PAVEMENT DESIGN

18.1 TRAFFIC CONVERSION TO EQUIVALENT SINGLE AXLE LOAD

Empirical designs are based on commercial vehicles or on equivalent 8.16 ton single axle (ESA) or on equivalent 18 kips single axle load (EAL). Most of the recent empirical design tables are based on equivalent axle load, either ESA or EAL (see Table 18.1).

It is, therefore, necessary to convert the traffic to equivalent single axle loads. Vehicles of different axle loads can be converted to equivalent single axle loads by using empirical relationships established by the American Association of State Highway Officials (AASHO) Road Test. The equivalency factor, sometimes also called damaging power, depends upon the number of axles of the vehicle, structural number of the pavement, and current serviceability of the pavement. The following equation has been derived from the AASHO Road Test for:

$$F_j = \frac{W_{18}}{W_x} = [\frac{L_x + L_2}{18 + 1}] \; 4.79 \; [\frac{10 \; G/\beta_{18}}{10 \; G/\beta_x}] = \frac{1}{L_2^{4.33}}$$

where,

F_j	=	$\frac{W_{18}}{W_x}$	=	axle load equivalency,
W_{18}	=	axle load in terms of a single axle of 18,000 pounds,		
L_x	=	axle load of a given vehicle W_x,		
L_2	=	code for axle configuration, 1 for a single axle and 2 for a tandem axle,		
G	=	$\log [\frac{4.2 - P_t}{4.2 - 1.5}]$	=	a function of the ratio of loss in serviceability at time t to the potential loss taken to a point where $P_t = 1.5$,
P_t	=	terminal serviceability index,		
B	=	$0.4 + \frac{0.081 \; (L_x + L_2)^{3.23}}{(SN + 1)^{5.19} \; L^{3.23}}$, and		
SN	=	structural number.		

An axle load study should be carried out to determine the axle loads of the different traffic on existing roads.

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* All figures and tables without source in this chapter are the author's own work.

Table 18.1 AASHTO equivalent factors - flexible pavement

Axle Load (kips)	Structural Number, SN 1	2	3	4	5	6
2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	0.002	0.003	0.002	0.002	0.002	0.002
6	0.01	0.01	0.01	0.01	0.01	0.01
8	0.03	0.04	0.04	0.03	0.03	0.03
10	0.08	0.08	0.09	0.08	0.08	0.08
12	0.16	0.18	0.19	0.18	1.17	0.17
14	0.32	0.34	0.35	0.35	0.34	0.33
16	0.59	0.60	0.61	0.61	0.60	0.60
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.59	1.56	1.55	1.57	1.60
22	2.49	2.44	2.35	2.31	2.35	2.41
24	3.71	3.62	3.43	3.33	3.40	3.51
26	5.36	5.21	4.88	4.68	4.77	4.96
28	7.54	7.31	6.78	6.42	6.52	6.83
30	10.38	10.03	9.24	8.65	8.73	9.17
32	14.00	13.51	12.37	11.46	11.48	12.17
34	18.55	17.87	16.30	14.97	14.87	15.63
36	24.20	23.30	21.16	19.28	19.02	19.93
38	31.14	29.95	27.12	24.55	24.03	25.10
40	39.57	38.02	34.34	30.92	30.40	31.25

Single Axles, Pt = 2.0

Tandem axles, Pt = 2.0

Axle Load (kips)	Structural Number, SN 1	2	3	4	5	6
10	0.01	0.01	0.01	0.01	0.01	0.01
12	0.01	0.02	0.02	0.01	0.01	0.01
14	0.02	0.03	0.03	0.03	0.02	0.02
16	0.04	0.05	0.05	0.05	0.04	0.04
18	0.07	0.08	0.08	0.08	0.07	0.07
20	0.10	0.12	0.12	0.12	0.11	0.10
22	0.16	0.17	0.18	0.17	0.16	0.16
24	0.23	0.24	0.26	0.25	0.24	0.23
26	0.32	0.34	0.36	0.35	0.34	0.33
28	0.45	0.46	0.49	0.48	0.47	0.46
30	0.61	0.62	0.65	0.64	0.63	0.62
32	0.81	0.82	0.84	0.84	0.83	0.82
34	1.06	1.07	1.08	1.08	1.08	1.07
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.76	1.75	1.73	1.72	1.73	1.74
40	2.22	2.19	2.15	2.13	2.16	2.18
42	2.77	2.73	2.64	2.62	2.66	2.70
44	3.42	3.36	3.23	3.18	3.24	3.31
46	4.20	4.11	3.92	3.83	3.91	4.02
48	5.10	4.98	4.72	4.58	4.68	4.83

Table 18.1 (continued)

Axle Load (kips)	Structural Number, SN					
	1	2	3	4	5	6
2	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	0.003	0.004	0.004	0.004	0.003	0.002
6	0.01	0.02	0.02	0.01	0.01	0.01
8	0.03	0.05	0.05	0.04	0.03	0.03
10	0.08	0.10	0.12	0.10	0.09	0.08
12	0.17	0.20	0.23	0.21	0.19	0.18
14	0.33	0.36	0.40	0.39	0.36	0.34
16	0.59	0.61	0.65	0.65	0.62	0.61
18	1.00	1.00	1.00	1.00	1.00	1.00
20	2.61	1.57	1.49	1.47	1.51	1.55
22	2.48	2.38	2.17	2.09	2.18	2.30
24	3.69	3.49	3.09	2.89	3.03	3.27
26	5.33	4.99	4.31	3.91	4.09	4.48
28	7.49	6.98	5.90	5.21	5.39	5.98
30	10.31	9.55	7.94	6.83	6.97	7.79
32	13.90	12.82	10.52	8.85	8.88	9.95
34	18.41	16.94	13.74	11.34	11.18	12.51
36	24.02	22.04	17.73	14.38	13.93	15.50
38	30.90	28.30	22.61	18.06	17.20	18.98
40	39.26	35.89	28.51	22.50	21.08	23.04

Single axles, $P_t = 2.5$ Tandem axles, $P_t = 2.5$

Axle Load (kips)	Structural Number, SN					
	1	2	3	4	5	6
10	0.01	0.01	0.01	0.01	0.01	0.01
12	0.02	0.02	0.02	0.02	0.01	0.01
14	0.03	0.04	0.04	0.03	0.03	0.02
16	0.04	0.07	0.07	0.06	0.05	0.04
18	0.07	0.10	0.11	0.09	0.08	0.07
20	0.11	0.14	0.16	0.14	0.12	0.11
22	0.16	0.20	0.23	0.21	0.18	0.17
24	0.23	0.27	0.31	0.29	0.26	0.24
26	0.33	0.37	0.42	0.40	0.36	0.34
28	0.45	0.49	0.55	0.53	0.50	0.47
30	0.61	0.65	0.70	0.70	0.66	0.63
32	0.81	0.84	0.89	0.89	0.86	0.83
34	1.06	1.08	1.11	1.11	1.09	1.08
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.75	1.73	1.69	1.68	1.70	1.73
40	2.21	2.16	2.06	2.03	2.08	2.14
42	2.76	2.67	2.49	2.43	2.51	2.61
44	3.41	3.27	2.99	2.88	3.00	3.16
46	4.18	3.98	3.58	3.40	3.55	3.79
48	5.08	4.80	4.25	3.98	4.17	4.49

Source: AASHTO Pavement Design Guide 1985

Private cars do not contribute significantly to the structural damage caused to road pavements by traffic. Therefore, for the purpose of structural design, cars can be ignored and only the total number and axle loading of commercial vehicles, that will use the road during its design life need be considered. A commercial vehicle is defined as any goods or public service vehicle that has an unladen weight of 1.5 tons or more.

18.2 LANE DISTRIBUTION OF TRAFFIC

The design traffic load for pavement design is obtained by determination of the actual traffic in the design lane. This can be done either by actual studies of traffic in each lane or by an empirical method of distribution of the total traffic in both directions. The traffic distribution by direction varies from 0.3 to 0.7 of the total of two direction traffic. Normally the directional distribution is taken as 0.5.

Once the traffic in each direction is known, this is again distributed by lane, if there is more than one lane for the traffic in a particular direction. For single lane, one-directional traffic the distribution is taken as 1.0 of the directional traffic. This lane distribution is reduced by 0.2 for each additional lane.

The following distribution may be followed for design of traffic in developing countries by using empirical design nomographs, most of which are developed for roads with two or more lanes.

i) Single-lane (3.5 - 3.75 width) :
design traffic = 2 x traffic in both directions, to account for channelization of wheel loads.

ii) Single-lane (4 to 4.5m width) :
design traffic = 1.5 x traffic in both directions.

iii) Single-lane (4.5 to 5.5m width) :
design traffic = 1 x traffic in both directions.

iv) Two-lane, single carriageway road (5.5 to 6.5m width) :
design traffic = 0.75 x traffic in both directions.

v) Two-lane, single carriageway roads (6.5 to 7.5m width) :
design traffic = 0.5 x traffic in both directions.

The traffic need not be increased for load concentration effect as in cases i) and ii) above when design is carried out using a Structural Number (SN) or mechanistic empirical methods.

18.3 DESIGN LOAD

The design traffic is considered in terms of the cumulative number of standard axles (in the lane carrying maximum traffic) to be carried during the design life of the road.

The following equation may be used to calculate the total design traffic load:

$$N_D = \frac{365 \times N\,[(1 + r)^n - 1]}{r}$$

where,

N_D	=	total design traffic in ESA,
N	=	initial average daily traffic in the design lane in ESA,
r	=	growth rate, and
n	=	design life in years.

18.4 DESIGN METHODS

Some of the methods based on material characterization for pavement design are:

- CBR method**,
- R - value method**,
- Structural number method**, and
- Mechanistic design method.***

These methods are commonly known by agency, examples of which are given below.

1. National Crushed Stone Association
2. California
3. AASHTO, 1985
4. Transport and Road Research Laboratory
5. Asphalt Institute

The following discussion is an introduction to a few of the several methods of pavement design. The reader is, however, advised to refer to appropriate literature for further details.

18.4.1 *CBR Method*

In this method the California Bearing Ratio (CBR) used is for material characterization. The thickness of different layers of a pavement can be obtained by using CBR values of the materials to be used in different layers. Standard design charts, or nomographs, prepared by several agencies are available which allow determination of thickness against the input of traffic load and CBR. Most agencies have developed design charts relating axle load to the thickness of surface and sub-base of a given material type for various sub-grade CBR values. This requires adaptation of the base and sub-base materials having the same properties as specified in the development of design charts. Adjustment for variability in the properties of base, sub-base, and surface materials available is therefore not possible in using such design charts.

** empirical methods

*** Analytical and empirical method

18.4.2 *U.S. Corps of Engineers' Method*

Section 18.5.1 is an example of design using this method.

18.4.3 *The TRRL Method*

The Transport and Road Research Laboratory's (TRRL) Road Note 31 and Road Note 29 describe empirical methods based on performance tests for the design of pavements for traffic loads of up to 2.5 million ESA and greater than 2.5 million ESA respectively. TRRL Road Note 31 is especially developed for the tropical and subtropical conditions of developing countries. Traffic load and sub-grade CBR are basic inputs. The surface, base, and sub-base materials have to conform to those adopted in the design for application of this method. Sections 18.5.2 and 18.5.3 present design example by this method.

18.4.4 *R-value Method*

This method uses R-value, which is a stabilometer value to characterize the property of each layer as against CBR value in the CBR method. This method is based on California methods.

18.4.5 *Structural Number (AASHTO 1985) Method*

The Structural Number (SN) is defined as an index number derived from an analysis of traffic, roadbed, soil conditions, and regional factors that may be converted to the thickness of various flexible-pavement layers through the use of suitable layer coefficients related to the type of material being used in each layer of the pavement structure. The layer coefficients (designated by a, a_2, and a_3, for surface, bases, and sub-base respectively) give the empirical relationship between SN for a pavement structure and layer thickness and express the relative ability of a material to function as a structural component of the pavement.

Analytically, the SN is given by:

SN	=	$a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$,
D_1, D_2, D_3	=	actual thickness of surface, base, and sub-base, and
m_2, m_3	=	drainage coefficients for base and sub-base respectively.

The Structural Number Design Method is based the on the 1950-1960 American Association of State Highway Officials (AASHO) Road Test. The advantages of the SN method of design are:

- provision for drainage conditions,
- flexibility to design for variable material properties for different layers, and
- use or reliability.

Section 18.5.4 presents an example of design by this method.

18.4.6 *Mechanistic Empirical Method*

This method, also known as the layered elastic design method, is an analytical method that calculates stresses and strains at different depths of the pavement layer. Fatigue of asphalt surface and rutting of sub-grade, under the designed load, are used as failure criteria for the design of different layers of the pavement. This method tries to characterize the problems of all materials in terms of the dynamic modulus of elasticity. The dynamic modulus of elasticity, sometimes called resilient modulus, is a dynamic test response defined as the ratio of repeated axial deviator stress to the recoverable axial strain, E_a:

$$M_R = \frac{\sigma_1}{\epsilon_a} \quad .$$

18.4.7 *Criteria for Failure*

Rutting Criteria for Failure

Rutting is the permanent deformation resulting from traffic-associated distress. It is the phenomenon of longitudinal depressions in the wheel paths, resulting from compaction or lateral migration of one or more pavement layer materials under the action of traffic and environment. The equation developed by Finn et al. for failure by deformation of sub-grade is given by the Equation in Section 18.5.6.

Fatigue Criteria for Failure

Fatigue is defined as the phenomenon of load-induced cracking due to repeated stress at strain levels below the ultimate strength of the material. The strain at the bottom of the asphalt layer is the measure of cracking. The allowable strain is different for different quality surface layers. Several equations have been developed by various agencies to equate the failure to a given level of allowable strain. The fatigue model is given by the Equations in Section 18.5.7.

18.4.8 *Advantages and Disadvantages of Mechanistic Design*

Advantages of Mechanistic Design

With mechanistic design, it is possible to try several combinations of material and thickness for the various layers in order to come up with the most economical solution for the same strength.

Disadvantages of Mechanistic Design

The following are some problems associated with mechanistic design:

the assumption that pavement layers are homogeneous, isotropic, and elastic is not completely true,

- computer use is essential,
- the dynamic modulus of elasticity of *in situ* materials cannot be easily determined, and
- the dynamic modulus of elasticity for layers of granular material is dependent on the stress conditions of the layer and stress sensitivity relationships are needed to adjust the lab modulus to the actual conditions.

Several agencies have developed design charts based on mechanistic analysis and performance tests for their own conditions. Section 18.5.5 and 18.5.6 present examples of design by these methods. Section 18.5.7 is based on The CHEVPC computer programme.

Each user agency may develop their own failure equations and use them with the stresses or strains calculated by computer analysis in order to design pavement layers. The following discussions present the background of existing computer programmes for layered-elastic analysis. Section 18.5.7 presents examples of designs using computer analysis and given failure equations.

18.4.9 *Existing Computer Programmes for Layered-Elastic Analysis*

The following discussion presents an overview of existing computer programmes for multi-layered elastic analysis based on background material after NewComb, University of Washington, 1985.

Background

Much of what is currently used in the structural analysis of pavements is based on technology that has evolved over the last 60 years. The first use of the layered-elastic theory in pavement design occurred in 1926 when Westergarrd applied these principles to Portland Cement Concrete (PCC) pavements. Burmister later used the theory of elasticity as an approach to the solution of multi-layered, elastic pavement structures. In the development of his solution, Burmister assumed that each layer could be represented as a homogeneous, isotropic, and linear elastic material. Each layer was assumed to extend infinitely in the horizontal direction, and the bottom layer was assumed to extend infinitely downwards. The other layers were assumed to have finite thickness.

In pavement systems, loads generally occur over an elliptical area. The resulting vertical stresses are distributed in a bell-shaped fashion on the horizontal surfaces. The maximum stress is located on a vertical line to the midpoint of the load. Several influence charts and tables have been developed to determine the stresses, strains, and deflections in a one-layer system for any value of Poisson's Ratio.

Typical pavements are composed of different layers and material stiffness decreases with depth. The end result is the reduction of stresses, strains, and deflections in the sub-grade compared to the one layer case.

