Pavement Analysis and Design

TE-503A / TE-503

Lecture-11
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DTEM

Overseas Road Note 31 Method

This Road Note gives recommendations for the structural design of bituminous surfaced roads in tropical and subtropical climates. It is aimed at highway engineers responsible for the design and construction of new road pavements and is appropriate for roads which are required to carry up to 30 million cumulative equivalent standard axles in one direction. The design of strengthening overlays is not covered nor is the design of earth, gravel or concrete roads. Although this Note is appropriate for the structural design of flexible roads in urban areas, some of the special requirements of urban roads, such as the consideration of kerbing, sub-soil drainage, skid resistance, etc., are not covered.

Overseas Road Note 31 Method-Basis for Design Catalogue

The pavement designs incorporated into the fourth edition of Road Note 31 are based primarily on:

(a) The results of full-scale experiments where all factors affecting performance have been accurately measured and their variability quantified.

(b) Studies of the performance of as-built existing road Networks.

Overseas Road Note 31 Method-Design Process

There are three main steps to be followed in designing a new road pavement. These are:

(i) estimating the amount of traffic and the cumulative number of equivalent standard axles that will use the road over the selected design life

(ii) assessing the strength of the subgrade soil over which the road is to be built

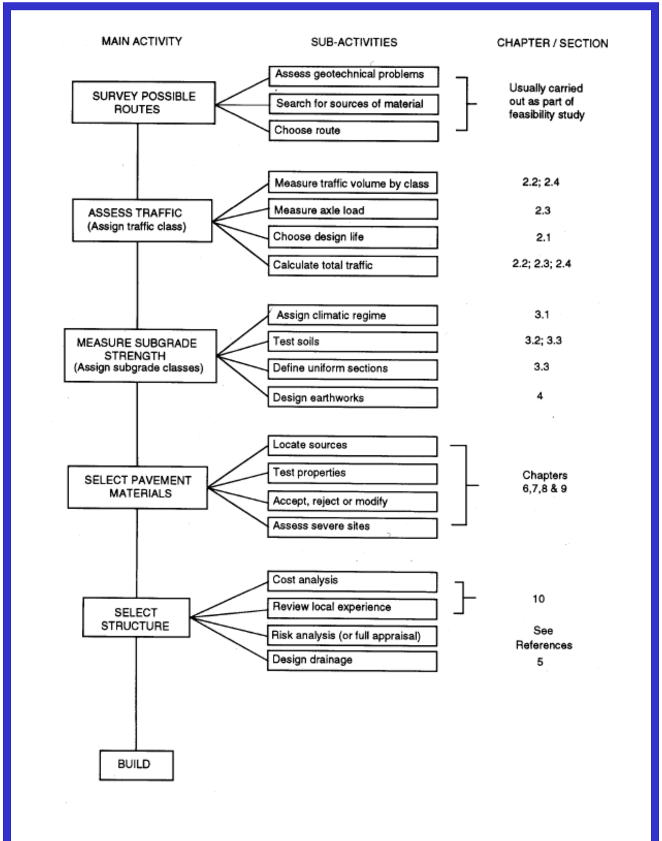
(iii) selecting the most economical combination of pavement materials and layer thicknesses that will provide satisfactory service over the design life of the pavement (It is usually necessary to assume that an appropriate level of maintenance is also carried out).

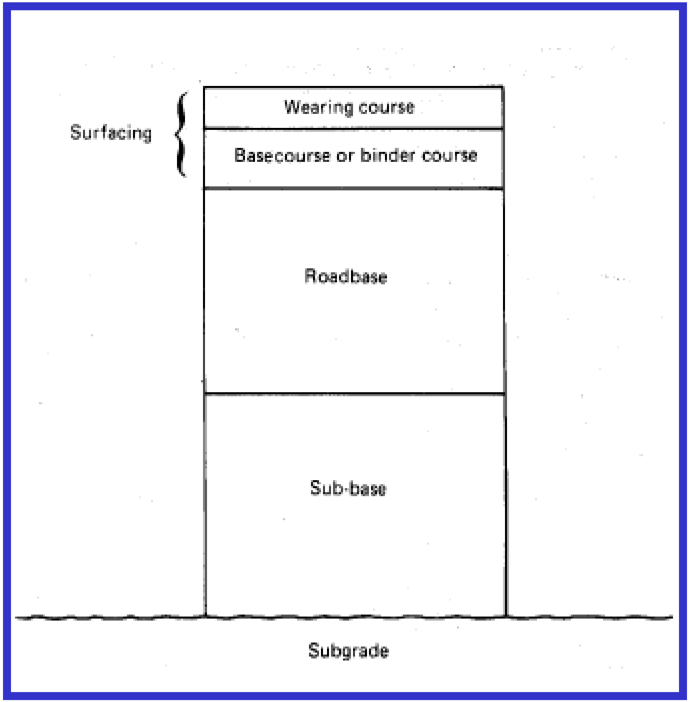
Overseas Road Note 31 Method-Design Process

This Note considers each of these steps in turn and puts special emphasis on five aspects of design that are of major significance in designing roads in most tropical countries:

- **The influence of tropical climates on moisture conditions in road subgrades.**
- **The severe conditions imposed on exposed bituminous surfacing materials by tropical climates and the implications of this for the design of such surfacings.**
- **The interrelationship between design and maintenance. If an appropriate level of maintenance cannot be assumed, it is not possible to produce designs that will carry the anticipated traffic loading without high costs to vehicle operators through increased road deterioration.**
- **The high axle loads and tyre pressures which are common in most countries.**
- **The influence of tropical climates on the nature of the soils and rocks used in road building.**

Flexible Pavement Design
Overseas Road Note 31 Method-Design Process





Overseas Road Note 31 Method-Traffic forecasting

Even with a developed economy and stable economic conditions, traffic forecasting is an uncertain process. In a developing economy the problem becomes more difficult because such economies are often very sensitive to the world prices of just one or two commodities. In order to forecast traffic growth it is necessary to separate traffic into the following three categories:

(a) Normal traffic. **Traffic which would pass along the existing road or track even if no new pavement were provided.**

(b) Diverted traffic. **Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.**

(c) Generated traffic. **Additional traffic which occurs in response to the provision or improvement of the road.**

Flexible Pavement Design

Overseas Road Note 31 Method-Equivalence factor

Equivalence factors for different axle loads

Wheel load (single & dual) (10^3 kg)	Axle load (10^3 kg)	Equivalence factor
1.5	3.0	0.01
2.0	4.0	0.04
2.5	5.0	0.11
3.0	6.0	0.25
3.5	7.0	0.50
4.0	8.0	0.91
4.5	9.0	1.55
5.0	10.0	2.50
5.5	11.0	3.83
6.0	12.0	5.67
6.5	13.0	8.13
7.0	14.0	11.3
7.5	15.0	15.5
8.0	16.0	20.7
8.5	17.0	27.2
9.0	18.0	35.2
9.5	19.0	44.9
10.0	20.0	56.5

$$\text{Equivalence factor} = \left(\frac{\text{Axle load (kg)}}{8160}\right)^{4.5}$$

Overseas Road Note 31 Method-Determination of cumulative equivalent standard axles

(i) Determine the daily traffic flow for each class of vehicle weighed using the results of the traffic survey and any other recent traffic count information that is available.

(ii) Determine the average daily one-directional traffic flow for each class of vehicle.

(iii) Make a forecast of the one-directional traffic flow for each class of vehicle to determine the total traffic in each class that will travel over each lane during the design life.

(iv) Determine the mean equivalence factor of each class of vehicle and for each direction from the results of this axle load survey and any other surveys that have recently been carried out.

(v) The products of the cumulative one-directional traffic flows for each class of vehicle over the design life of the road and the mean equivalence factor for that class should then be calculated and added together to give the cumulative equivalent standard axle loading for each direction. The higher of the two directional values should be used for design.

Overseas Road Note 31 Method-Traffic classes

Traffic classes

Traffic classes	Range (10^6 esa)
T1	< 0.3
T2	0.3 - 0.7
T3	0.7 - 1.5
T4	1.5 - 3.0
T5	3.0 - 6.0
T6	6.0 - 10
T7	10 - 17
T8	17 - 30

Overseas Road Note 31 Method-Subgrade strength classes

Subgrade strength classes

Class	Range (CBR %)
S1	2
S2	3 - 4
S3	5 - 7
S4	8 - 14
S5	15 - 29
S6	30

Flexible Pavement Design

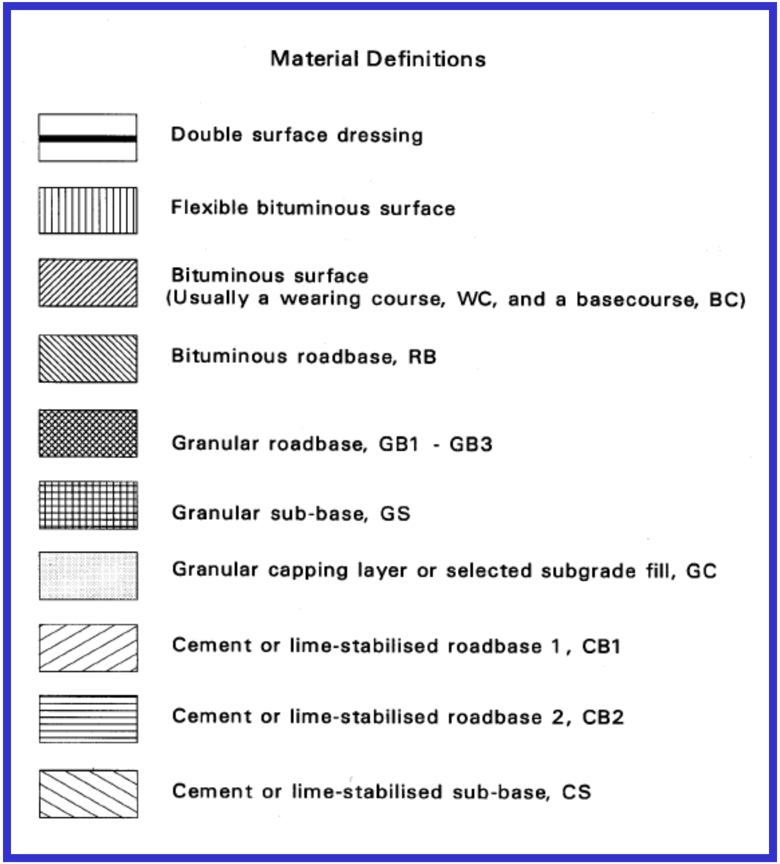
Overseas Road Note 31 Method-Properties of unbound materials

