicitly in the AASHTO procedure and currently available design software (DNPS86). While they are required in the Caltrans design procedure, pavement drainage is not directly accounted for and lifecycle cost analysis is not included in the currently available design software. (*9*)
- While the thickness of asphalt concrete remains constant with changes in drainage conditions in the AASHTO procedure, thicknesses of base and subbase layer can change considerably depending upon expected drainage. The Caltrans procedure

assumes that pavement designs without special drainage priorities are adequate and that their inclusion make Caltrans pavement designs conservative. However, no quantitative analysis is performed to account for differences in the soils susceptibility to water or rainfall for a particular location.

- Predicted pavement fatigue life is sensitive to different asphalt concrete mixes. Results of fatigue estimates for pavements containing mixes using AR-4000 asphalt binders from California Valley and California Coastal sources indicated that the Coastal binder performed better in all of the structures analyzed. Differences in predicted performance of pavements containing the two mixes depended on subgrade support and the thickness of the asphalt concrete layer.
- Bearing in mind that the current laboratory-to-field shift factor included in the University of California Berkeley fatigue analysis and design procedure was calibrated against the Caltrans pavement design procedure "lowest cost" pavements, and the Valley asphalt mix, the fatigue performance predictions suggest that Caltrans "lowest cost" pavement designs are adequate at a 90-percent reliability level for all Traffic Indices and subgrade R-values of 20 and 40. These results also suggest that the Caltrans "lowest cost" pavement designs for a subgrade R-value of 5 may not be adequate at this reliability level. Moreover, all Caltrans "thinnest allowable asphalt concrete layer" designs are likely not adequate at the 90-percent reliability level. It would also appear that AASHTO pavement designs may not be adequate at the 90-percent reliability level. Substitution of the Coastal asphalt mix for the Valley asphalt mix did not change these conclusions.

- Pavements designed for high volumes of truck traffic (i.e., Traffic Indexes of 13 and 15) using the Caltrans "thinnest allowable asphalt concrete layer" design appear to be particularly vulnerable to premature fatigue cracking. Inclusion of thicker asphalt concrete layers such as those used in the "lowest cost" pavements included in this study appears to result in substantial improvement in predicted fatigue performance.
- The ability of the mechanistic-empirical pavement analysis and design procedure described herein to quantitatively evaluate the effects of: 1) pavement structure, including different layer thicknesses producing the same gravel equivalent; 2) materials selection; and 3) subgrade strengths; has been demonstrated. Moreover, the procedure directly relates the effects of pavement design, materials design, and loading to a specific mode of pavement distress. Effects of drainage on pavement performance can also be directly investigated provided stiffness characteristics of the unbound materials are determined by laboratory tests for different levels of saturation.

## 6.3 Recommendations

Based on the information presented herein, the following recommendations are made:

1. Given that Caltrans and AASHTO design methods are not interchangeable, the results of this report should be made available to agencies considering the use of the AASHTO method for pavement design in California.
2. Lifecycle cost analysis should be included in any Caltrans design software. While the Caltrans design manual requires lifecycle cost analysis for pavements designed for larger Traffic Indices, a standard procedure for these analyses is not included

in the design procedure document (*3*), nor is it a part of the current design software. (*9*)

3. Drainage considerations should be explicitly included in the Caltrans design procedure. The current design procedure assumes that inclusion of drainage features, such as asphalt treated permeable base (ATPB) and edge drains, will improve the adequacy of designs. Results from the study of ATPB performed as a part of the CAL/APT program indicate that this may not always be true. For circumstances where drainage features do improve pavement performance, the improvement should be quantified and included in lifecycle cost analyses. To assist in this process, drainage coefficients could be developed for the existing Caltrans procedures using the fatigue analysis and design procedure described in this report. To accomplish this requires the conduct of laboratory tests to define the effects of saturation on the stiffnesses of typical pavement materials and methods to estimate the effects of drainage on saturation.
4. Gravel Factors for asphalt concrete should be re-evaluated. Results presented in this report suggest that the structural value of asphalt concrete in reducing fatigue damage is underestimated by the current Gravel Factors. The difference between these factors and the apparent improvement in fatigue performance that comes from thicker asphalt concrete layers is especially apparent for heavy-duty pavements designed to carry large volumes of truck traffic.
5. Caltrans should move towards a mechanistic-empirical fatigue analysis and design procedure for the design of asphalt concrete pavements. One framework that might be followed is that utilized herein — it has the advantage of some

validation from the Goal 1 CAL/APT tests. Implementation can occur in stages, and the following approach has a high probability of success.

a. Validate the procedure and its elements, such as the shift factor and temperature conversion factor, by accelerated pavement testing. This first step is already being carried out as part of Goal 1 of the CAL/APT program.

b. Validate the procedure and its elements with long-term pavement performance data. This step is more difficult than the first step, but is necessary to obtain the confidence in the procedure needed for full-scale implementation.

c. Use the procedure to "shadow" the current design procedure. This step will permit comparison by designers and provide familiarity with the procedure. It will also permit Caltrans to identify designs in the current procedure that might not provide the desired performance because of materials (for example the asphalt concrete mix), drainage, or other factors not included in the current procedure.

d. Use the procedure to resolve questions and problems that cannot be addressed by the current procedure, such as:

- the effects of water damage on materials (an example of this use is included in Reference (*3*);
- the effects of trucks with different loading characteristics (weights, axle configurations, tire pressures, etc.) that might be permitted on

California highways under the North American Free Trade Agreement (NAFTA);

- the use of new materials, the properties of which can be determined in the laboratory and included in analyses of the type presented in Chapter 5 of this report; and
- the use of new types of asphalt pavement structures, for example, the "rich-bottom" structure for fatigue resistance presented in another report for Caltrans prepared by the University of California, Berkeley (*2*).

e. Full-scale implementation.

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