In two-layer systems, materials within a specific layer are assumed to be homogeneous, isotropic, and elastic. Furthermore, the layers are assumed to extend an infinite distance horizontally. The surface layer has a finite depth and the underlying layer is assumed to be semi-infinite in the vertical. For boundary and continuity considerations, there are no shearing and normal stresses outside the loaded area for the surface layer and the layers are assumed to be in continuous contact.

For a three-layer pavement system, several charts and tables have been developed by Peattie, Jones, and Fox (1962) to determine the stresses, strains, and deflections. Peattie developed graphical solutions for vertical stress in three-layer systems. Jones presented solutions for horizontal stresses in a tabular form. Both these solutions were based upon Poisson's Ratio of 0.5 for all layers.

The logical extension of these solutions was the development of computer programmes in order to expedite analysis and allow greater flexibility in the accommodation of material properties and multiple loads. Even the most elementary of these allow for materials with Poisson's Ratio other than 0.5. Some are capable of ascertaining the effects of multiple gear configurations and/or non-linear material behavior. Recently, Bush developed a computer programme which essentially works layered-elastic analysis in reverse to determine material properties from non-destructible deflection measurements.

Finite element analysis has been recently proposed to evaluate the response of pavement structures to loading. This method defines the pavement in terms of elements that are connected at nodal points. The stiffness at each nodal point is calculated by means of assuming displacement variation within the element along with a knowledge of the stress-strain behavior of the element material. Equilibrium at each nodal point may be expressed by two equations which are used to solve the unknown displacements. Once the displacements at all the nodal points have been calculated, the stresses and strains for each element may be computed.

Computer Programme Descriptions and Operating Notes

This section provides the user with a general overview of some computer programmes for pavement designs in the U.S.A. It contains information on the principles of operation, the assumptions associated with each programme, the characteristics and limitations, and specific warnings on possible pitfalls the user may encounter. The information presented here is intended to be supplemented by the user's manuals and other references that contain more detailed descriptions of the programme.

The layered-elastic system computer programme was developed at the University of California, Berkeley, and can be used to analyze up to ten identical loads on a five-layer system. The programme computes various components of stresses, strains, and displacements along with principal values in a three-dimensional, ideal layered-elastic system.

The top surface of the pavement is assumed to have no shear. As with other layered-elastic systems, the layers are assumed to have uniform thicknesses and to be infinite in horizontal distance. Layered interfaces are assumed to be continuous. A finite thickness may be used for the bottom layer or it may be assumed semi-infinite. If a finite thickness is used, the programme assigns a rigid underlying layer to support it and a continuous or frictionless interlayer must be assumed.

Input data for ELSYM5 are any two of three load determinants (load in pounds, stress in pounds per square inch, radius of load in inches), load position, elastic modulus, Poisson's Ratio, location of analysis points, and thickness of each layer (except the lowest). Coordinates for load positions and analysis points are expressed in terms of x and y for horizontal locations and z for depth.

Loads are assumed to be uniform, static, and circular, and the principle of superposition is used for determining the effect of multiple loads. Hicks et al. (1978) identified the following programme characteristics:

(a) one to five systems may be evaluated in a single run,
(b) one to five layers may be used in the systems,
(c) one to ten identical circular loads may be applied to the pavement,
(d) one to 100 locations may be specified for pavement response results (stress, strain, deflection),
(e) no depth may be specified for pavement response results if the point is below the top of the rigid underlying layer,
(f) no negative data are allowed except for horizontal distances,
(g) Poisson's Ratio can be any value except one; for a sub-grade on rigid support, Poisson's Ratio must not be within the range of 0.748 to 0.752, and
(h) results are approximate at or near the pavement surface and at some horizontal distances from the load; this is due to a truncated series used in the integration process.

CHEVNL (CHEVPC)

This programme presents solutions for multi-layered elastic systems. The original version of this computer programme was developed by the Chevron Research Company (formerly California Research Corporation) in the early 1960s. It computes stresses, strains, and deflections as a result of a single, uniform, circular load applied vertically to the pavement. This system is capable of analyzing up to 15 layers. All layers are of finite thickness, except the bottom which is of semi-infinite thickness. The horizontal dimension is infinite for all layers. The surface of the pavement is assumed to have no shear forces acting upon it.

Radial and vertical distances are expressed in cylindrical coordinates as R and Z respectively. The z-axis at $R=0$ extends through the centre of the load.

The vertical load and contact pressure are used to describe the problem loading conditions. Using these parameters, the programme computes the load radius. Material properties of individual layers are expressed in terms of the modulus of elasticity (resilient modulus), Poisson's Ratio, and thickness.

At least 30 radial and 30 vertical points may be selected for analysis. In addition to typical strain values, the programme identifies and describes the maximum, principal tensile strain with respect to its angle from the radial axis. The following programme operating characteristics have been identified:

(a) up to 15 layers may be incorporated in the programme,
(b) the materials may be assigned any values of moduli,
(c) Poisson's Ratio may be any value other than one,
(d) the mathematics are relatively easy and self-contained, and
(e) the effects of multiple gears must be computed outside of the programme, using superposition.

BISAR

This computer programme uses the layered-elastic theory to solve stresses, strains, and displacements in pavement systems with one or more uniform circular loads applied vertically on the surface. BISAR has the additional capability of considering surface loads to be combinations of vertical, normal, and unidirectional horizontal forces. The usual layered-elastic assumptions apply in this programme except for continuity. Layer interfaces are assumed to be either in full contact or frictionless.

Stresses, strains, and displacements, due to each load, are accumulated separately in a cylindrical coordinate system. In multiple load problems, the cylindrical coordinate system is transformed to Cartesian. The effects of multiple loads are computed by summing the effects of each individual load. Specific output parameters must be designated in the programme for locations and components.

Inputs for the layers include the modulus of elasticity, Poisson's Ratio, thickness, and boundary conditions (rough or frictionless). Particular care should be used in selecting the desired parameters for output. These must be consistent with the coordinate system established by the programmer.

Some of the characteristics of BISAR include:

(a) a maximum of 10 layers may be used,
(b) up to 99 systems may be evaluated in one run,
(c) up to 99 points within a system may be specified for evaluation,
(d) no negative data may be used as input except for horizontal distances, and
(e) there are no provisions for non-linear behaviour in the materials.

PSAD2A

This program is a multi-layered elastic system which may be used to determine stresses and strains while allowing material moduli to vary with stress levels. It also automatically computes stresses and strains caused by dual wheel configurations. The major advantage of PSAD2A is the estimation of the sub-grade modulus from the modulus-deviator stress relationship which is used as input. This relationship may be described by the equation (see Figures 18.1 and 18.2):

$$M_R = K_1 \sigma_d^{k_2}$$

where,

M_R	=	resilient modulus (psi) of sub-grade
σ_d	=	deviator stress (psi), see Figure 18.3, and
K_1, k_2	=	material constants.

Base or sub-base course stress sensitive materials are characterized by modulus-bulk stress relationships as defined (see Figure 18.2).

$$M_R = K_1 \theta^{k_2}$$

where,

M_R	=	resilient modulus (psi),
Θ	=	bulk stress (psi), see Figure 18.3, and
K_1, k_2	=	regression constants.

For non-stress dependent materials, a horizontal relationship is used as input. Another advantage of PSAD2A is that overburden pressures (stresses) may be incorporated into the solution by superimposing load-induced and overburden stress.

The iterative process of this programme compares the stress state in the material with the initially assumed modulus. This is repeated until the stress state and modulus value are reconciled to specified accuracy limits. It should be noted that the first set of stresses, displacements, and strains in the output are for a single load. The following output pages list stresses and strains due to dual loads.

Particular characteristics of this programme have been identified as follows:

(a) five layers must be used in the analysis,
(b) output values may be obtained for 48 to 121 points in the pavement,
(c) no negative data may be used as input,
(d) Poisson's Ratio may be any value except one, and
(e) three to 20 modulus-deviator stress points may be used as input.

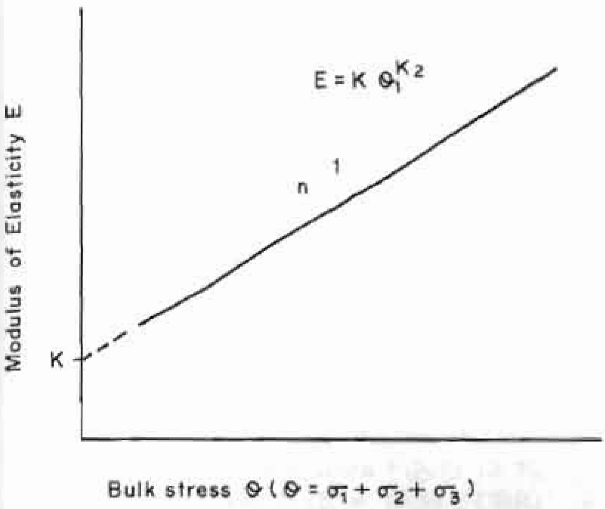
BISDEF

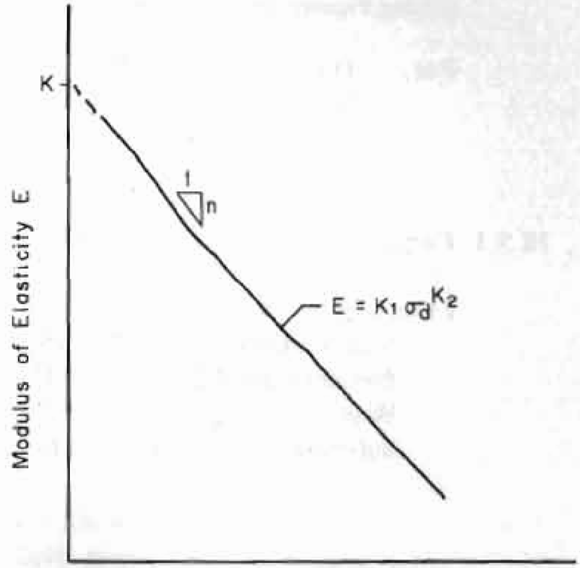
As mentioned earlier, this programme provides a means for predicting the moduli of up to four layers from non-destructive deflection data. It does this by iteratively matching deflection values with material properties using the BISAR layered-elastic programme. There may be a maximum of four deflection measurements and one load used for input. Deflection points are defined in terms of x and y coordinates as well as depth. The load is defined in terms of its centre x and y coordinates, vertical stress, and radius.

A certain amount of judgment must be used when considering input values for initial material properties. Since the programme iteratively matches deflection values with layer moduli, a tolerance must be specified for stopping the programme. Ten per cent is recommended for this value. Also, a maximum number of iterations (usually three) must be specified to stop the programme to prevent the use of an excessive amount of computer time.

A minimum and maximum allowable modulus must be specified for each material of unknown modulus. Boundary conditions must be set as either rough or frictionless. An initial estimate of the modulus, Poisson's Ratio, and the thickness of all layers except the sub-grade must be input to the programme . The closer the initial modulus estimate is to the actual value, the faster the programme will close and the less costly the run. Bush (1980) recommends that the modulus of asphalt concrete be determined by first estimating the temperature at mid-depth. Furthermore, he suggests a range between 600,000 and 1,500,000 psi as a modulus range for cement-stabilized materials.

BISDEF uses a free-format input. If the programme has not closed within the specified tolerance in the allotted number of iterations, the programme will notify the user. If this happens, the user is advised to adjust the modulus input values and rerun the programme.

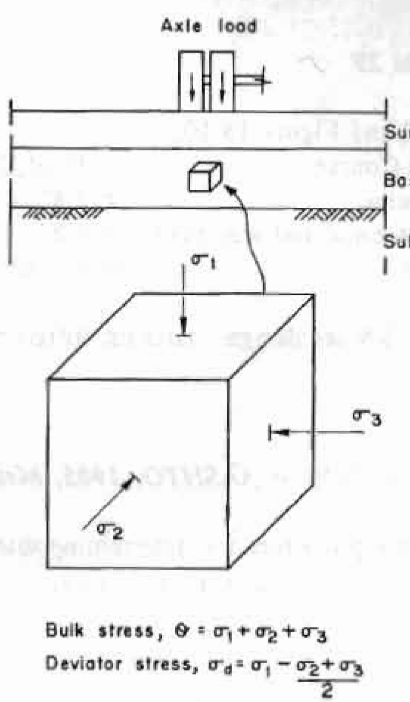




Source: University of Washington 1986

Fig. 18.1 Modulus-deviator stress relationship for fine-grained materials

Fig. 18.2 Modulus-bulk stress relationship for coarse-grained materials



Source: University of Washington 1986

Figure 18.3 Principal stresses due to axle load

18.5 EXAMPLE OF NEW PAVEMENT DESIGN BY DIFFERENT METHODS

(Using Figures 18.4 to 18.24 and Tables 18.2 to 18.13a)

For, Traffic = 2x106 ESA
Sub-grade CBR = 7%

18.5.1 *Corps of Engineers CBR Method*

From Figure 18.4,

total thickness	=	16"
therefore use AC	=	3"
Base	=	7"
Sub-base	=	6"

18.5.2 *TRRL Road Note 31*

From Figure 18.5,

Bituminous Premix	= 150 mm	= 2"
Base	= 150mm	= 6"
Sub-base	= 158mm	= 6"

18.5.3 *TRRL Road Note 29*

From Table 18.3 and Figure 18.10,

Wearing Course	= 1" of DBM
Base Course	= 2.4" of DBM
Base (untreated and wet mix)	= 6.8"
Sub-base	= 6" (minimum thickness from Figure 18.6)

Figures 18.7 to 18.9 are design charts for different surfacing and bases.

18.5.4 *Structural Number (SN) = AASHTO, 1985, Method*

Figure 18.11 illustrates the procedure for determining thicknesses

(i) Full depth AC,

a) assume, E_{ac}	= 500,000 psi, (E_{ac} = 500,00 psi at 82°F from Fig. 18.13)
reliability	= 85%
standard deviation	= 0.45, and
serviceability loss	= 3.00.

from Figure 18.12,

$SN_1 = 3.5$

therefore, $D_1 = SN_1/a_1 = 3.5/0.46 = 7.6"$. ($a_1 = 0.46$ from Fig. 18.14)

b) assume for emulsified AC Type II of Asphalt Institute,

E_{ac} = 2,000,000 psi,
a_1 = 0.29, and
$D_1 = SN_1/a_1 = 3.5/0.29 = 12"$.

Assume for 4 layers,

E_{ac}	=	500,000 psi,
E_{base} for CBR 70, from Figure 18.15	=	28000 psi,
$E_{Sub\text{-}base}$ for CBR 20, from Figure 18.16	=	127000 psi, and
$E_{sub\text{-}grade}$ for CBR 7, (E = 1500 x CBR)	=	10500 psi.

From Figure 18.12,

$SN_1 = 2.3$,
$SN_2 = 3.2$, and
$SN_3 = 3.5$.

From Figure 18.14, $a_1 = 0.46$,
from Figure 18.15, $a_2 = 0.13$, and
from Figure 18.16, $a_3 = 0.07$,

therefore, $D_1 = SN_1 / a_1 = 2.3/0.46 = 5"$,
$D_2 = SN_2\text{-}SN_1/a_2 = 3.2 - 2.3 / 0.13 = 6.92 = 7"$,

therefore, $SN_2 = 7 \times 0.13 = 0.91$,

$$D_3 = [SN_3\text{-}(SN_1+SN_2)]/a_3$$
$$= [3.5\text{-}(2.3 + 0.9)]/0.07 = 4.14 = 4.2" .$$

Adopt, D_1 = 5",
D_2 = 7", and
D_3 = 4.2" .

Figure 18.17 illustrates the sub-grade modulus relationship for various pressures of sub-grade strength.

(iii) Assume for 4 layers using emulsified AC Type II,

E_{ac} = 200,000 psi,
E_B = 28,000 psi, E_{sb} = 127000 psi, Esgri = 10500 psi,

SN_1 = 2.3, SN_2 = 3.2, SN_3 = 3.5,

a_1, from Figure 18.14 = 0.29,
a_2 = 0.13, and
a_3 = 0.07.