Properties of unbound materials

Code	Description	Summary of specification
GB1,A	Fresh, crushed rock	Dense graded, unweathered crushed stone, non-plastic parent fines
GB1,B	Crushed rock, gravel or boulders	Dense grading, PI < 6, soil or parent fines
GB2,A	Dry-bound macadam	Aggregate properties as for GB1,B (see text), PI < 6
GB2,B	Water-bound macadam	Aggregate properties as for GB1,B (see text), PI < 6
GB3	Natural coarsely graded granular material including processed and modified gravels	Dense grading, PI < 6 CBR after soaking > 80
GS	Natural gravel	CBR after soaking > 30
GC	Gravel or gravel-soil	Dense graded, CBR after soaking > 15

Notes 1. These specifications are sometimes modified according to site conditions, material type and principal use (see text).
2. GB = Granular roadbase, GS = Granular sub-base, GC = Granular capping layer.

Overseas Road Note 31 Method-Material Definitions



Material Definitions

- Double surface dressing
- Flexible bituminous surface
- Bituminous surface (Usually a wearing course, WC, and a basecourse, BC)
- Bituminous roadbase, RB
- Granular roadbase, GB1 - GB3
- Granular sub-base, GS
- Granular capping layer or selected subgrade fill, GC
- Cement or lime-stabilised roadbase 1, CB1
- Cement or lime-stabilised roadbase 2, CB2
- Cement or lime-stabilised sub-base, CS

Overseas Road Note 31 Method-Numerical problem

If predicted ESAL = 18.6×10^6 and subgrade CBR=4%, determine thicknesses of AC, base and subbase for a flexible pavement.

Traffic classes

Traffic classes	Range (10^6 esa)
T1	< 0.3
T2	0.3 - 0.7
T3	0.7 - 1.5
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T8	17 - 30

Subgrade strength classes

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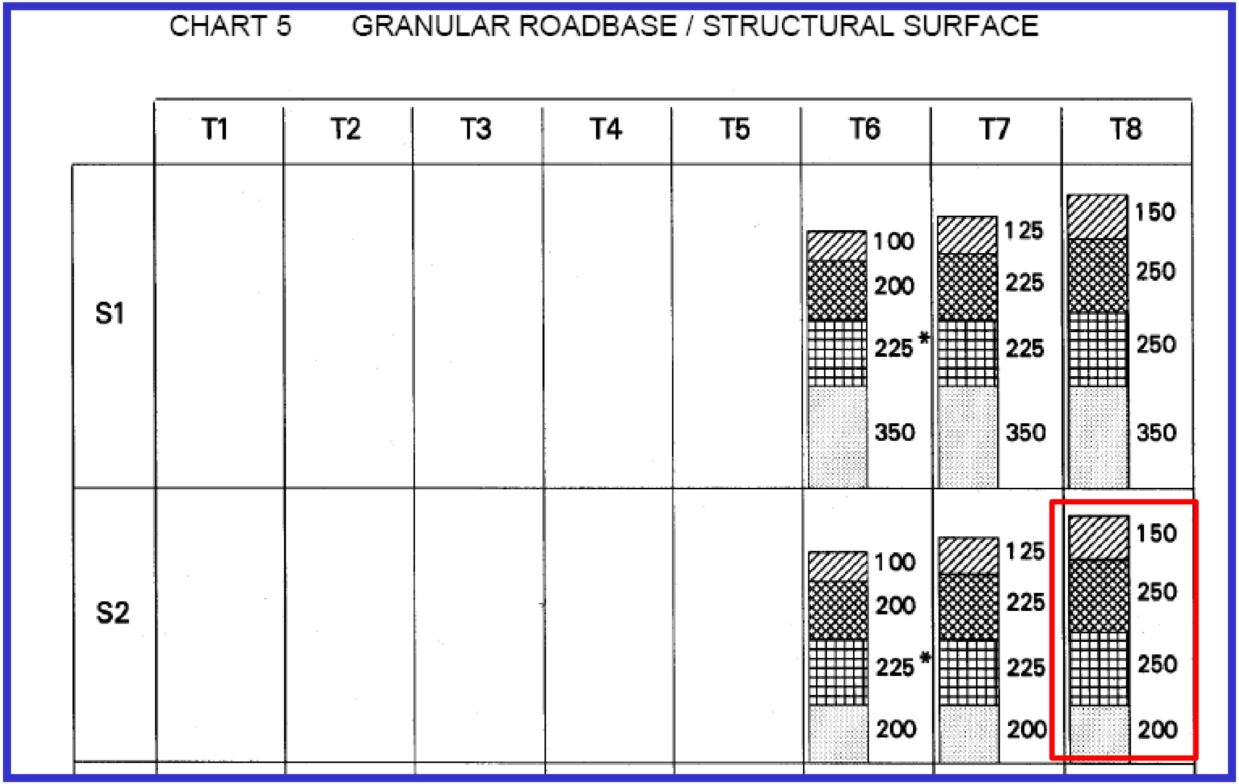
Flexible Pavement Design

Overseas Road Note 31 Method-Numerical problem



CALIBRATED MECHANISTIC DESIGN PROCEDURE

The design equations presented in the 1986 AASHTO design guide were obtained empirically from the results of the AASHO Road Test. To develop a mechanistic pavement analysis and design procedure suitable for future versions of AASHTO guide, a research project entitled "Calibrated Mechanistic Structural Analysis Procedures for Pavements" was awarded to the University of Illinois under NCHRP 1-26.

The research included both flexible and rigid pavements, and a two-volume report, hereafter referred to as Report 1-26, was prepared for the National Cooperative Highway Research Program (NCHRP, 1990). Some of the information presented herein was obtained from Report 1-26.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

The calibrated mechanistic procedure is a more specific name for the mechanistic-empirical procedure. It contains a number of mechanistic distress models that require careful calibration and verification to ensure that satisfactory agreement between predicted and actual distress can be obtained.

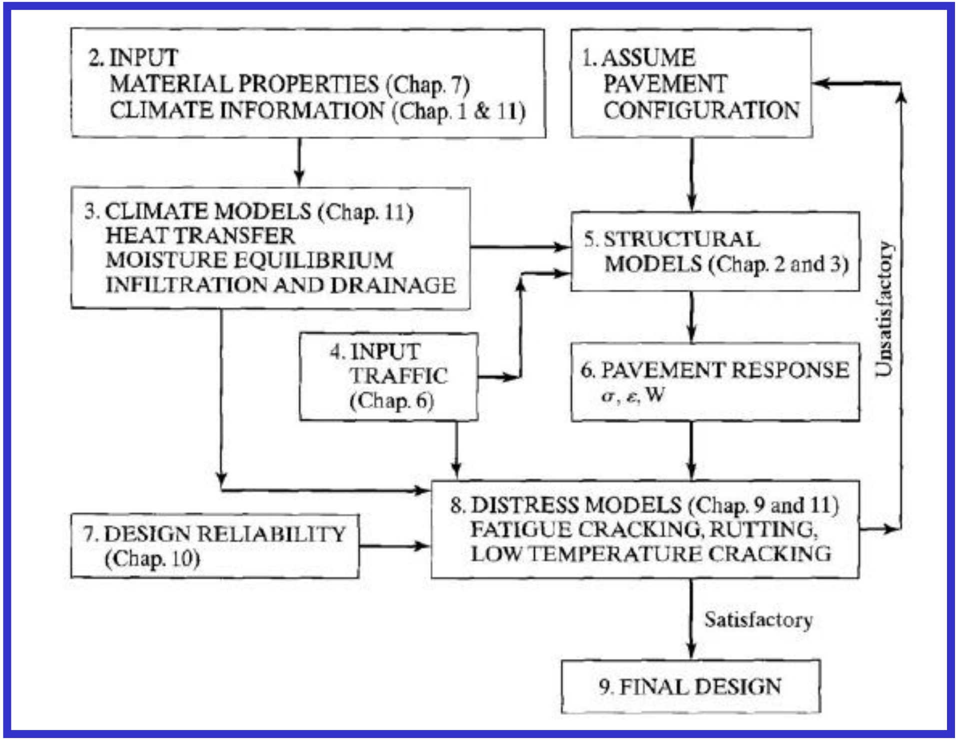
The purpose of calibration is to establish transfer functions relating mechanistically determined responses to specific forms of physical distress. Verification involves the evaluation of the proposed models by comparing results to observations in other areas not included in the calibration exercise. This procedure has been used in several design methods, such as the Asphalt Institute method. However, these existing methods are based on many simplifying assumptions and are not as rigorous as is desirable.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Figure shows a general methodology for flexible pavement design. In the figure, it is assumed that the materials to be used for the pavement structure are known and that only the pavement configuration is subjected to design iterations.

If changing the pavement configuration does not satisfy the design requirements, it might be necessary to change the types and properties of the materials to be used.