$D_1 = SN_1/a_1 = 2.3/0.29 = 7.93 = 8"$
therefore, $SN_1 = 8 \times 0.29 = 2.32$.
$D_2 = (SN_2 - SN_1)/a_2 = (3.2-2.32)/0.13 = 0.88/0.13 = 6.77 = 7"$
therefore, $SN_2 = 7 \times 0.13 - 0.91$
therefore, $D_3 = \{SN_3 - (SN_1 + SN_2)\}/a_3 = 4.14 = 4.2"$
therefore,
D_1 = 8",
D_2 = 7", and
D_3 = 4.2".

Table 18.4 gives values of coefficient m to account for drainage conditions in pavement layers. The equations in Figure 18.11 account for drainage in base by using the factor m_2 and in the sub-base by using factor m_3. Tables 18.5 to 18.7 give suggested reliability levels, minimum thickness of pavement layers, and analysis period for different traffic levels.

18.5.5 *Asphalt Institute Method*

MR = 1500 CBR = 1500 x 7 = 10500 psi

i) From Figure 18.18,
full depth AC = 9.63" .

ii) From Figure 18.19, full depth Type II emulsified AC = 11,81"
(emulsified asphalt mix made with semi-processed
crusher, pit run, or bank aggregates).

iii) From Figure 18.20,
AC = 210 mm = 8.3", and
base (untreated) agg = 150mm = 6"
(ASTM D 2940).

iii) From Figure 18.20,
AC = 210 mm = 8.3", and
base (untreated) agg = 150mm = 6"
(ASTM D 2940).

18.5.6 *TRRL Laboratory Report 1132*

From Figure 18.21,

HRA = 115mm = 4.5" for sub-grade CBR of 5%,
base = 222mm = 8.75", and
sub-base = 225mm = 9" .

18.5.7 *Mechanistic - Empirical Design Using CHEVPC Computer Programme*

In this method, the load capacity of the pavement is determined by fatigue and rutting criteria. The Finn et al. (1977) model for 10 per cent and 45 per cent cracking for fatigue and for 0.75 in deformation of sub-grade has been used in this example. The relationship is given by:

$\log N_f \;\; 10\% = 15.947 - 3.291 \log \epsilon_t/10^{-6} - 0.854 \log MR/10^3$,
$\log N_f < 45\% = 15.986 - 3.291 \log \epsilon_t/10^{-6} - 0.854 \log MR/10^3$,

N_r = 1.077 x 1018 (1/106 x ϵvs)4.4843,
N_f = load in ESA to failure by fatigue of surface layer,
N_r = load in ESA to failure by rutting of sub-grade,
ϵ_t = tensile strain at the bottom of AC,
ϵ_{vs} = vertical compressive strain at top of sub-grade, and
MR = resilient modulus of AC.

The strains are calculated by using CHEVPC. The stress sensitivity of untreated aggregates is not accounted for here. Figure 18.22 and Table 18.8 give examples of calculation of stresses and strains using CHEVPC. The thickness for the desired load (traffic) is calculated by iterative process for trial runs for different layers. Table 18.2 compares load to failure for different combinations of thickness and materials.

18.5.8 *R-value Method*

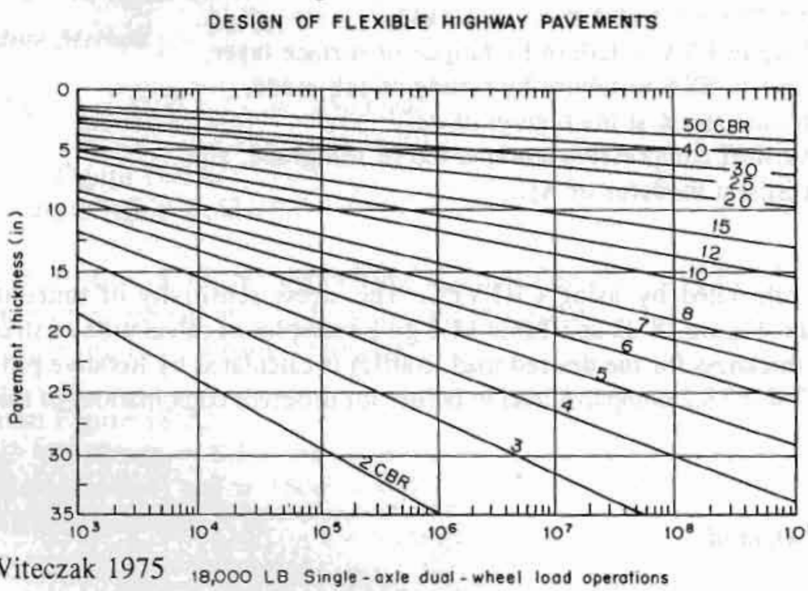
Figure 18.23 is a design chart for design by the R-value method. Figure 18.24 and Tables 18.9 and 18.10 are examples of design guide for treated and untreated bases.

Table 18.2 Comparison of new pavement design by different methods

No. of layers	Properties of material in psi	Thickness of Layers: CBR Method			Mechanistic-Empirical Based Methods			
		U.S. Corps of Engrs	TRRL RN 29	TRRL RN 31	SN Method	Asphalt Institute	TRRL LR 1132	CHEVPC
2.	E_{AC} = 5000,000 E_{sgr} = 10,000	-------		-------	AC = 9.6	AC = 9.6	-----	AC=8
2.	E_{ac} = 200,000 E_{sgr} = 10,500	-------		-------	AC = 12	AC = 11.8	------	AC=11.5
4.	E_{ac} = 5000,000 E_b = 28,000 E_{sb} = 12,700 E_{sgr} = 10,500	AC = 3" Base = 7" Sbase= 6"	DMB =3.5 Base= 7" Sbase= 6"	"Premix=2" Base =6" Sbase =6" Sbase=4.2"	AC =5" Base =6" Base =7"	AC =8.3" Base=6"	DBM =4.5 Base =8.8" Sbase=9"	"Low fatigue life with untreated bases and sub-bases.

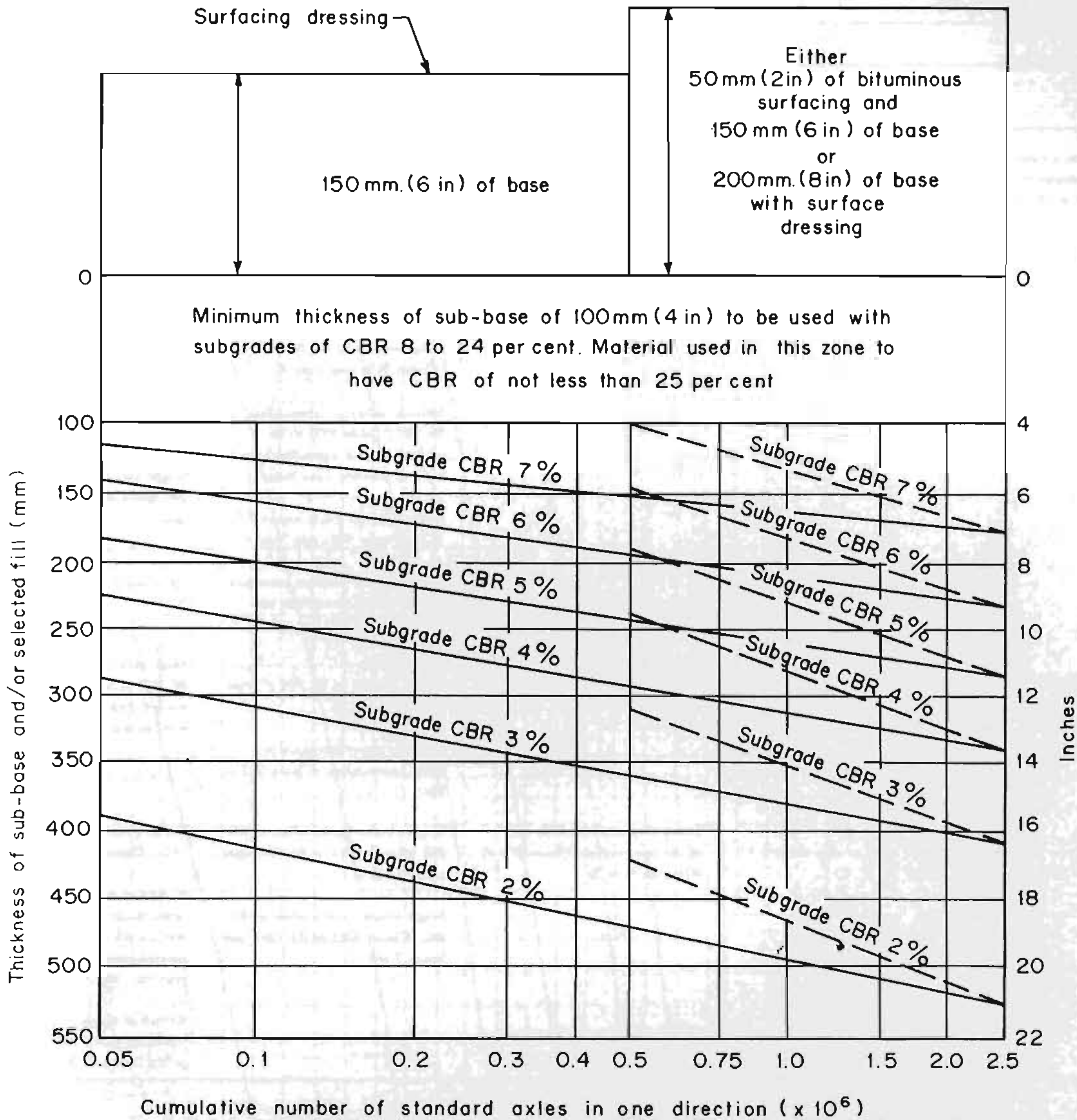
The design values from TRRL Lr 1132 are based on 5 per cent sub-grade CBR and may be lower for 7 per cent sub-grade CBR. The AASHTO Method uses the actual material properties as against standard materials adopted in other methods. The CHEVPC Method indicated that fatigue life is a problem with thinner surfacing or in pavements with untreated bases.

Thickness from variable methods may be converted to a desired combination of this surfacing and thicker based and sub-based by adopting equivalency factors, 1" AC =3.0 of untreated crushed stone base, and 1 untreated crushed stone base = 2 untreated, gravel sub-base. Also dense, graded asphalt concrete may be treated equal to 1.43" of emulsified asphalt concreter. It may be noted that the fatigue life of thin surfacing over untreated base will be lower than the design life of base and sub-base layers and periodic (every 5 years) renewal of the thin surfacing is essential.



Source: Yoder and Witeczak 1975

Figure 18.4 CBR design curves for 18,000 E.A.L.



Source: TRRL Road Note 31, 1971

If desired to provide at the time of construction a pavement capable of carrying more than 0.5 million standard axles, the designer may choose either a 150 mm (6m) base with a 50 mm (2 in) bituminous surfacing or a 200 mm (8 in) base with a double surface dressing. For both of these alternatives, the recommended sub-base thickness is indicated by the broken line.

Alternatively, a base 150 mm (6 in) thick with a double surface dressing may be laid initially and the thickness increased when 0.5 million standard axles have been carried. The extra thickness may consist of 50 mm (2 in) of bituminous surfacing or at least 75 mm (3 in) of crushed stone with a double surface dressing. The largest aggregate size in the crushed stone must not exceed 19 mm (3/4 in) and the old surface must be prepared by scarifying to a depth of 50 mm (2 in). For this stage of the construction procedure, the recommended thickness of sub-base is indicated by the solid line.

Fig. 18.5 Pavement design chart for flexible pavements

Table 18.3 Recommended bituminous surfacings for newly constructed flexible pavements

Traffic (cumulative number of standard axles)			
Over 11 million (1)	25-11 million (2)	0.5-2.5 million (3)	Less than 0.5 million (4)
Wearing course (crushed rock, or slag, coarse aggregate only) Minimum thickness 40 mm Rolled asphalt to BS 594 Rolled asphalt to BS 594 (pitch bitumen) may be used (clause 907)		Wearing course Minimum thickness 20 mm Minimum thickness (pitch-bitumen binder may be used) (Clause 907) Dense tar surfacing to BTIA Specification (Clause 909) Could asphalt to BS 1690 (Clause 910) (see Note 4) Medium-textured tarmacadam to BS 802 (Clause 913) to be surface-dressed immediately or as soon as possible - see Note 4) Dense bitumen macadam to BS 1621 (Clause 908) (see Note 4) Open-textured bitumen macadam to BS 1621 (Clause 912) (see Note 4)	Two course a) Wearing course - 20 mm cold asphalt to BS 1690 (Clause 910) (see Note 4), coated macadam to BS 802 BS 1621, BS 1241 or BS 2040 (Clause 913, 912 or 908) (see Notes 2 and 4) b) Base course, coated macadam to BS 802, BS 1621, BS 1241, or BS 2040 (Clause 906 or 905) (see Note 2) Single course rolled asphalt to BS 594 (pitch-bitumen binder may be used) Dense tar surfacing to BTIA specification (Clause 908) (see Note 4)
Base course Minimum thickness 60 mm	Base course Rolled asphalt to BS 594 (Clause 902) (see Note 2)	Base course Rolled asphalt to BS 594 (Clause 902) (see Note 2) Dense bitumen macadam per dense tarmacadam (Clause 903 or 904)	Medium textured tarmacadam to BS 802 (Clause 913) (to be surface-dressed immediately or as soon as possible - see Note 4)
Dense bitumen macadam or dense tarmacadam (crushed rock or slag only) (Clause 903 or 901)	Dense bitumen macadam or dense tarmacadam (Clause 903 to 904) (see Note 3)	Single course tarmacadam to BS 802 (Clause 906) or BS 1241 (see notes 2 and 5) Single course tarmacadam to BS 802 (Clause 906) or BS 1241 (see Notes 2 and 5)	Dense bitumen macadam to BS 1621 (Clause 908) (see Note 4) 60 mm of single coarse tarmacadam to BS 802 (Clause 906) or BS 1241 (to be surface-dressed immediately or as soon as possible - see Note 4) 60 mm of single, coarse bitumen macadam to BS 1621 (Clause 905) or BS 2040 (see Note 4)

Source: TRRL Road Note 29 (Note 1), 1970

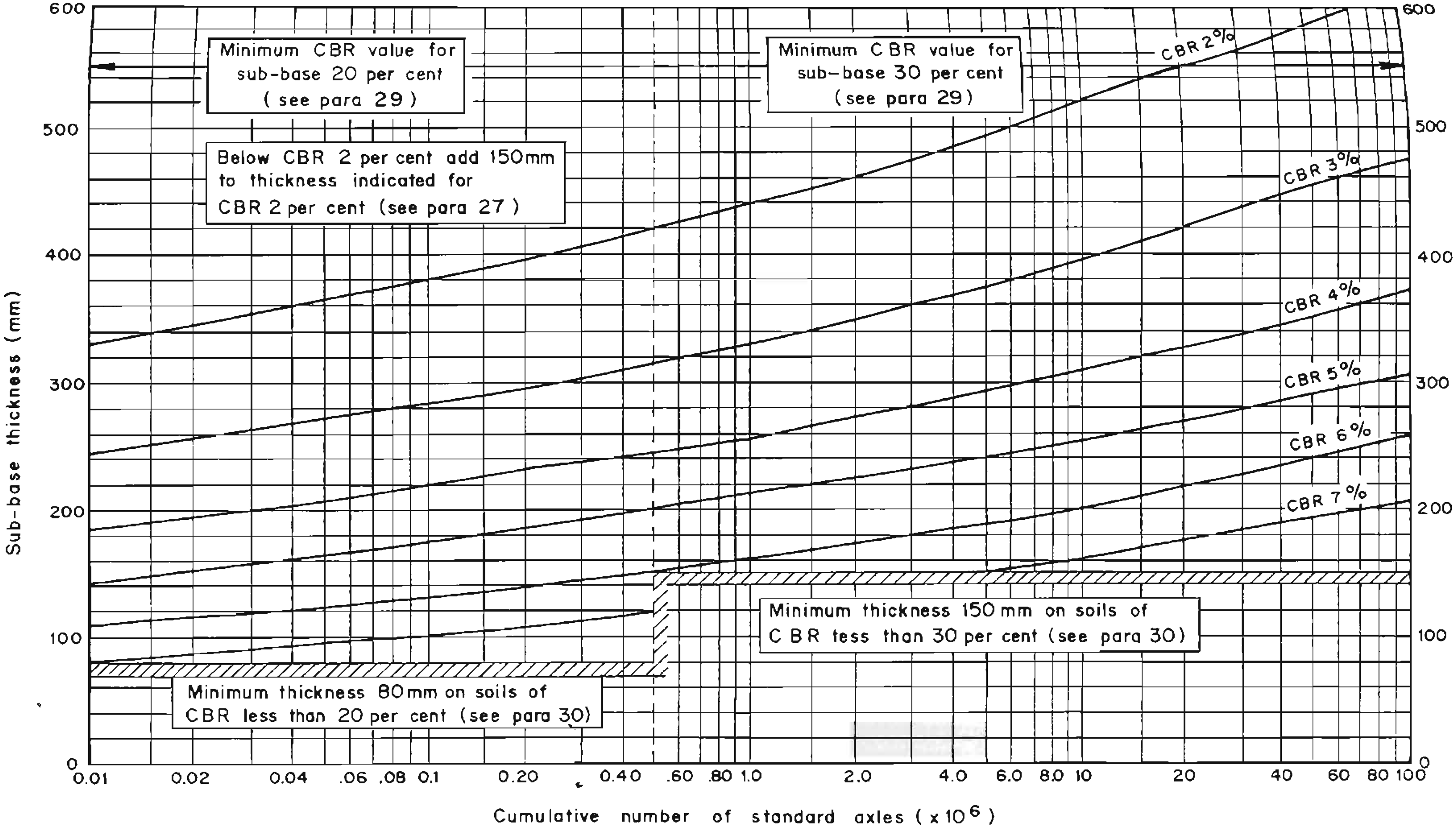
1. The thickness of all layers of bituminous surfacing should be consistent with appropriate British Specification.

2. When gravel other than limestone is used, 2 per cent of Portland should be added to the mix and the percentage of fine aggregate reduced accordingly.

3. Gravel tarmacadam is not recommended as base course for roads designed to carry more than 2.5 million standard axles.

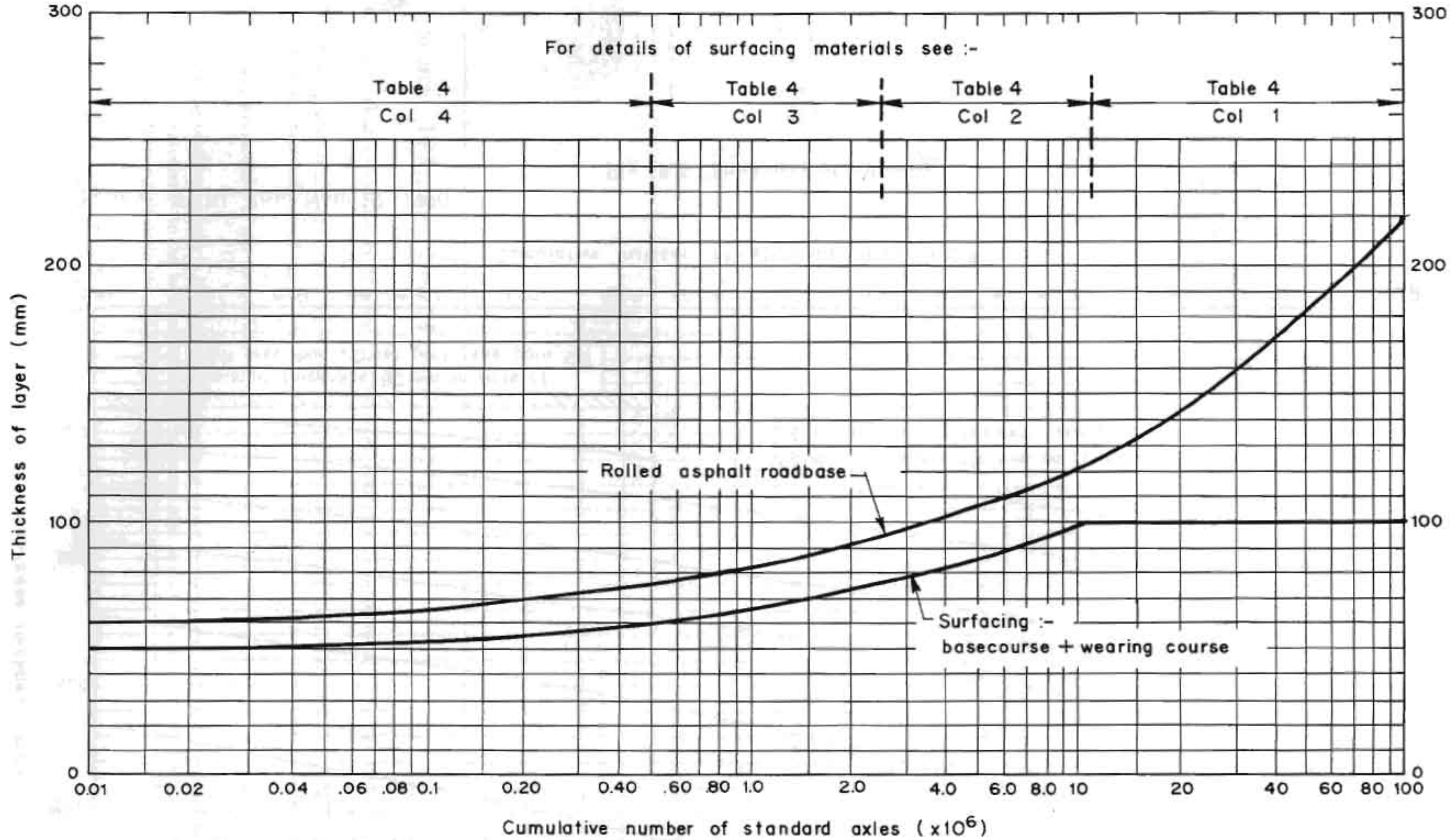
4. When the wearing course is neither rolled asphalt nor dense tar surfacing, and where it is not intended to apply a surface dressing immediately to the wearing course, it is essential to seal the construction against the ingress of water by applying a surface dressing either to the road base or to the base course.

5. Under a wearing course of rolled asphalt or dense tar surfacing, the base course should consist of rolled asphalt to BS 594 (Clause 902) or of dense coated macadam(Clause 903 or 904).