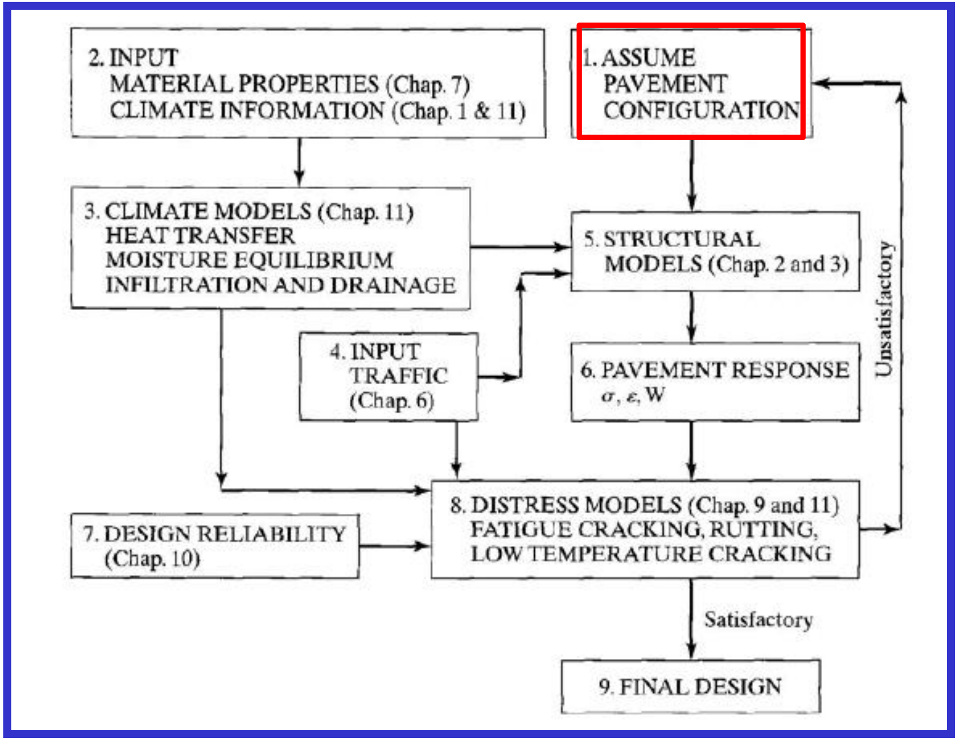
Once a new material is selected, the process is repeated until a satisfactory design is obtained.



Flexible Pavement Design

CALIBRATED MECHANISTIC DESIGN PROCEDURE

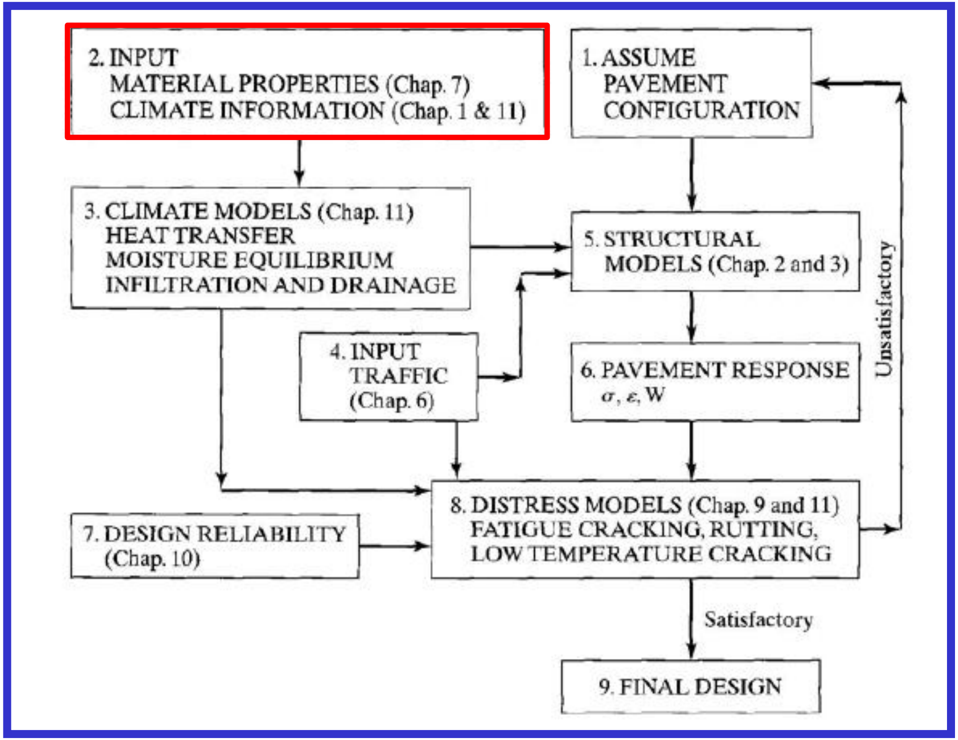
1.Pavement configuration includes the number of layers, the thickness of each layer, and the type of materials.



Flexible Pavement Design

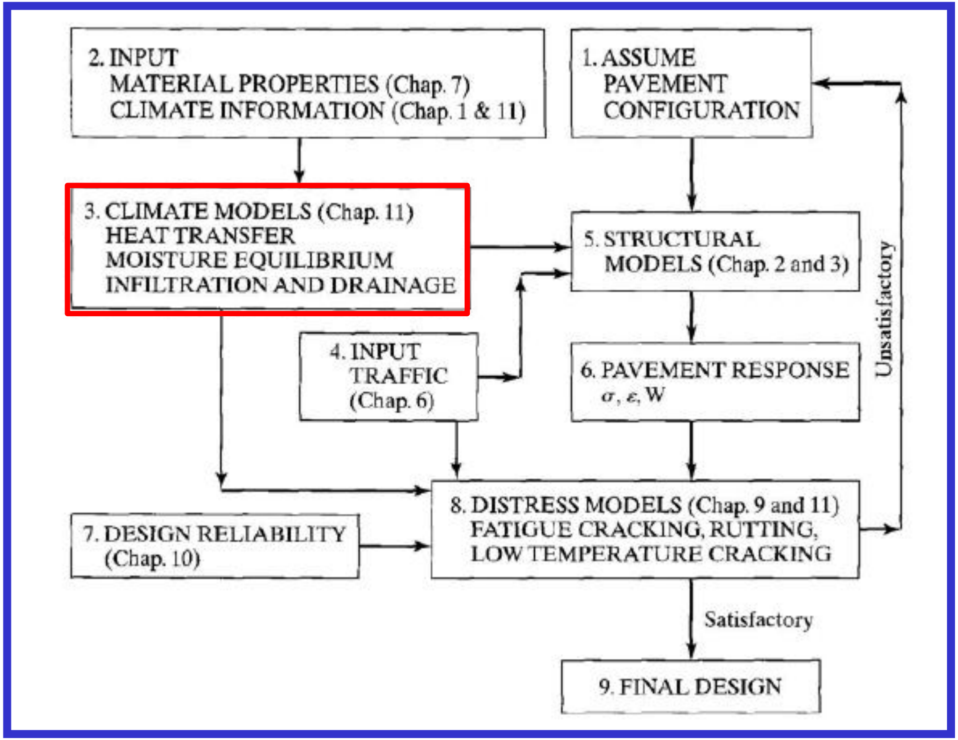
CALIBRATED MECHANISTIC DESIGN PROCEDURE

2. The basic material properties for the structural models are the resilient moduli of HMA, base, subbase, and subgrade; those for the distress models involve the various failure criteria, one for each distress. If temperature and moisture at different times of the year vary significantly, it is unreasonable to use the same modulus for each layer throughout the entire year. Each year should be divided into a number of periods, each with a different set of moduli based on the climatic data specified.



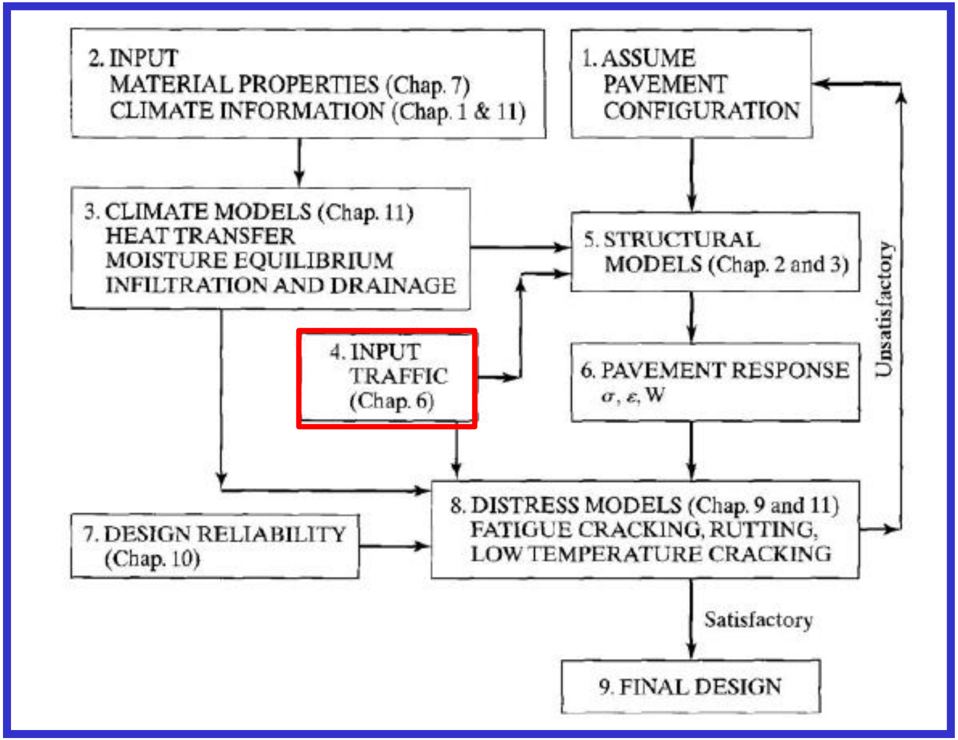
CALIBRATED MECHANISTIC DESIGN PROCEDURE

3. Climatic models include the heat transfer model (Dempsey and Thompson, 1970) for determining the temperature distribution with respect to space and time, the moisture equilibrium model (Dempsey et al., 1986) for determining the final moisture distribution in the subgrade, and the infiltration and drainage model (Liu and Lytton, 1984) for predicting the degree of saturation of granular bases.