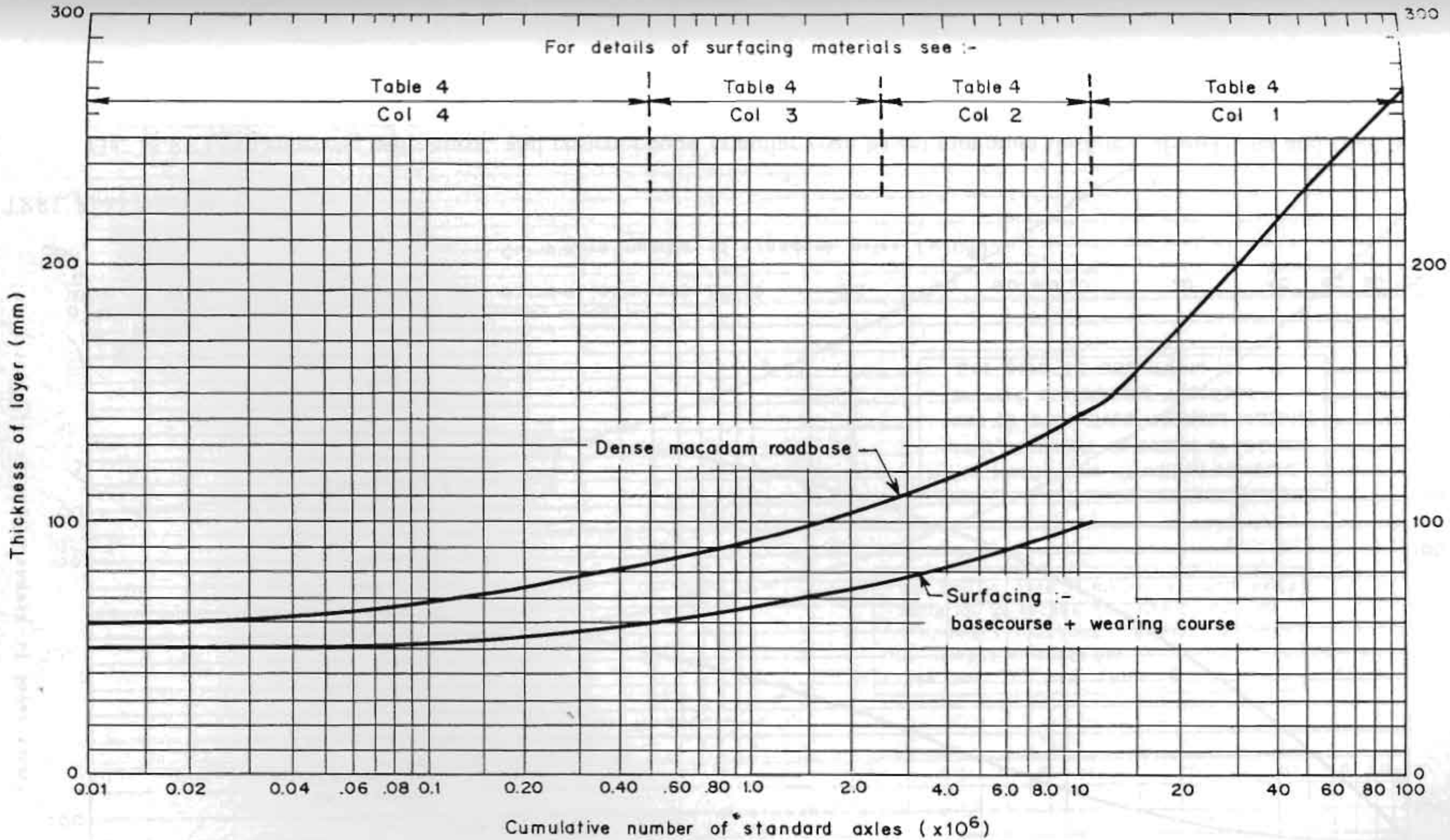
Source: TRRL Road Note 29, 1970

Fig 18.6 Thickness of sub-base



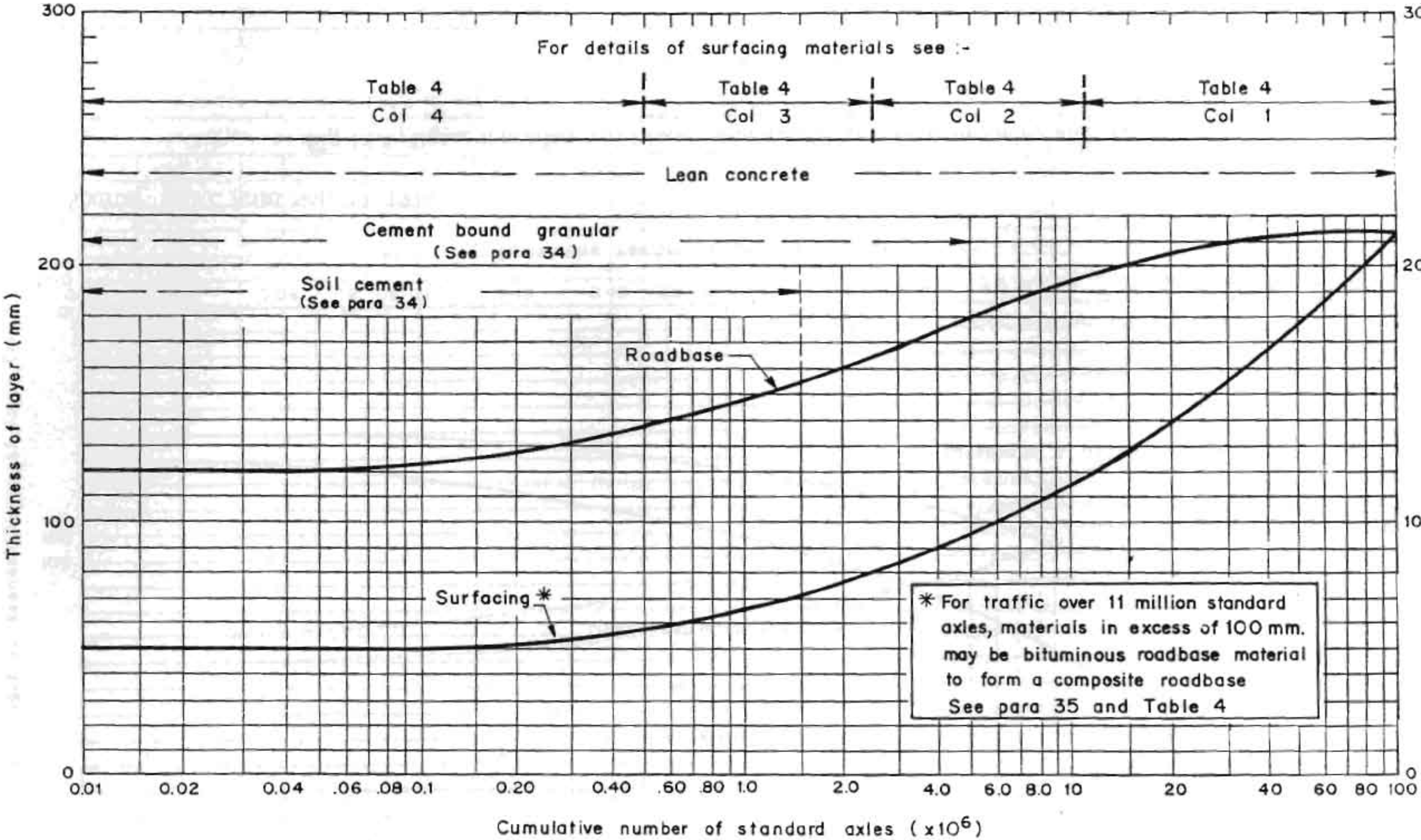
Source: TRRL Road Note 29, 1970

Fig 18.7 Rolled asphalt road base: minimum thickness of surfacing and road base



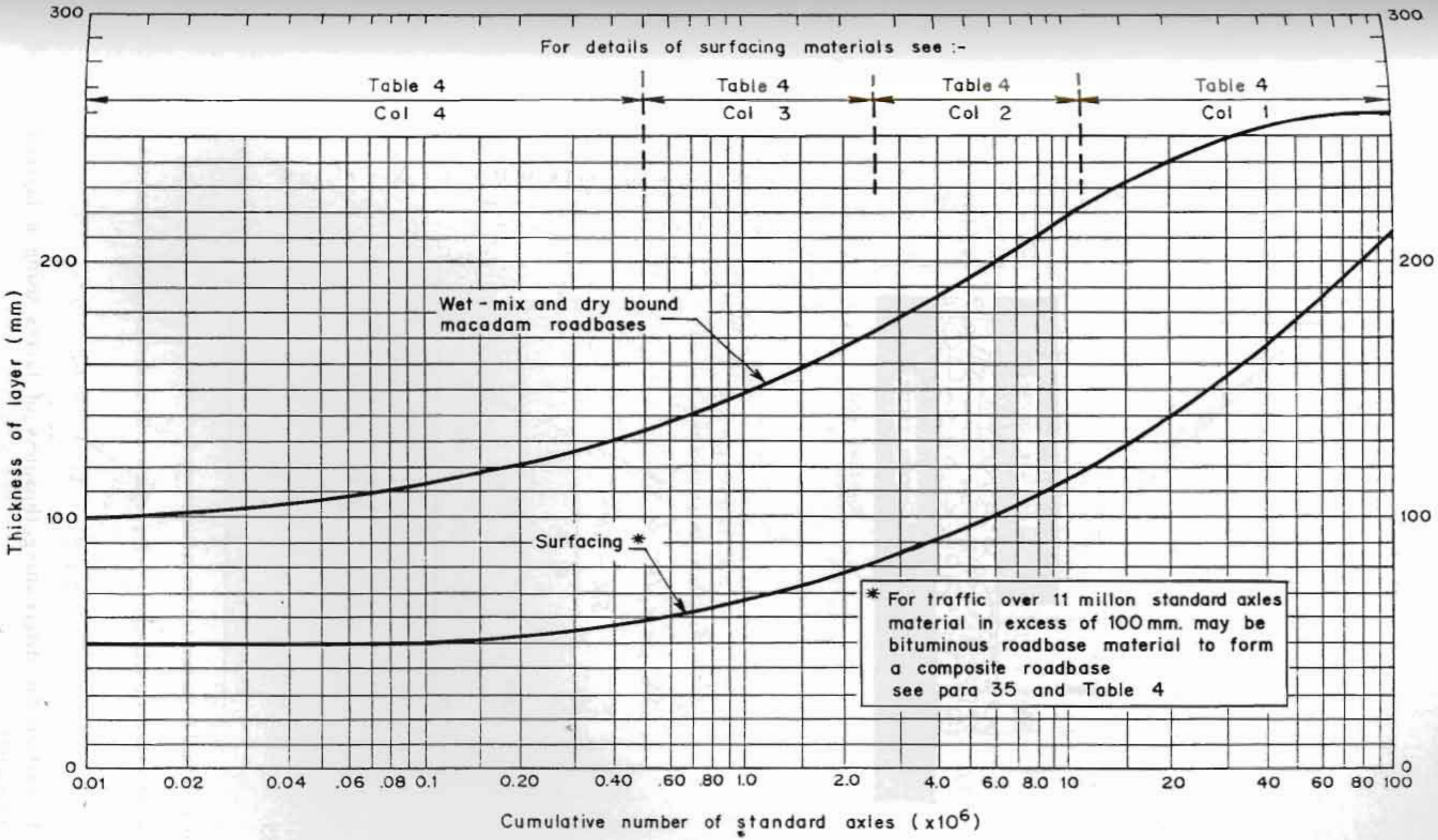
Source: TRRL Road Note 29, 1970

Fig 18.8 Dense macadam road base: minimum thickness of surfacing and road base



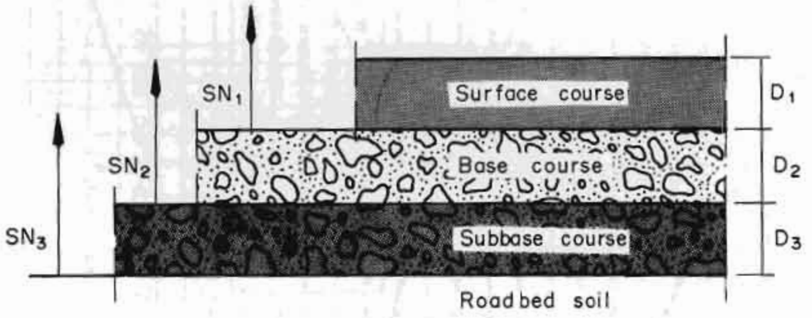
Source: TRRL Road Note 29, 1970

Fig. 18.9 Lean concrete, soil cement, and cement-bound granular road bases: minimum thickness of surfacing and road base



Source: TRRL Road Note 29, 1970

Fig. 18.10 Wet-mix and dry-bound macadam road base: minimum thickness of surfacing and road base



$$D^*_1 \geq SN_1 a_1$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN^*_1}{a_2 \, m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (Sn^*_1 + SN^*_2)}{a_3 \, m_3}$$

Source: AASHTO 1985

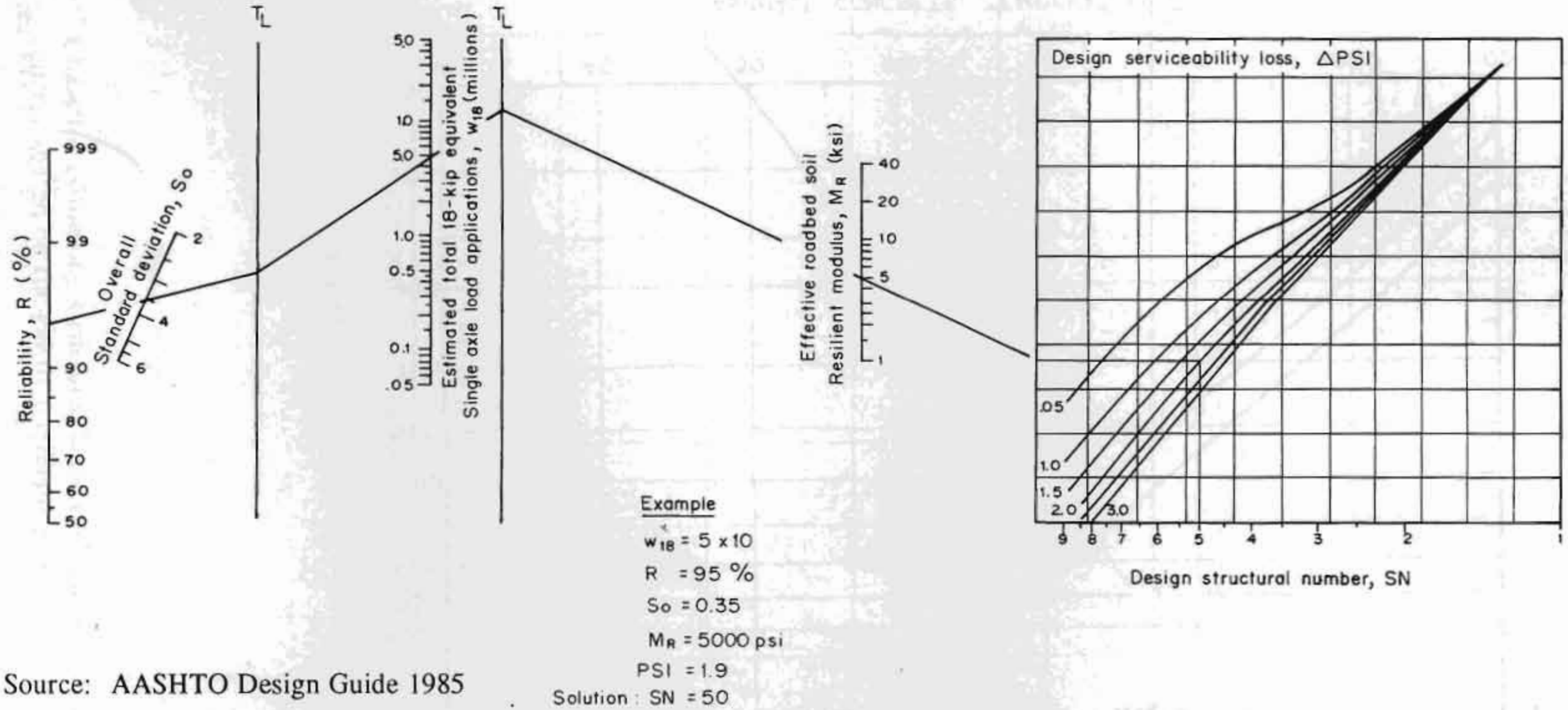
1) a, D, m and SN are as defined in the text and are minimum required values.

2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

Fig. 18.11 Procedure for determining thickness of layers using a layered analysis approach

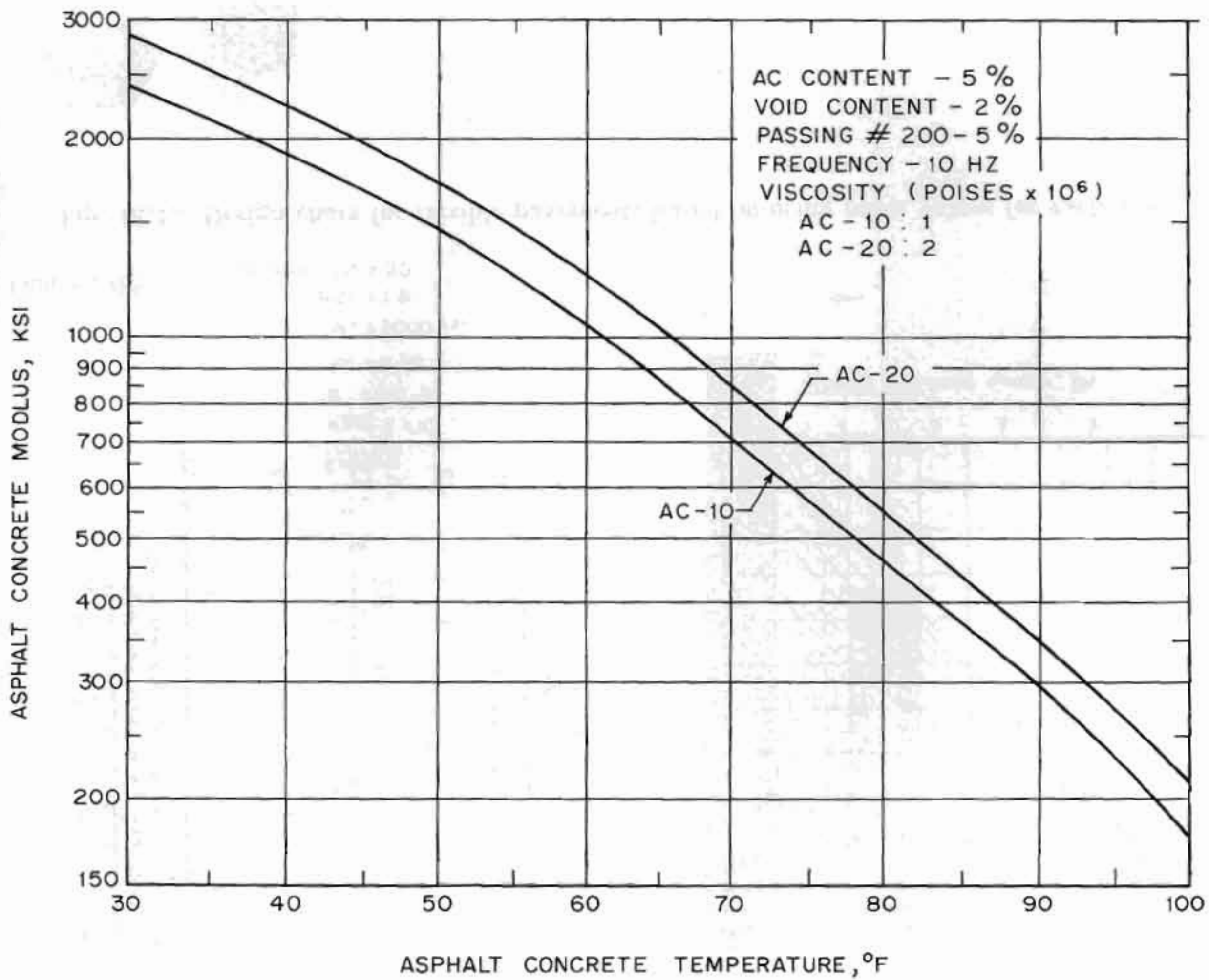
NOMOGRAPH SOLVES:

$$\log_{10} 18 = Z_R * S_o + 9.36 * \log_{10}(SN + 1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{SN+1^{5.19}}} + 2.32 * \log_{10} M_R - 8.07$$



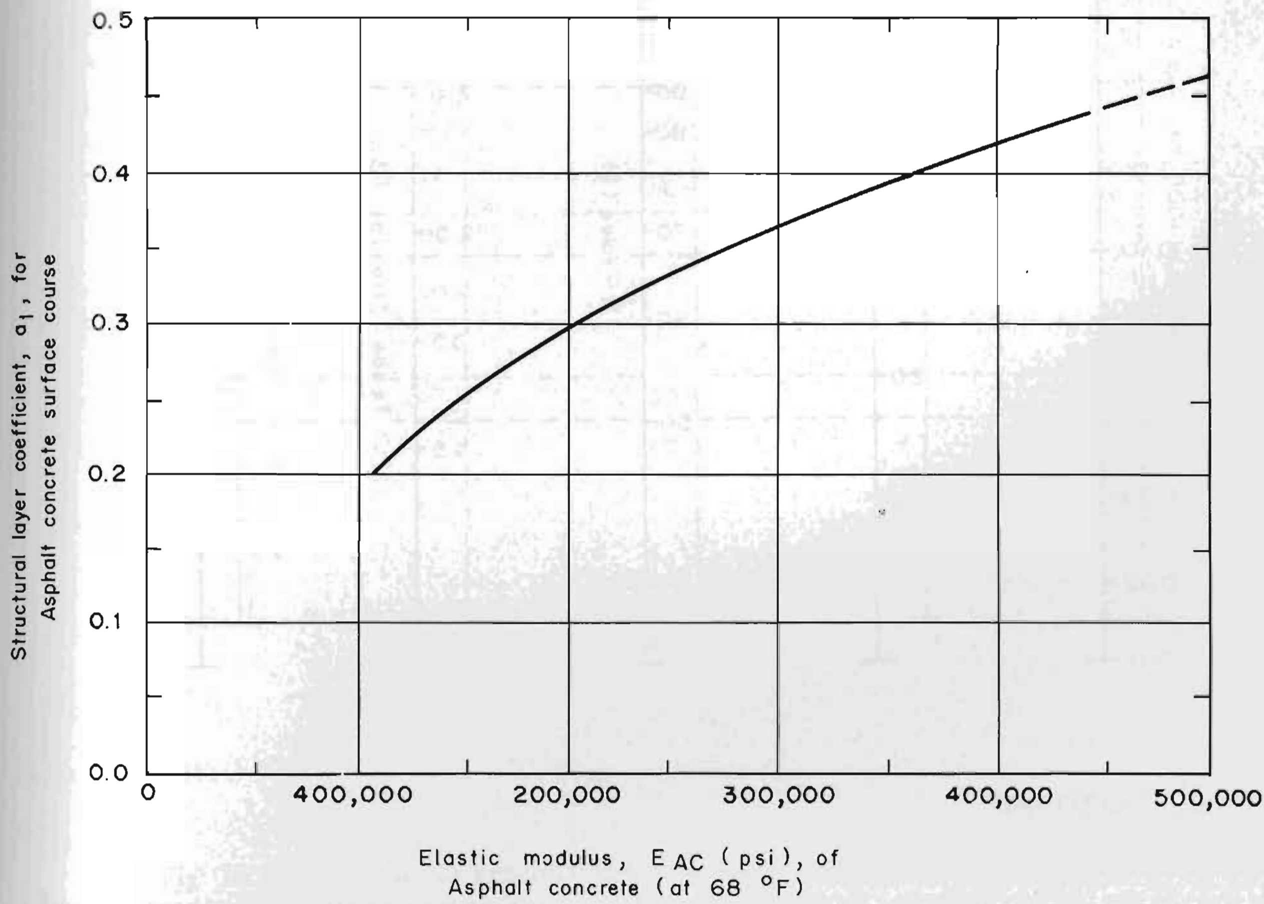
Source: AASHTO Design Guide 1985

Fig. 18.12 Design chart for flexible pavements based on using mean values for each input



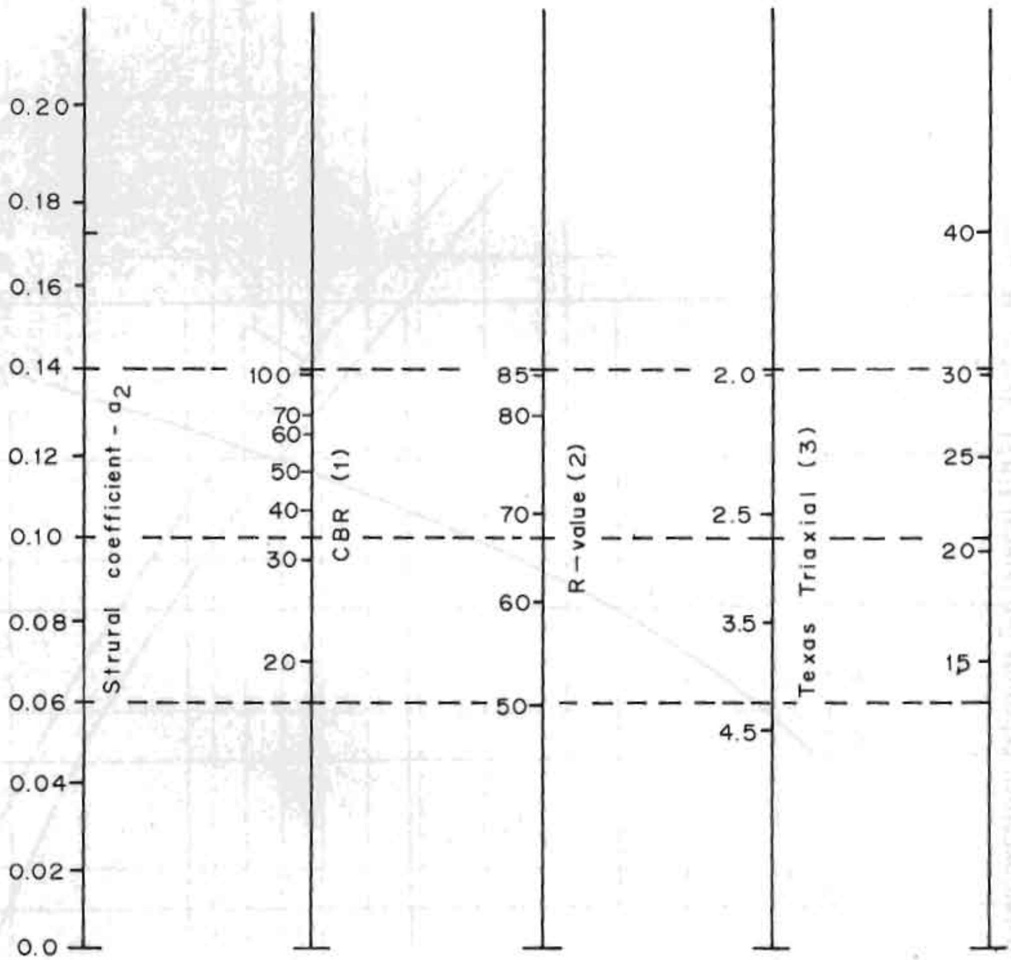
Source: Thompson and Cation 1986

Fig 18.13 Asphalt concrete modulus-temperature relations for typical IDOT (Illinois Department of Transportation) Class I



Source : AASHTO 1985

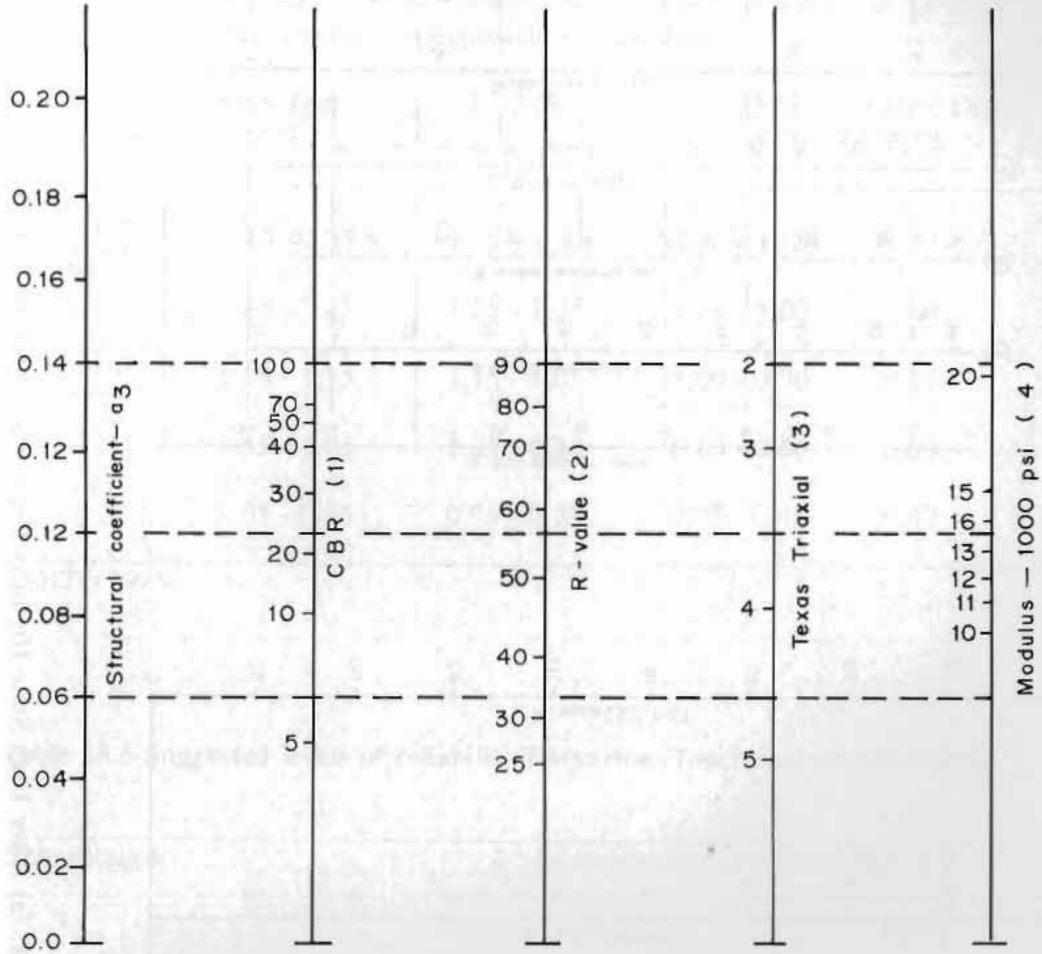
Fig. 18.14 Chart for estimating structural layer coefficient of dense-graded asphalt concrete based on the elastic (resilient) modulus



Source: AASHTO 1985

Fig. 18.15 Variation in granular base layer coefficient (a_2) with various base strength parameters

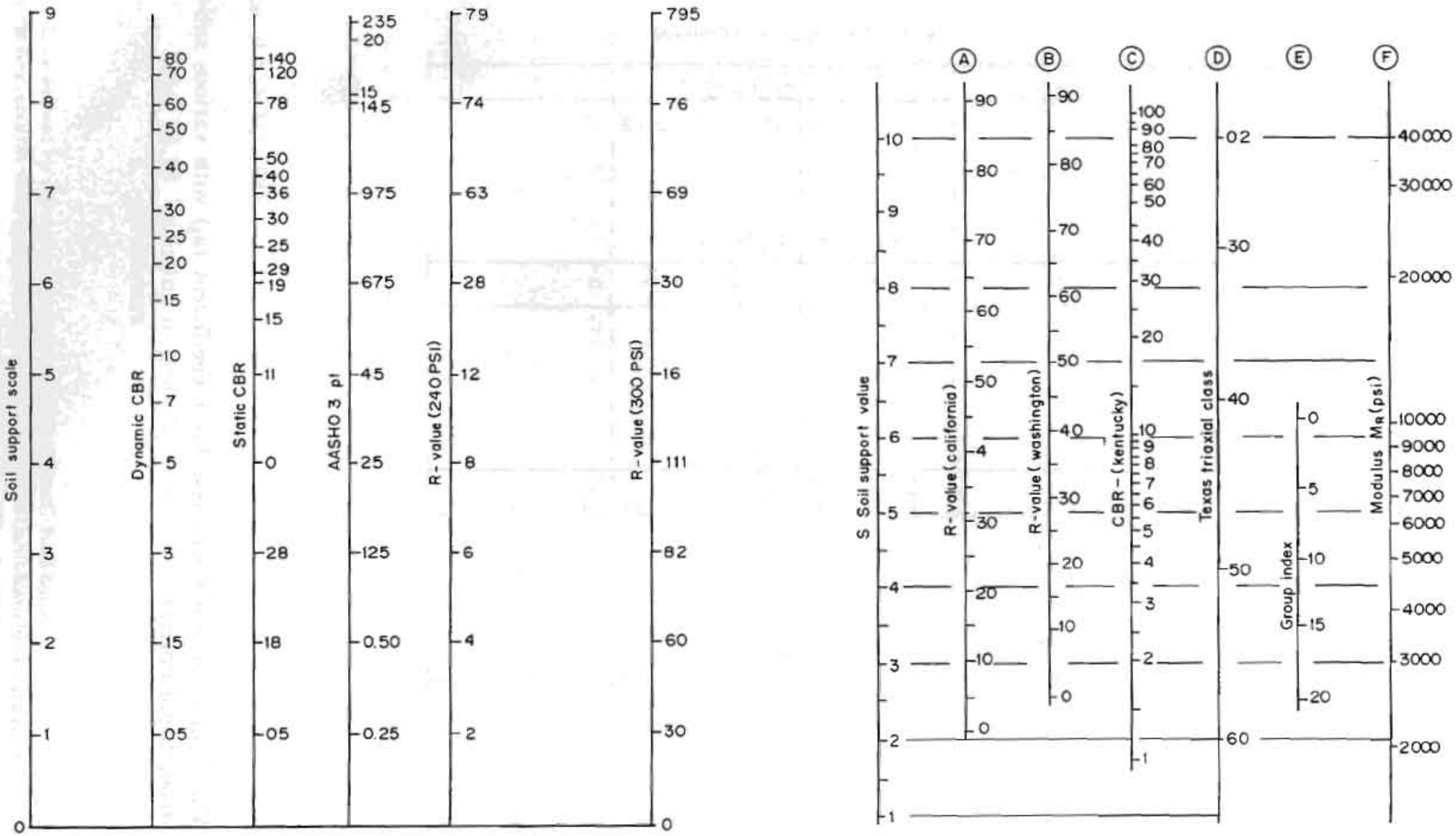
(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico, and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on National Cooperative Highway Research Programme (NCHRP)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



Source: AASHTO 1985

Fig. 18.16 Variation in granular sub-base layer coefficient (a_3) with various sub-base strength parameters

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



Source: Yoder and Witeczak 1975

Fig. 18.17 Soil support value correlations. (a) After Utah State Highway Department and (b) from Van et al. (NCHRP 128)

Table 18.4 Recommended m_i values for modifying structural layer coefficient of untreated base and sub-base materials in flexible pavements

Quality of Drainage	Per cent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1 %	1 - 5 %	5 - 25 %	Greater Than 25 %
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1. 20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Source : AASHTO 1985

Table 18.5 Suggested levels of reliability for various functional classifications

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and other freeways	85 - 99.9	80 - 99.9
Principle arterials	80 - 99	75 -95
Collectors	80 - 95	75 - 95
Local	50 - 80	50 - 80

Source: AASHTO 1985

Note: Results based on a survey of the AASHTO Pavement Design Task Force

Table 18.6 Minimum thickness (inches)

Traffic, ESAL'S	Asphalt Concrete	Aggregate Base
less than 50,000	1.0 (or surface treatment)	4
50,001 - 150,000	2.0	4
150,001 - 500,000	2.5	4
500,001 - 2,000,000	3.0	6
2,000,001 - 7,000,000	3.5	6
greater than 7,000,000	4.0	6

Source : AASHTO 1985

Table 18.7 Analysis periods for pavement design

Highway Conditions	Analysis Period (years)
High volume, urban	30 - 50
High volume, rural	20 - 50
Low volume, paved	15 - 25
Low volume, aggregate surface	10 - 20

Table 18.8 Design of pavement by mechanistic - empirical method using CHEVPC computer programme

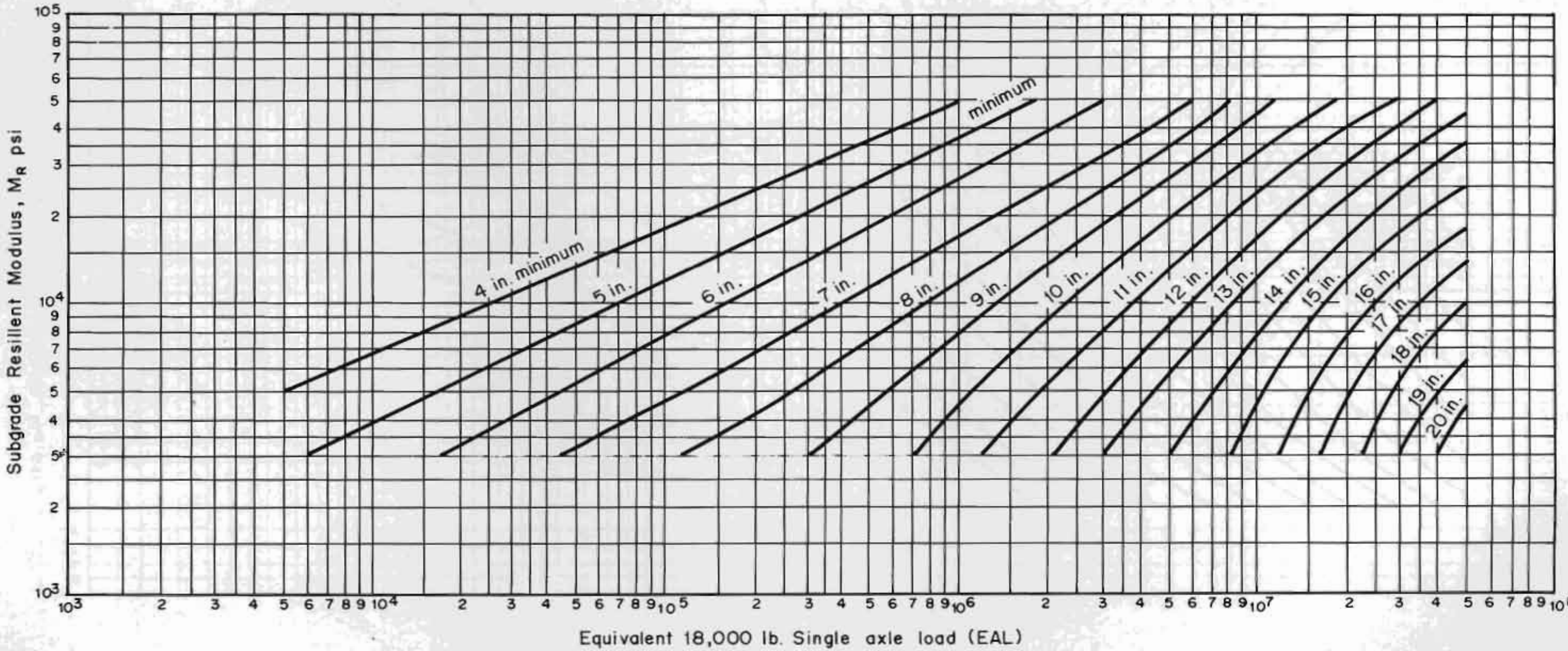
Eac	Thickness of Layers	ϵ_t	ϵ_{vs}	Fatigue Life Finn model A.C.		Rutting Life Finn Model
psi	inches			10% crack	45% crack	
				Nf	Nf	Nr
500000	8.50	157.36	343.6	2.58E+6	2.83E+6	4.57E+6
500000	7.50	187.80	405.80	1.45E+6	159E+6	2.17E+6
200000	11	1.87	439.40	2.15E+6	2.35E+6	1.52E+6
200000	12	184.70	389.08	3.33E+6	3.65E+6	2.62E+6
500000	5,7,10	326.60	276.80	2.34E+6	2.56E+5	1.20E+6
500000	5,10,10	226.89	274.50	7.76E+5	8.49E+5	1.25E+7
500000	5,15	221.40	315.04	8.40E+5	9.19E+5	4.15E+6
500000	5,15,10	219.40	208.56	8.65E+5	9.46E+6	4.29E+7
200000	8,7,10	237.70	271.80	1.45E+6	1.59E+6	1.31E+7
200000	8,10,10	228.70	230.10	1.65E+6	1.81E+6	2.76E+7
200000	8,15	222.80	292.58	1.80E+6	1.97E+6	9.39E+6
200000	8,15,10	220.78	177.50	1.85E+6	2.03E+6	8.83E+7

Failure Criteria:

Finn,
Fatigue:
$\leq$ 10% log Nf $= 15.947 - 3.291 \times \log \epsilon_t/10^{-6} - .854 \times \log MR/1000$
$\leq$ 45% log Nf $= 15.986 - 3.291 \times \log \epsilon_t/10^{-6} - .854 \times \log MR/1000$