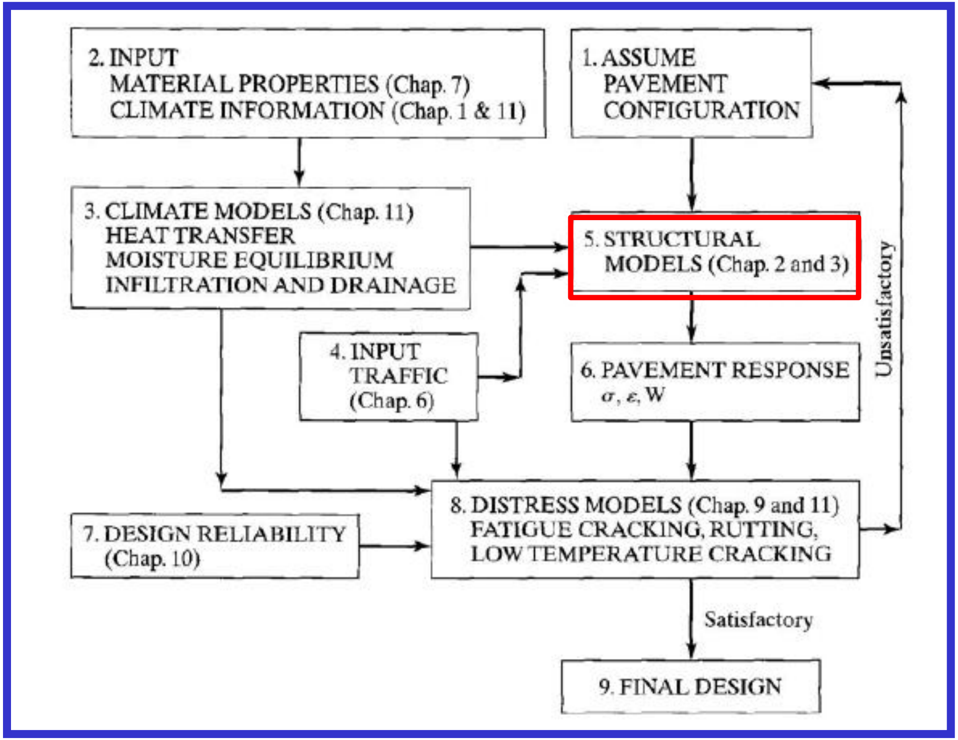
CALIBRATED MECHANISTIC DESIGN PROCEDURE

4. Traffic should be divided into a number of load groups, each with different load magnitudes and configurations and different numbers of load repetitions. When the design is based on each type of distress, it is unreasonable to use an equivalent single-axle load, because the equivalent factor for one type of distress is different from that for others. The load magnitude and configuration-for example; wheel spacing, contact radius, and contact pressure-are used in the structural models, but the number of repetitions is used in the distress models.



CALIBRATED MECHANISTIC DESIGN PROCEDURE

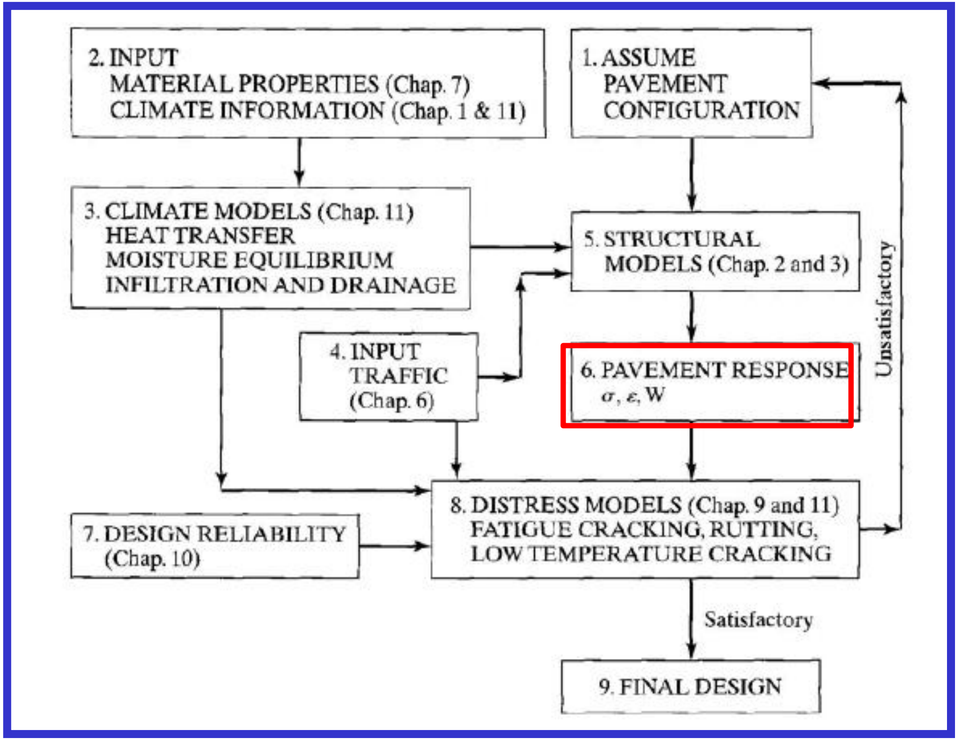
5. The finite element models can analyze nonlinear pavement systems more realistically than any other structural model by considering the variation of modulus within each layer.



Flexible Pavement Design

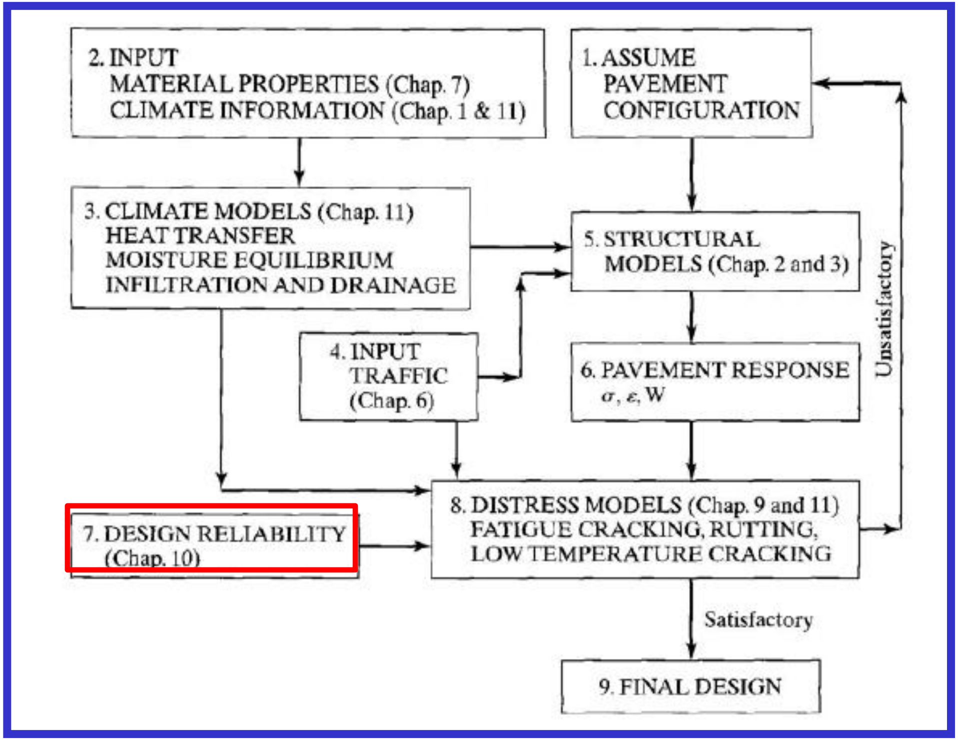
CALIBRATED MECHANISTIC DESIGN PROCEDURE

6. Pavement responses include stresses, strains and deflections. Theoretically, only those responses that contribute to each distress should be evaluated by the structural models and used in the distress models. However, if the responses can be related to surface deflections, it is possible to use surface deflections as input to the distress models. The most attractive feature of surface deflections is that they can be easily, rapidly, and inexpensively measured by utilizing automatic equipment.