Rutting:
$Nr = 1.07 \times 10^{18} \times (1/\epsilon_{vs})^{4.483}$

Full-depth asphalt concrete

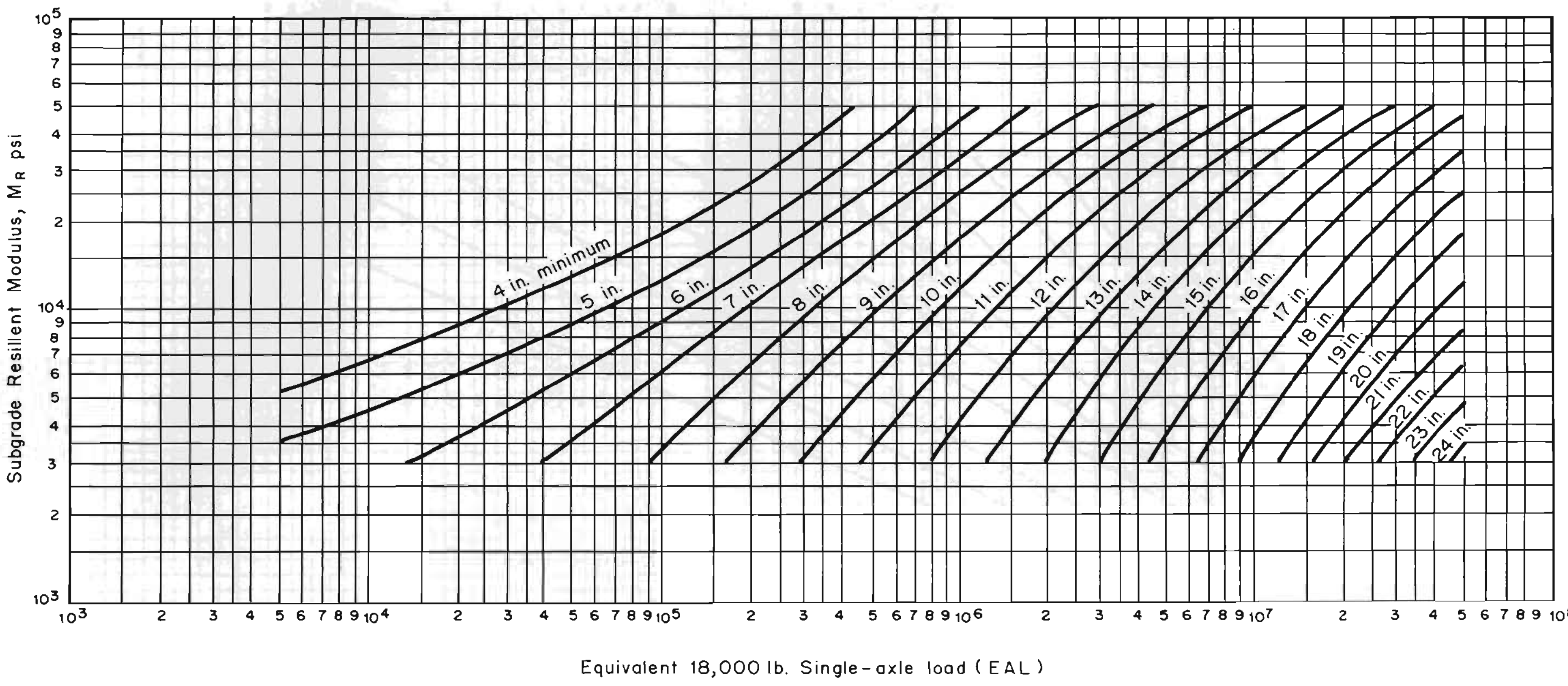


Source: Asphalt Institute 1981

Fig 18.18 Design chart

Emulsified asphalt mix type II

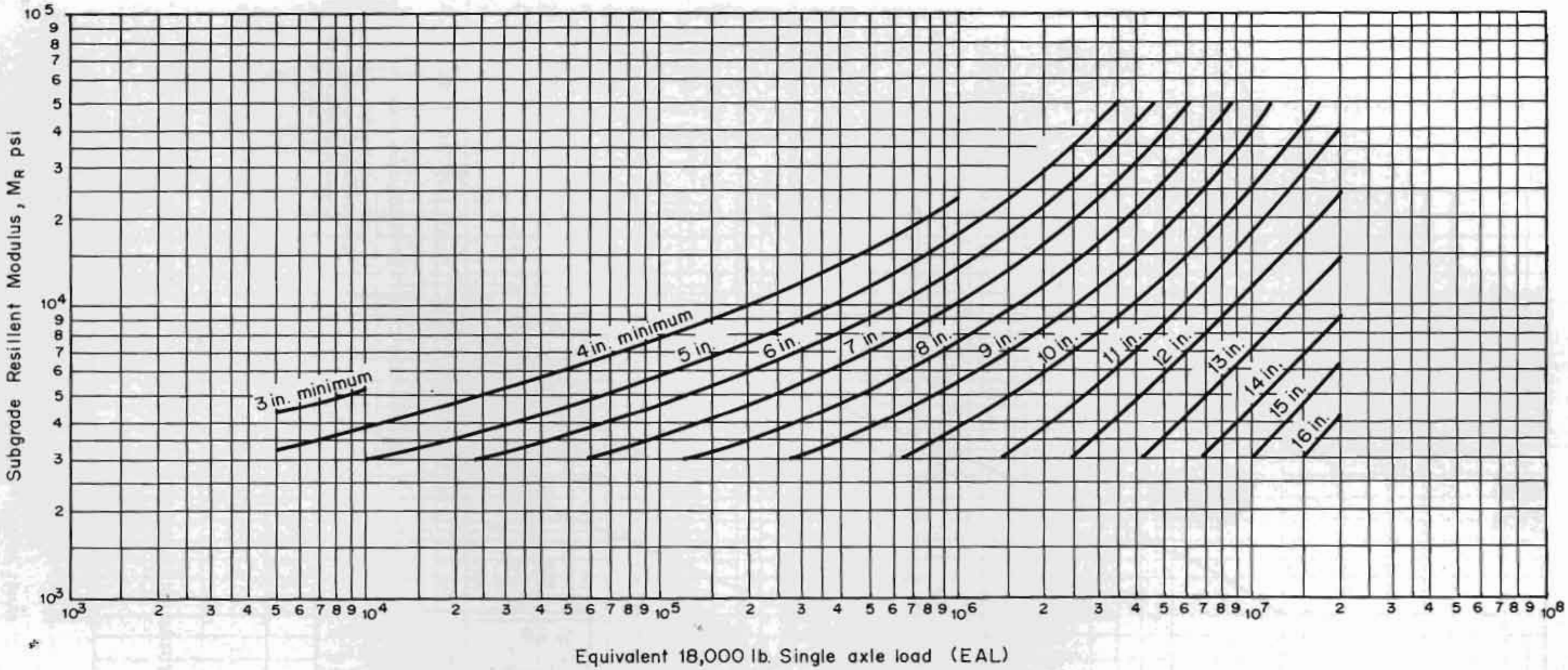
(Emulsified asphalt mixes made with semi processes, crusher run, pit-run, or bank run aggregates)



Source: Asphalt Institute 1981

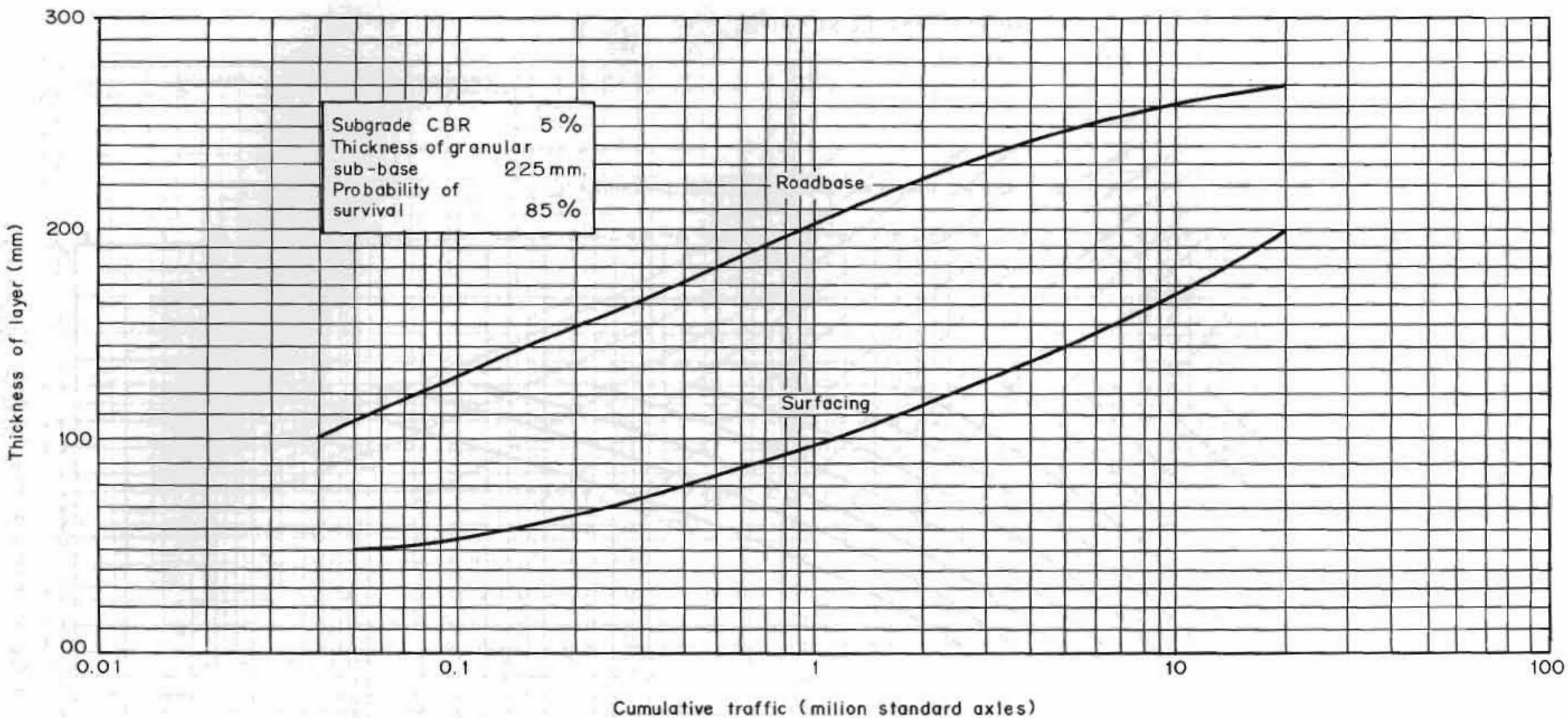
Fig 18.19 Design chart

Untreated aggregate base 6.0 in thickness



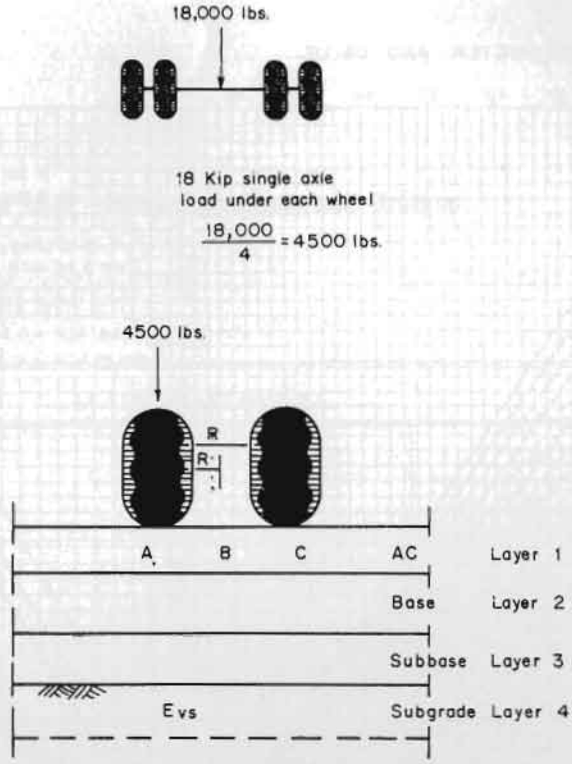
Source: Asphalt Institute 1981

Figure 18.20 Design chart



Source: TRRL Laboratory Report 1132, 1984

Figure 18.21 Design curves for roads with wet mix road base



THE PROBLEM PARAMETERS ARE

TOTAL LOAD: 4500.00 LBS
LOAD RADIUS: 4.37 IN.

TIRE PRESSURE: 75.00 PSI

LAYER 1 HAS MODULUS 5,00000 POISSON'S RATIO 0.350 AND THICKNESS 8.50 IN.
LAYER 2 HAS MODULUS 10500 POISSON'S RATIO 0.450 AND IS SEMI-INFINITE

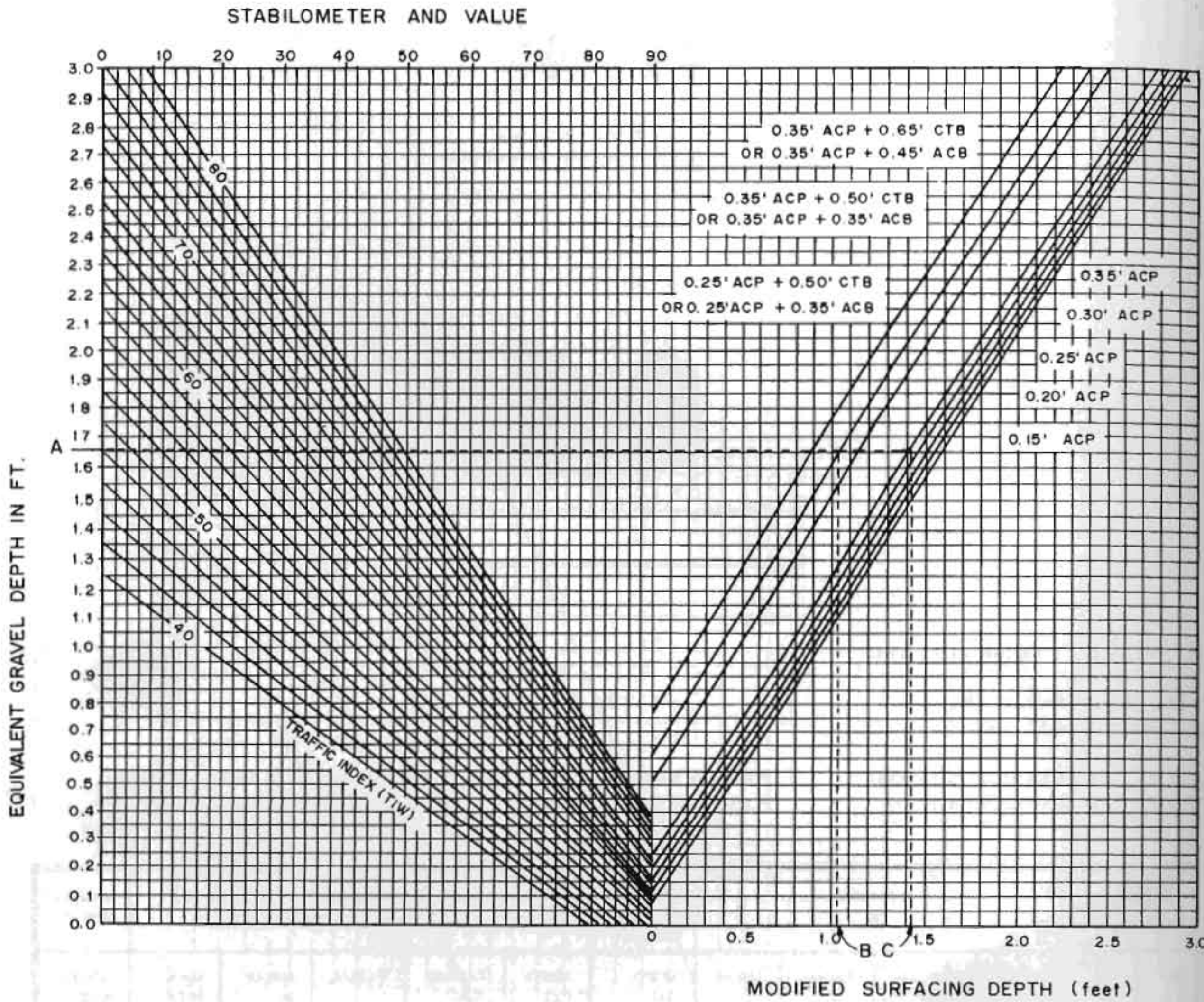
Location		STRESSES PSI				DEF-LECTION INCHES	STRAINS MICRO INCHES/INCH					ANGLE DEG
R	Z	VERTI-CAL	TANGEN-TIAL	RADIAL	SHEAR	VERTI-CAL	VERTI-CAL	TANGEN-TIAL	RADIAL	SHEAR IN MICRO RAD.	MAX. PRIN. IN TENS-ILE DIP	WITH P. AXIS
0.00	-8.50	-3.2648	71.7945	71.7545	0.0000	0.007855	-107.04	95.62	95.62	0.00	95.62	TR
0.00	8.50	-3.2648	-0.8458	-0.8458	0.0000	0.007855	-238.44	95.62	95.62	0.00	95.62	TR
5.50	-8.50	-2.6379	53.4541	42.5668	-0.5544	0.007474	-72.77	78.68	50.36	-2.99	78.68	T DIP
5.50	8.50	-2.6379	-0.8240	-1.0291	-0.5544	0.007474	-171.81	78.68	50.36	-153.13	78.68	T DIP
11.00	-8.50	-1.7787	30.2756	14.1699	-0.5118	0.006699	-34.67	51.88	8.39	-2.76	51.88	T DIR
11.00	8.50	-1.7787	-0.7226	-1.0375	-0.5118	0.006699	-98.97	51.88	8.39	-141.36	51.98	T DIR

Note: Maximum strain under dual wheel load
= [strain at B due to load under each wheel],
or [strain at A due to load at A and C],
whichever is greater.