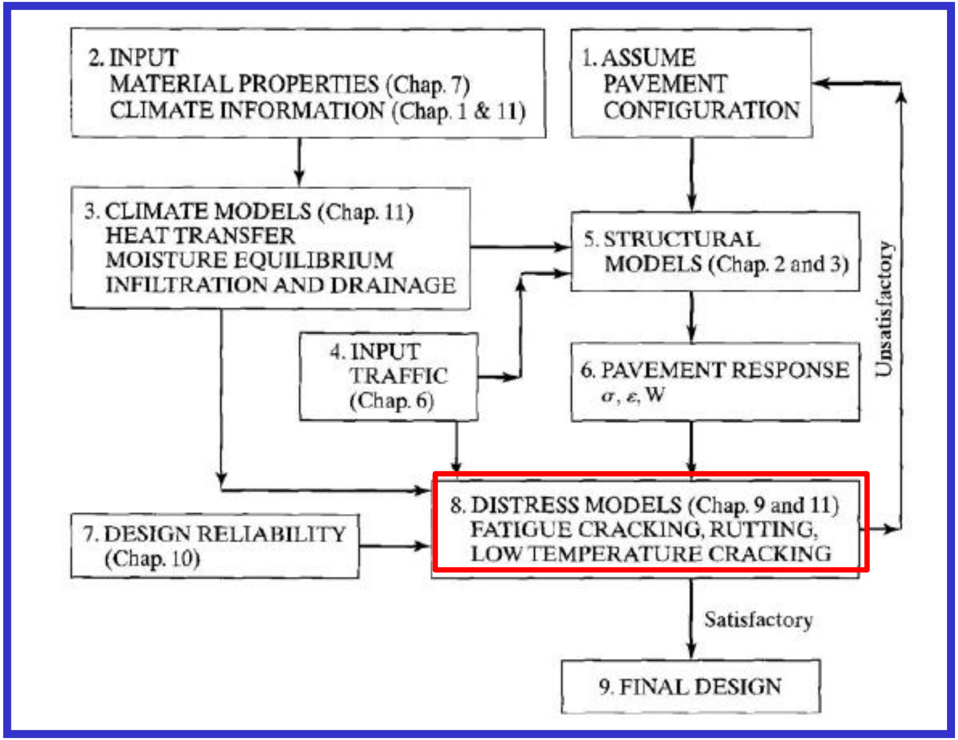
CALIBRATED MECHANISTIC DESIGN PROCEDURE

7. Variabilities of materials, climate, traffic, and construction practice require the use of a probabilistic method to evaluate pavement distress. To determine the design reliability, the variances or coefficients of variation of some input parameters reflecting materials, climate, traffic and even layer thicknesses must also be specified in steps 1 to 4 so that the variances of stress, strain and deflection can be computed for use in the distress models.



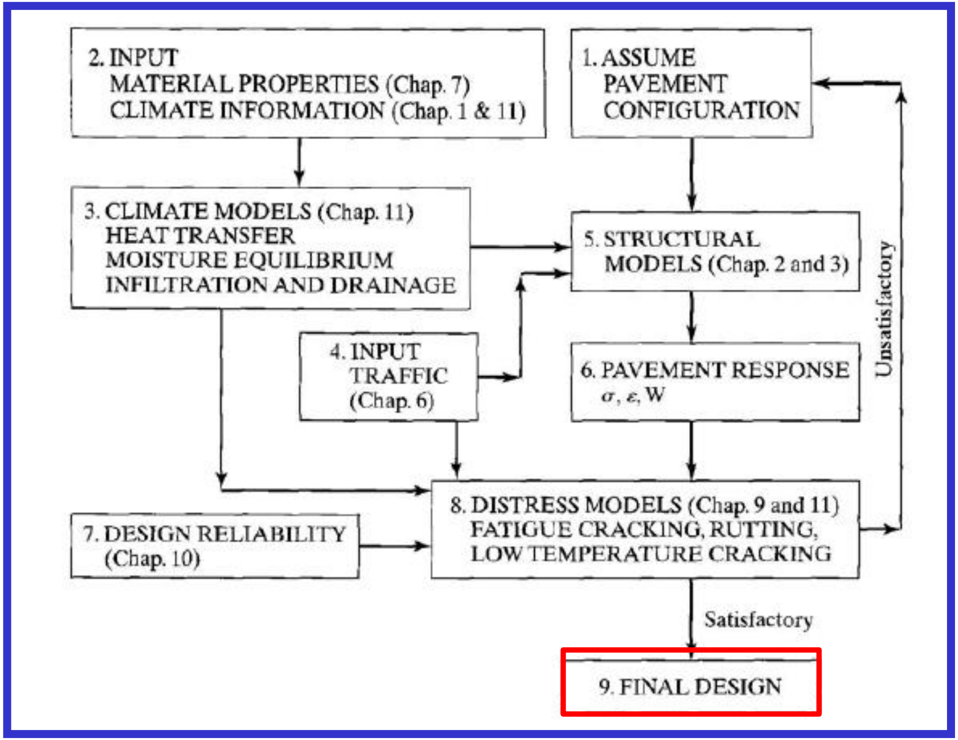
CALIBRATED MECHANISTIC DESIGN PROCEDURE

8. Distress models include fatigue cracking, rutting, and low-temperature cracking. The roughness or performance model can also be included. If the reliability for a certain distress is less than the minimum level required, the assumed pavement configuration should be changed, and steps 5, 6, and 8 should be repeated, until the required level of reliability is met.



CALIBRATED MECHANISTIC DESIGN PROCEDURE

9. The final design is selected when the assumed pavement thicknesses satisfy the reliability requirement for each type of distress.



CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models

Temperature and moisture are significant climatic inputs for pavement design. The modulus of the HMA depends on pavement temperature; the moduli of the base, subbase and subgrade vary appreciably with moisture content. Report 1-26 indicated that the current technology, as utilized in the climatic–materials–structural (CMS) model developed at the University of Illinois (Dempsey et al., 1986), is adequate for characterizing the pavement temperature regime, but that the capabilities for moisture modeling are not as advanced as those for temperature modeling.

The strength and modulus of cohesive soils (and of granular materials with a high percentage of fines) are very sensitive to even a small change in moisture content, say ±1%. Report 1-26 also indicated that the moisture equilibrium model in the CMS model for determining subgrade moisture is a reasonable and practical choice for design purpose.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Heat Transfer Model

The heat transfer model was originally developed at the University of Illinois (Dempsey and Thompson, 1970) for evaluating frost action and temperature regime in multilayered pavement systems. The model applies the finite difference method to solve the following Fourier equation for one-dimensional heat flow:

$$\frac{\partial T}{\partial t} = \alpha \frac{\partial^2 T}{\partial z^2}$$

Here, *T* is the temperature, *t* is the time, *z* is the depth below surface, and *α* is the thermal diffusivity, which is related to the thermal conductivity and heat capacity of the pavement materials. Given the initial temperature distribution and the two boundary conditions (at the pavement surface and at a depth *H* below the surface), the above Eq. can be solved.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

$$\frac{\partial T}{\partial t} = \alpha \frac{\partial^2 T}{\partial z^2}$$

Climate Models-Heat Transfer Model

Convection and radiation play a dominant role in transferring heat between the air and the pavement surface, whereas conduction plays a separate role in transferring heat within the pavement system. The total depth *H* is a variable input parameter in the heat transfer model. It can be determined from a study of deep soil temperatures at a given geographic location. For example, studies of soil temperatures in northern Illinois have indicated that the ground temperature is about 51°F (10.6°C) 12 ft (3.7 m) below surface. The temperature at this depth can be used as a boundary to solve the above Eq.

Inputs to the heat transfer model are the climatic data and the thermal properties of paving materials and soils. The climatic data include maximum and minimum daily air temperatures, percent sunshine and wind speed. The thermal properties include thermal conductivity, heat capacity and latent heat of fusion.

The model recognizes three sets of thermal properties, depending on whether the material is in an unfrozen, freezing or frozen condition. To facilitate the application of the heat transfer model to various pavement-related problems, a comprehensive climate data base for the state of Illinois was developed by Thompson et al. (1987).

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model

The moisture equilibrium model in the CMS model (Dempsey et al., 1986) is based on the assumption that the subgrade cannot receive moisture by infiltration through the pavement.

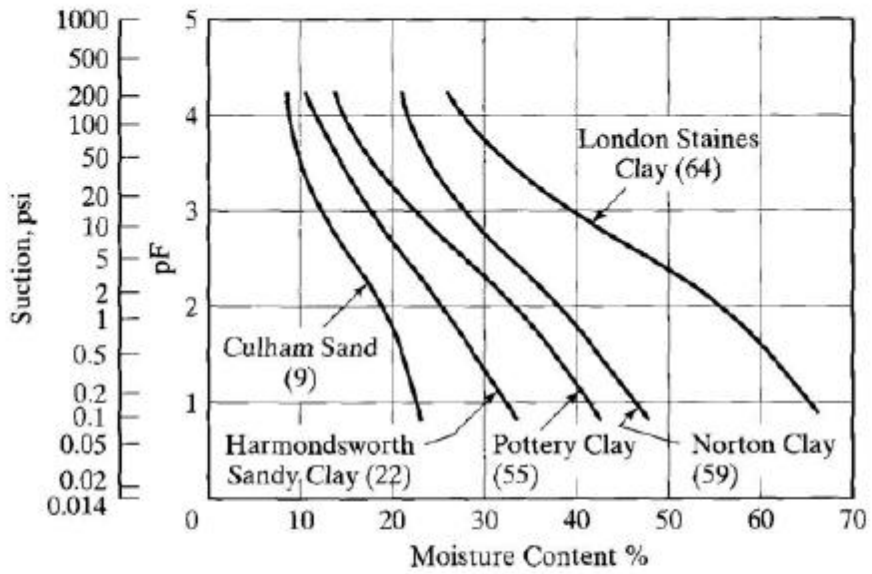
Any rainwater will drain out quickly through the drainage layer to the side ditch or longitudinal drain, so the only water in the subgrade is the capillary water caused by the water table.

Because of the thermodynamic relationship between soil suction and moisture content, a simple way to determine the moisture content in a soil is to determine its suction, which is related to the pore water pressure.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model

Figure shows the suction–moisture curves for five different soils with varying clay contents, as indicated by the numerals in parentheses under each soil title. These curves were obtained in the laboratory by drying tests, in which different levels of vacuum or suction were applied to a wet soil specimen until the moisture was reduced to an equilibrium value. The suction is expressed in the pF scale, which is the logarithm of water tension in cm, as defined by Schofield (1935).

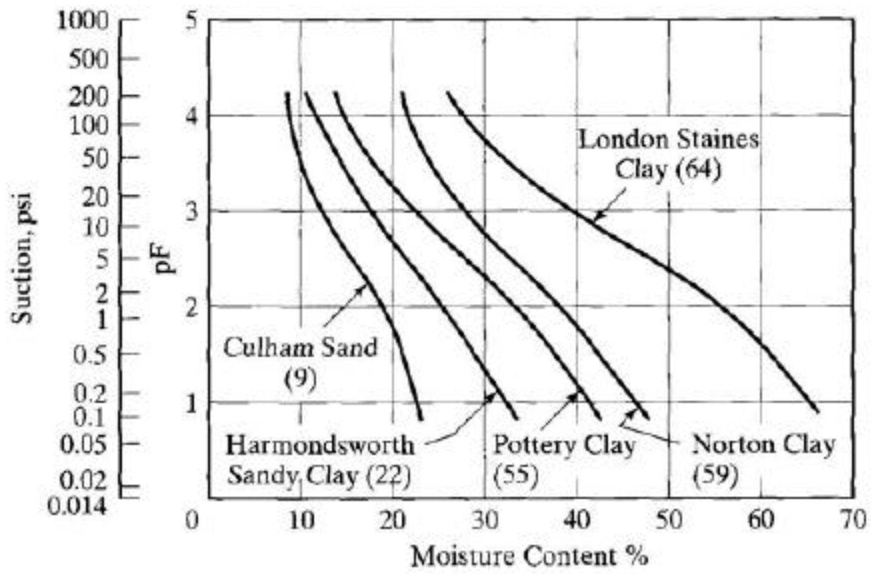


CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model

The corresponding values in terms of psi are shown on the left scale.

It can be seen that suction increases as the moisture content decreases or the clay content increases. The increase in suction is due to the smaller menisci formed between soil particles. In the CMS model, empirical relationships were used to define the suction-moisture curve based on the liquid limit, the plasticity index and the saturated moisture content of the soil.



CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model

When there is no loading or overburden pressure, suction is equal to the negative pore pressure. When a load or overburden is applied to an unsaturated soil with a given moisture content or suction, the suction or moisture content remains the same but the pore pressure becomes less negative. The relationship between suction and pore pressure can be expressed as:

$$u = S + \alpha p$$

in which *u* is the pore pressure when soil is loaded; *S* is the soil suction, which is a negative pressure; *p* is the applied pressure (or overburden), which is always positive; and *α* is the compressibility factor, varying from 0 for unsaturated, cohesionless soils to 1 for saturated soils. For unsaturated cohesive soils, *α* is related to the plasticity index *PI* by (Black and Croney, 1957):

$$\alpha = 0.03\, PI$$

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model

The pore pressure in a soil depends solely on its distance above the ground water table:

$$u = -z\,\gamma_w$$

Here, z is the distance above the water table, and γ_w is the unit weight of water. This simple fact can be explained by considering soils as a bundle of capillary tubes with varying sizes. Water will rise in each of these capillary tubes to an elevation that depends on the size of the tube. At any distance z above the water table, a large number of menisci will form at the air-water interfaces, causing a tension at each elevation corresponding to the height of capillary rise.

$$u = S + \alpha\,p$$

or

$$S = u - \alpha\,p$$

or

$$S = -z\,\gamma_w - \alpha\,p$$

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model

The procedures for determining the equilibrium moisture content at any point in a pavement system can be summarized as follows:

1. Determine the distance *z* from the point to the water table.
2. Determine the loading or overburden pressure *p*.
3. Determine the compressibility factor a from Eq. [$\alpha = 0.03\ PI$]
4. Determine the suction *S* from Eq. [$S = -z\,\gamma_w - \alpha\,p$]
5. Determine the moisture content from the suction–moisture curve.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model-Numerical problem

Figure-1 shows an 8-in. full-depth asphalt pavement on a subgrade composed of two different materials. The top 16 in. of subgrade is a Culham sand; below it is a Norton clay with a *PI* of 18. The relationship between the suction and moisture content of these two soils is shown in Figure-2. The water table is located 12 in. below the top of the clay. The unit weight of each material is shown in the figure-1. Predict the moisture contents at point A on top of the clay, point B at the bottom of the sand layer, and point C on top of the sand layer.

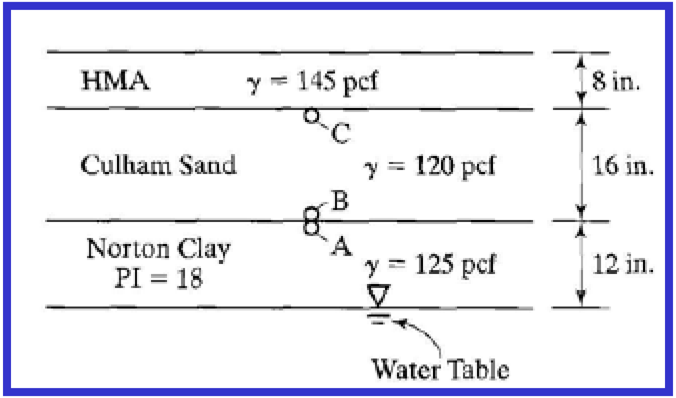


Figure-1

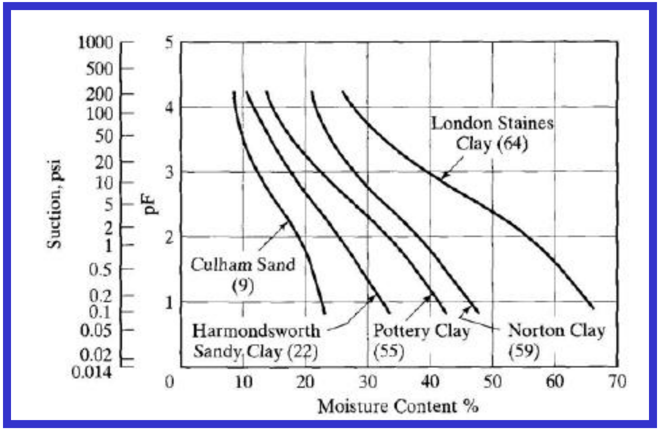
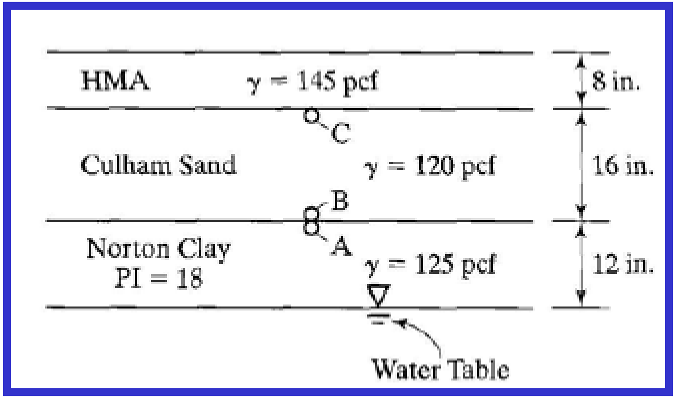


Figure-2

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Moisture Equilibrium Model-Numerical problem

Point	*p* (psf)	α = 0.03 PI	$S = -z\,\gamma_w - \alpha\,p$ (psf/psi)	mc (%) From fig.-2
A	256.7	0.54	-201 / -1.40	38
B	256.7	0	-62.4 / -0.43	21
C	96.7	0	-145.6 / -1.01	19.5



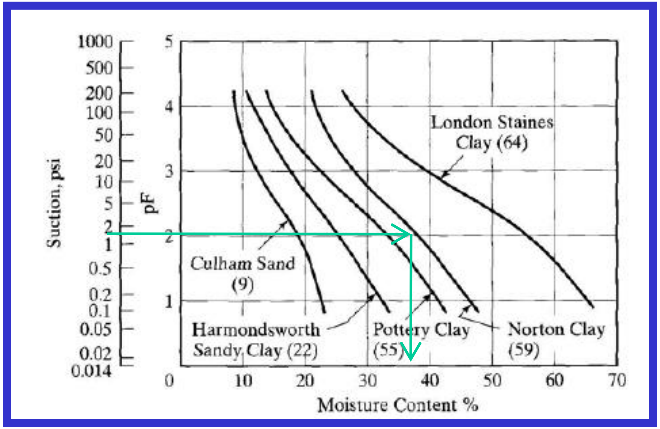


Figure-1

Figure-2

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Climate Models-Infiltration and Drainage Model

The infiltration and drainage model developed by Texas A&M University (Liu and Lytton, 1984) can be used to evaluate the effects of rainfall on the degree of saturation and the resilient moduli of the base course and subgrade. The degree of saturation for the base and the subgrade is predicted daily by considering the probability distribution of the amount of rainfall, the probabilities of wet and dry days, the infiltration of water into the pavement through cracks and joints, the drainage of the base course, and the wet and dry probabilities of pavement sub-layers.

Under a FHWA contract, Texas A&M University is in the process of upgrading the infiltration and drainage model. A research study has been initiated by the FHWA to combine the CMS model, the infiltration and drainage model, and the frost model, which is a mathematical model of coupled heat and moisture flow in soils developed by the Cold Regions Research and Engineering Laboratory (Johnson et a1.,1986).

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Structural Models

Report 1-26 recommended the use of elastic layer programs (ELP) and the ILLIPAVE finite element program for the development of a future AASHTO design guide.

It suggested the use of the modulus-depth relationship obtained from ILLI-PAVE to establish the various moduli for the ELP, thus capitalizing on the stress-sensitive feature of ILLI-PAVE and the multiple wheel capability of ELP.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models

Distress models are sometimes called transfer functions that relate structural responses to various types of distress. Distress models are the weak link in the mechanistic-empirical methods, and extensive field calibration and verification are needed to establish reliable distress predictions.

Usable transfer functions for HMA fatigue and subgrade rutting are available, but those for the rutting of HMA and granular materials are marginal and require further development.

Report 1-26 indicated that the use of the "rutting rate" concept advanced by Ohio State University (Majidzadeh et al., 1976) appears to be very promising, because it can be applied to all paving materials, including HMA, granular materials, and fine-grained soils.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models

The report also recommended the use of the Shahin-McCullough thermal cracking model (1972) as a checking procedure to assess the thermal cracking potential after the thickness design is completed; if it is unsatisfactory, a softer asphalt cement should be used.

Because HMA rutting is the major cause of permanent deformation on heavily traveled pavements with thick HMA, the design procedure may be simplified by checking the rutting potential after the thickness design is completed. If it is unsatisfactory, the selection of different mixture design procedures and practices should be made until the rut depth is reduced to the acceptable limit.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Fatigue Cracking Models

Miner's (1945) cumulative damage concept has been widely used to predict fatigue cracking. It is generally agreed that the allowable number of load repetition is related to the tensile strain at the bottom of the asphalt layer. The amount of damage is expressed as a damage ratio between the predicted and the allowable number of load repetitions. Damage occurs when the sum of damage ratios reaches the value 1. Because of variabilities, damage will not occur all at once when the ratio reaches exactly 1. If mean parameter values are used for design, a damage ratio of 1 indicates that the probability of failure is 50%-that is, that 50% of the area will experience fatigue cracking. By assuming the damage ratio to have a log normal distribution, the probability of failure, or the percentage of area cracked, can be computed and checked against field performance.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Fatigue Cracking Models

The major difference in the various design methods is the transfer functions that relate the HMA tensile strains to the allowable number of load repetitions. In the Asphalt Institute and Shell design methods, the allowable number of load repetitions N_f to cause fatigue cracking is related to the tensile strain ε_t at the bottom of the HMA and to the HMA modulus E_1 by:

$$N_f = f_1(\epsilon_t)^{-f_2} (E_1)^{-f_3}$$

For the standard mix used in design, the Asphalt Institute equation for 20% of area cracked is:

$$N_f = 0.0796(\epsilon_t)^{-3.291} (E_1)^{-0.854}$$

and the Shell equation is:

$$N_f = 0.0685(\epsilon_t)^{-5.671} (E_1)^{-2.363}$$

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Fatigue Cracking Models

Because f_2 is much greater than f_3, the effect of ε_t on N_f is much greater than that of E_1. Therefore, the E_1 term may be neglected:

$$N_f = f_1(\epsilon_t)^{-f_2}$$

This equation has been used by several agencies :

Illinois Department of Transportation (Thompson, 1987)

$$N_f = 5 \times 10^{-6} (\epsilon_t)^{-3.0}$$

Transport and Road Research Laboratory (Powell *et al.*, 1984)

$$N_f = 1.66 \times 10^{-10} (\epsilon_t)^{-4.32}$$

Belgian Road Research Center (Verstraeten *et al.*, 1982)

$$N_f = 4.92 \times 10^{-14} (\epsilon_t)^{-4.76}$$

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Fatigue Cracking Models

It can be seen that the exponent f_2 of the fatigue equations varies from 3.0 to 5.671, but the coefficient f_1 varies over several orders of magnitude-from 5 x 10^{-6} to 4.92 x 10^{-14}. The exponents f_2 and f_3 are usually determined from fatigue tests on laboratory specimens, but f_1 must shift from laboratory to field values by calibration. Pell (1987) indicated that the shift factor can range from 5 to 700. Differences in materials, test methods, field conditions, and structural models imply that a large variety of transfer functions is expected. No matter what transfer function is used, it is important to calibrate the function carefully, by applying an appropriate shift factor, so that the predicted distress can match field observations.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

Two procedures have been used to limit rutting: one to limit the vertical compressive strain on top of the subgrade, the other to limit the total accumulated permanent deformation on the pavement surface based on the permanent deformation properties of each individual layer. In the Asphalt Institute and Shell design methods, the allowable number of load repetitions N_d to limit rutting is related to the vertical compressive strain ε_c on top of the subgrade by:

$$N_d = f_4 \left(\epsilon_c\right)^{-f_5}$$

$$N_d = f_4 (\epsilon_c)^{-f_5}$$

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

TABLE 11.1 Subgrade Strain Criteria Used by Various Agencies

Agency	f_4	f_5	Rut depth (in.)
Asphalt Institute	1.365×10^{-9}	4.477	0.5
Shell (revised 1985)			
50% reliability	6.15×10^{-7}	4.0	
85% reliability	1.94×10^{-7}	4.0	
95% reliability	1.05×10^{-7}	4.0	
U.K. Transport & Road Research Laboratory (85% reliability)	6.18×10^{-8}	3.95	0.4
Belgian Road Research Center	3.05×10^{-9}	4.35	

Note. 1 in. = 25.4 mm.

As can be seen from Table 11.1, the exponent f_5 falls within a narrow range, but the coefficient f_4 varies a great deal. Both f_4 and f_5 should be calibrated by comparing the predicted performance with field observations.

$$N_d = f_4 (\epsilon_c)^{-f_5}$$

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

In the subgrade strain method, it is assumed that, if the subgrade compressive strain is controlled, reasonable surface rut depths will not be exceeded. For example, designs by the Asphalt Institute method are expected not to have a rut depth greater than 0.5 in. (12.7 mm) and designs by the TRRL procedure are expected not to have a rut depth of more than 0.4 in. (10.2 mm).

TABLE 11.1 Subgrade Strain Criteria Used by Various Agencies

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Note. 1 in. = 25.4 mm.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

Unless standard thicknesses and materials are used for design, the evaluation of surface rutting based on the subgrade strain does not appear to be reasonable. Under heavy traffic with thicker HMA, most of the permanent deformation occurs in the HMA, rather than in the subgrade. Because rutting is caused by the accumulation of permanent deformation over all layers, it is more reasonable to determine the permanent deformation in each layer and sum up the results.

Many methods are available to determine rut depth. Here, only the Ohio State model, which was recommended in Report 1-26, will be described. The Ohio State model assumes a linear relationship between permanent strain ε_p and number of load repetitions N when plotted in log scales. However, instead of total rutting, the rutting rate is considered, as indicated by:

$$\frac{\epsilon_p}{N} = A(N)^{-m}$$

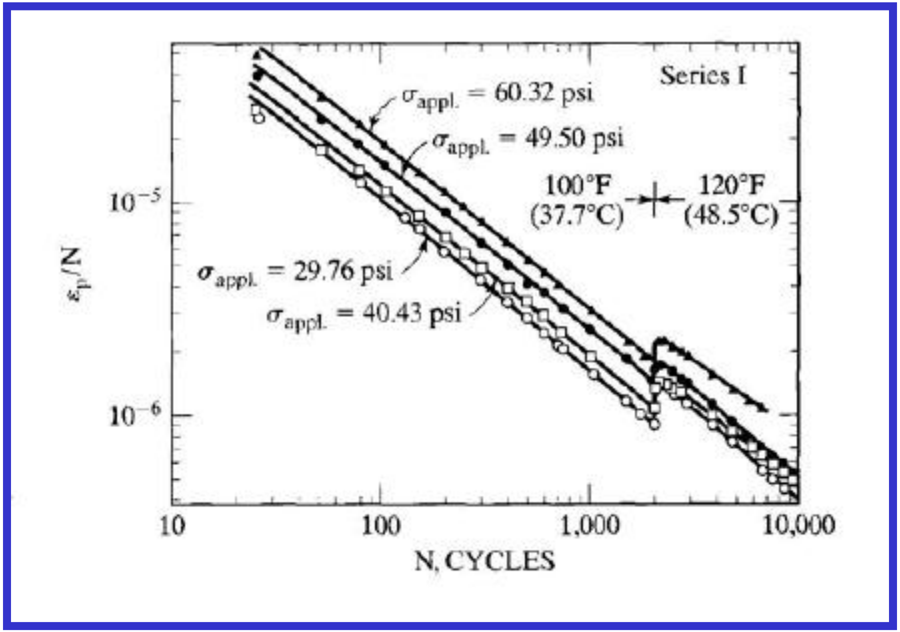
in which A is an experimental constant depending on material type and state of stress and m is an experimental constant depending on material type.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

$$\frac{\epsilon_p}{N} = A(N)^{-m}$$

Figure shows typical straight-line relationships between *log(ε_p/N)* & *logN* of HMA specimens under different stress and temperature conditions. All lines are nearly parallel, indicating that the value of *m*, which is the slope of the straight lines, is a constant independent of the levels of stress and temperature.



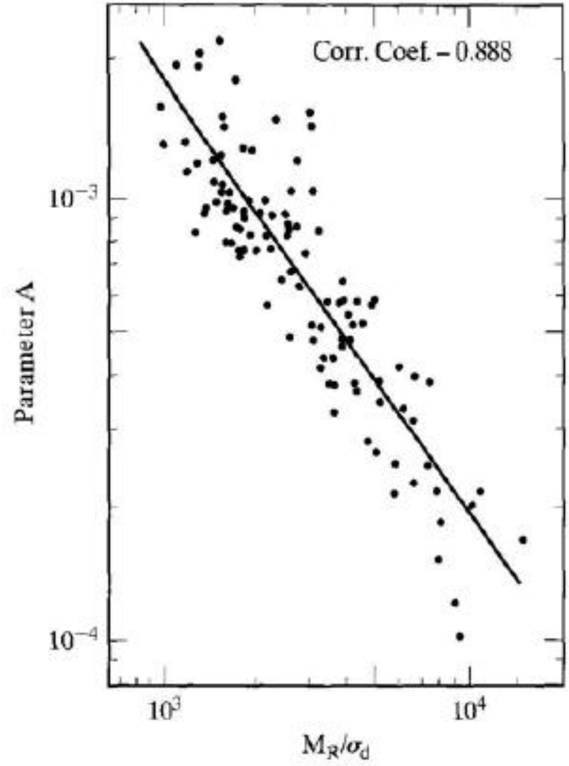
CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

To facilitate the computation of permanent deformation without subdividing the system into a large number of layers, the Ohio State model applies the direct method and defines the parameter *A* directly as a material property. Khedr (1986) conducted a large number of repeated load tests on HMA specimens and found a straight-line relationship between *logA* and $log(M_R/\sigma_d)$, as shown in Figure. Therefore, the value of *A* can be expressed as:

$$A = a\left(\frac{M_R}{\sigma_d}\right)^{-b} = a(\epsilon)^b$$

in which *a* and *b* are material constants, ε is the resilient strain, M_R is the resilient modulus and σ_d is the applied stress.



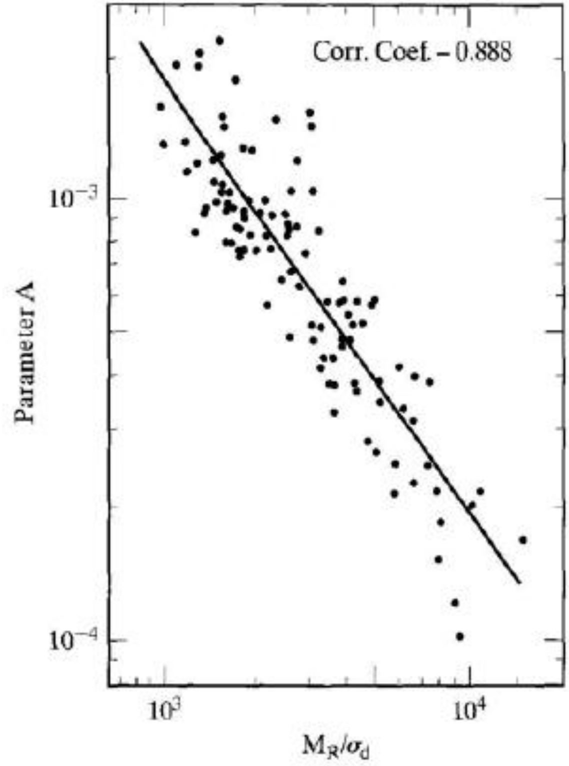
Flexible Pavement Design

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models

The final form of the Ohio State model is:

$$\epsilon_p = a(\epsilon)^b (N)^{1-m}$$

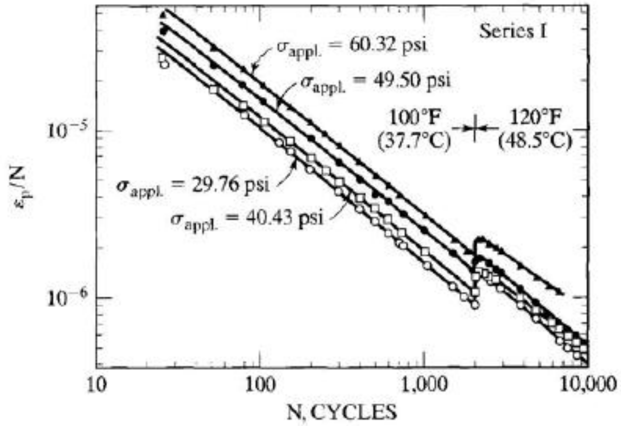


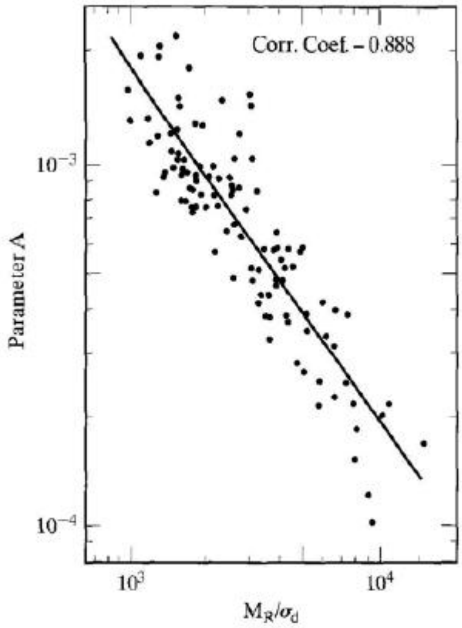
CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Rutting Models-Numerical problem

Based on the test results shown in Figures, estimate the coefficients of permanent deformation in the Ohio State model, as indicated by a, b, and m in Eq.

$$\epsilon_p = a(\epsilon)^b (N)^{1-m}$$

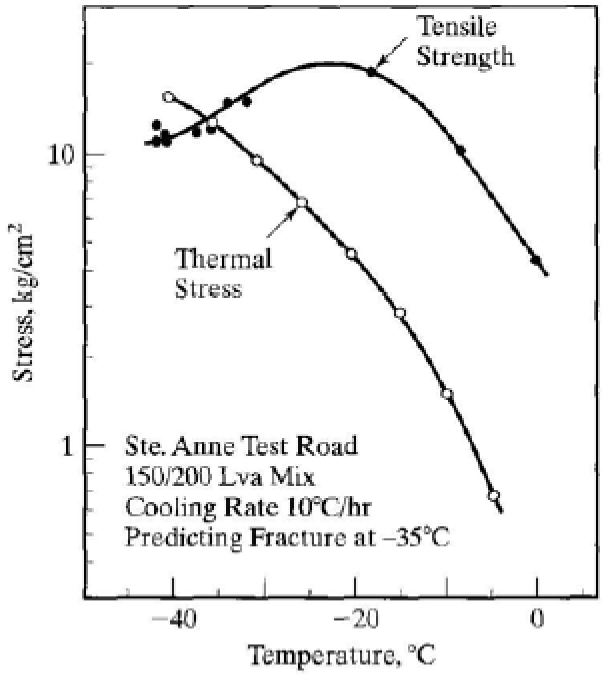




CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Thermal Cracking Models

There are two forms of thermal cracking in asphalt pavements: low-temperature cracking, and thermal fatigue cracking. The mechanism of low temperature cracking is illustrated in Figure. When the pavement temperature decreases, the tensile stress always increases, but the tensile strength increases only to a maximum and then decreases. Low-temperature cracking occurs when the thermal tensile stress in the HMA exceeds its tensile strength.



CALIBRATED MECHANISTIC DESIGN PROCEDURE

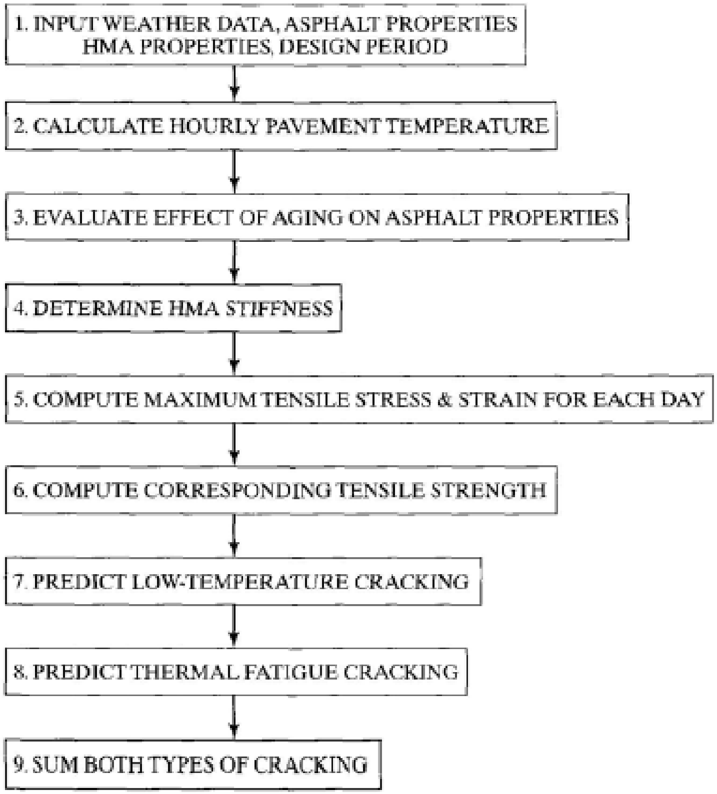
Distress Models-Thermal Cracking Models

If the tensile stress is smaller than the tensile strength, the pavement will not crack under a single daily temperature cycle but could still crack under a large number of cycles. This is called thermal fatigue cracking, which occurs when the fatigue consumed by daily temperature cycles exceeds the HMA fatigue resistance.

Several mechanistic thermal cracking models are available-for example, those developed by Finn et al. (1986), Ruth et al. (1982), Lytton et al. (1983) and Shahin and McCullough (1972). The latter two are the most comprehensive models that examine both low-temperature and thermal fatigue cracking and were recommended by Report 1-26 for further studies. They use the same basic structure to examine the accumulation of damage but with very different approaches. The model developed by Lytton et al. is more theoretical and is based on the principles of viscoelastic fracture mechanics, whereas that from Shahin and McCullough is more phenomenal and much easier to understand. Figure shows how thermal cracking is analyzed in the Shahin-McCullough model.

CALIBRATED MECHANISTIC DESIGN PROCEDURE

Distress Models-Thermal Cracking Models



Assignment No. 5

Pavement Analysis and Design by Yang H. Huang

Chapter-11

Problems 11.1 to 11.12 (Pages 530-532)

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