Example,

$\epsilon t = 95.62 + 51.8 = 147.42$, or $2 \times 78.68 = 157.36$
therefore adopt, $\epsilon t = 157.36$

Figure 18.22 Example of stress, strain calculations using CHEVPC



Source: University of Washington 1986

Fig. 18.23 Structural design chart for flexible pavement

TRAFFIC INDEX, $T1 = 6.7\ (EWL/10^6)$, EWL = equivalent 5000 lb wheel load in one direction

EXAMPLE:

given an R Value of 25 and a traffic index of 6.0, cover thickness requirements can be determined as follows: an equivalent gravel depth of 1.65' (round to 1.7) at point A. A modified surfacing depth of 1.05 ft at point B for a pavement of 0.35 ft ACP + 0.50 ft CTB, or 0.35 ft ACP + 0.35 ft ACB. A modified surfacing depth of 1.43 ft (round to 1.45) for pavement of 0.35 ft ACP.

AC, D = 0.35

Untreated Base,

D = 1.45 - 0.35 = 1.10 ft

Table 18.9 Minimum pavement designs

MAIN ROADWAYS			
TRAFFIC INDEX	PAVEMENT	TREATED BASE	
		CTB	ACB
7.0 or more 6.5 to 6.9 6.4 or less	0.35 Ft. ACP 0.35 Ft. ACP 0.25 Ft.	0.65 Ft. 0.50 Ft. 0.50 Ft.	0.45 Ft. 0.35 Ft. 0.35 Ft.
RAMPS			
TRAFFIC INDEX	PAVEMENT	TREATED BASE	
7.0 or more 6.5 to 6.9 5.7 to 6.4 5.0 to 5.6 4.9 or less	0.25 Ft. ACP 0.35 Ft. ACP 0.25 Ft. ACP 0.25 Ft. ACP 0.30 Ft. ACP 0.15 Ft. ACP	0.60 Ft. PCC 0.35 Ft. ACP 0.50 Ft. PCC 0.50 CTB or 0.35 Ft. ACB Untreated Untreated	
In arid-areas BST may be used			
REST AREAS			
DESIGNATED AREA	PAVEMENT	TOP COURSE	
RAMPS, ACCESS ROADS, AND TRUCK PARKING	0.35 Ft. ACP	0.30 Ft. crushed surfacing	
CAR PARKING	0.25 Ft. ACP		

Source: University of Washington 1986

Table 18.10 Minimum depths of crushed surfacing for flexible pavements

UNDER BITUMINOUS PAVEMENT AND TREATED BASES		
TYPE OF PAVEMENT OR TREATED BASE	MINIMUM CRUSHED SURFACING DEPTH WHERE BASE IS:	
	GRAVEL BASE CLASS A OR BALLAST	GRAVEL BASE CLASS B OR SUB-GRADE
HIGH (a) (e)	0.20 ft	0.30 ft
INTERMEDIATE	0.20 ft	0.30 ft
LOW	0.25 ft	0.35 ft
ACB	0.15 ft (or 0.20 ft) (b)	0.20 ft (or 0.25 ft) (b)
CTB	None (c)	0.05 ft (d)

NOTES:

a) Applies when exceptions are allowed in arid areas and treated base is not required.
b) Use where traffic index is equal to or greater than 7.0.
c) Requires only sufficient fine material for keying and levelling. May be crushed or screened. Usually CTB aggregate.
d) 0.10 ft minimum depth of crushed CTB aggregate may be used instead.
e) Includes Portland Cement Concrete pavement.

ON SHOULDERS		
TYPE OF SHOULDER TREATMENT	MINIMUM CRUSHED SURFACING DEPTH WHERE BASE IS:	
	GRAVEL BASE CLASS A OR BALLAST	GRAVEL BASE CLASS B OR SUB-GRADE
0.15 Ft ACP BST	0.15 ft 0.20 ft	0.20 ft 0.25 ft

On ramps, frontage roads, and other miscellaneous lines these values may be reduced 0.05 ft

Pavement types shown are divided into the following categories.

High — Asphalt concrete pavement 0.25 ft minimum on treated base.

Intermediate — Asphalt concrete pavement on untreated base.

Low — Bituminous surface treatment.

Exceptions are permitted in arid areas.

Source : University of Washington 1986

Table 18.11 Flexible pavement design catalogue for low-volume roads

Tables 18.10 and 18.11 present a catalogue of flexible pavement SN-values (structural numbers) that may be used for the design of low-volume roads when the more detailed design approach is not possible. Table 18.10 is based on the 50 per cent reliability level and Table 18.11 is based on the 75 per cent level. The range of SN shown for each condition is based on a specific range of 18 kip ESAL applications at each traffic level :

High	:	700,000	to	1,000,000
Medium	:	400,000	to	600,000
Low	:	50,000	to	30,000

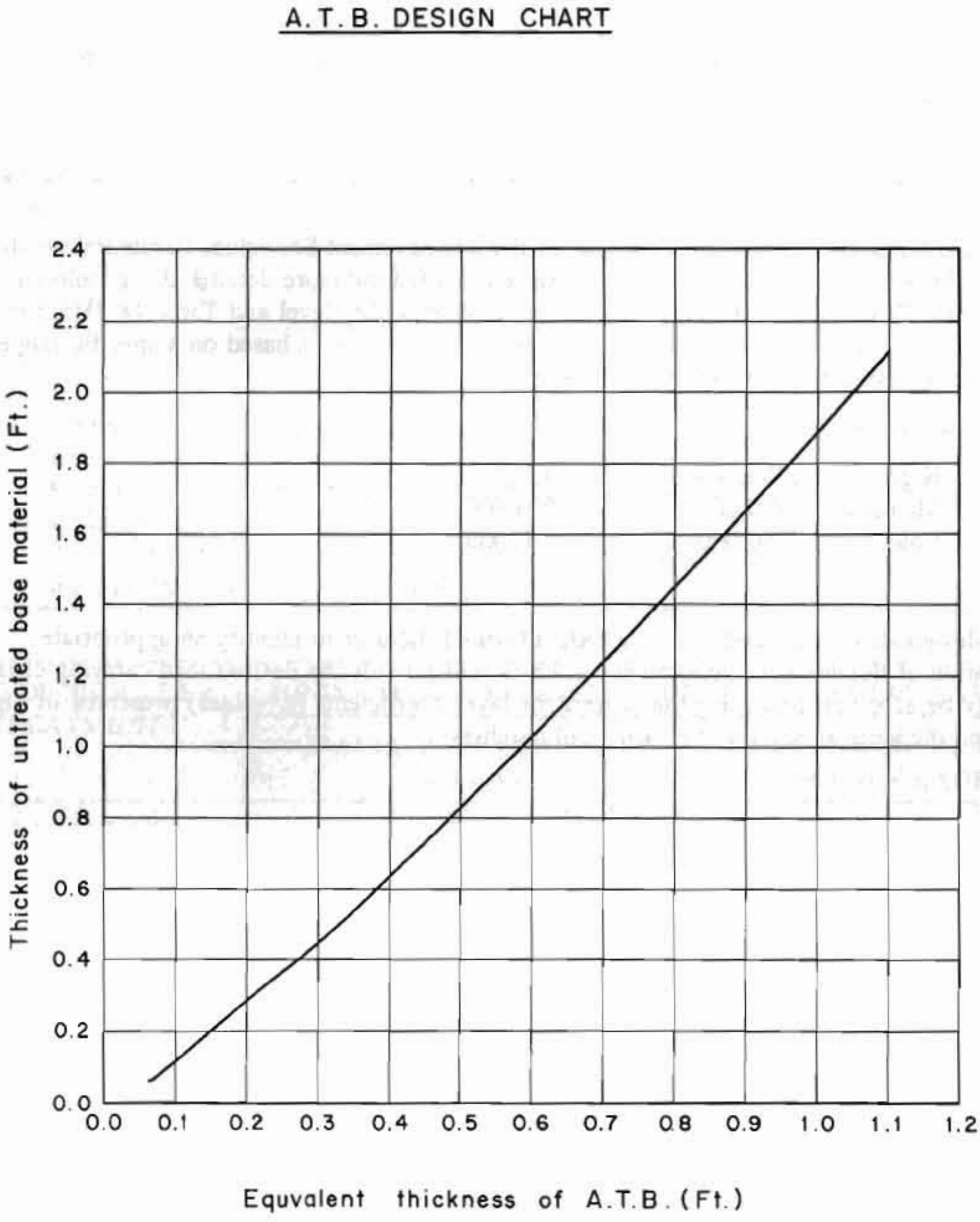
Once a design structural number is selected, it is up to the user to identify an appropriate combination of flexible pavement thickness which will provide the desired load-carrying capacity. This may be accomplished using the criteria for layer coefficients (a_i-values) presented in Figure 18.15 and the general equation for structural numbers :

$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

a_1,a_2,a_3 = layer coefficient for surface, base, and sub-base course materials, respectively, and

D_1,D_2,D_3 = thickness (in inches) of surface, base, and sub-base course, respectively.

Source: AASHTO 1985



Source: University of Washington

Fig. 18.24 Design chart

Table 18.12 Flexible pavement design catalogue for low-volume roads: recommended ranges of structural number (SN) for six U.S. climate regions, three levels of axle load traffic, and five levels of roadbed soil quality: inherent reliability 50%

Relative Quality of Roadbed Soil	Traffic Level	U.S. Climatic Region					
		I	II	III	IV	V	VI
Very Good	High Medium Low	2.3-2.5[1] 2.1-2.3 1.5-2.0	2.5-2.7 2.3-2.5 1.7-2.2	2.8-3.0 2.5-2.7 1.9-2.4	2.1-2.3 1.9-2.1 1.4-1.8	2.4-2.6 2.2-2.4 1.6-2.1	2.8-3.0 2.5-2.7 1.9-2.4
Good	High Medium Low	2.6-2.8 2.4-2.6 1.7-2.3	2.8-3.0 2.6-2.8 1.9-2.4	3.0-3.2 2.8-3.0 2.0-2.7	2.5-2.7 2.2-2.4 1.6-2.1	2.7-2.9 2.5-2.7 1.8-2.4	3.0-3.2 2.7-2.9 2.0-2.6
Fair	High Medium Low	2.9-3.1 2.6-2.8 2.0-2.6	3.0-3.2 2.8-3.0 2.0-2.6	3.1-3.3 2.9-3.1 2.1-2.8	2.8-3.0 2.5-2.7 1.9-2.4	2.9-3.1 2.6-2.8 1.9-2.5	3.1-3.3 2.8-3.0 2.1-2.7
Poor	High Medium Low	3.2-3.4 3.0-3.2 2.2-2.8	3.3-3.5 3.0-3.2 2.2-2.9	3.4-3.6 3.1-3.4 2.3-3.0	3.1-3.3 2.8-3.0 2.1-2.7	3.2-3.4 2.9-3.2 2.2-2.8	3.4-3.6 3.1-3.3 2.3-3.0
Very Poor	High Medium Low	3.5-3.7 3.2-3.4 2.4-3.1	3.5-3.7 3.3-3.5 2.4-3.1	3.5-3.7 3.3-3.5 2.4-3.1	3.3-3.5 3.1-3.3 2.3-3.0	3.4-3.6 3.1-3.3 2.3-3.0	3.5-3.7 3.2-3.4 2.4-3.1

Source: AASHTO 1985

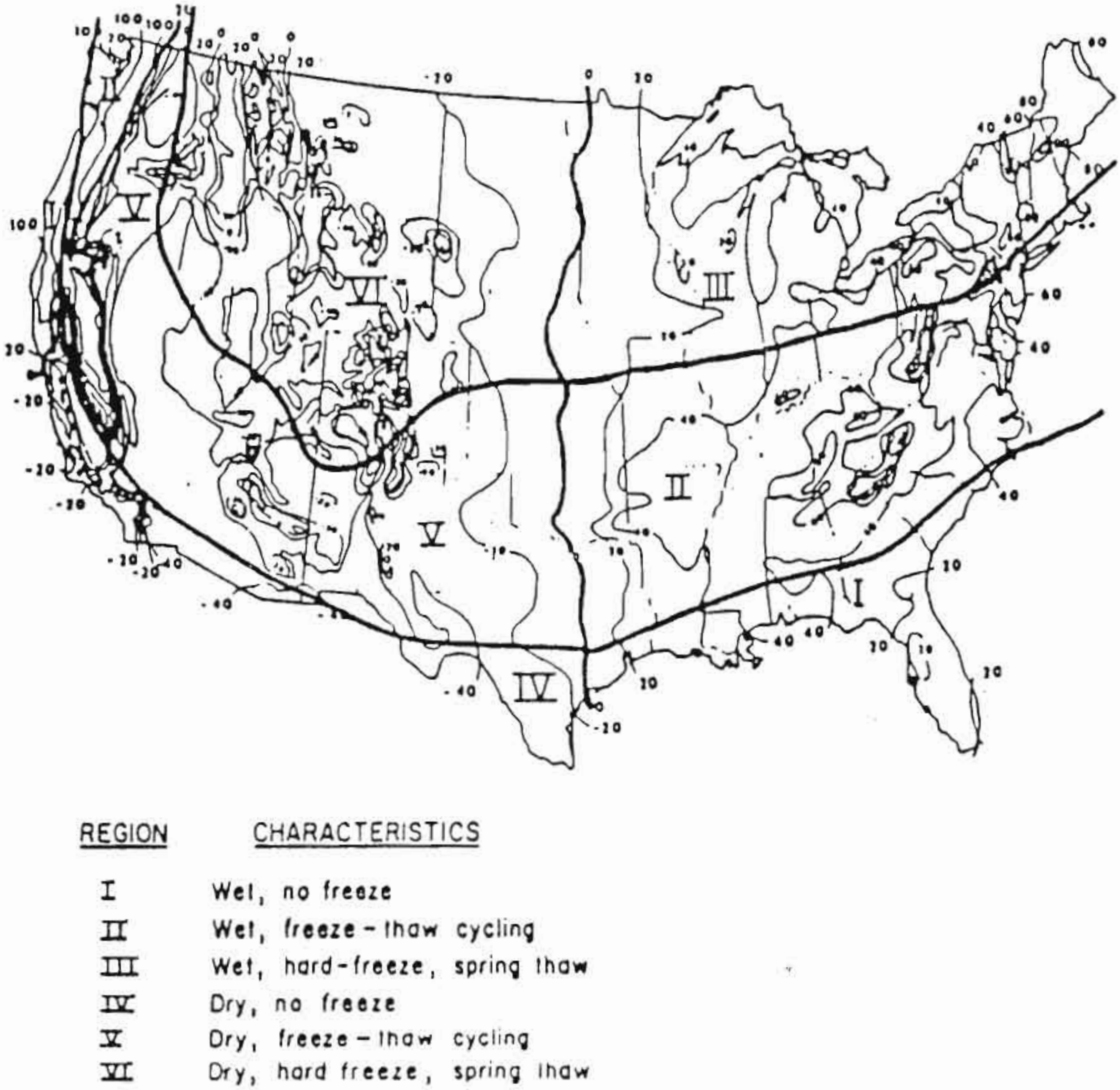
[1] Recommended range of structural number (SN).

Table 18.13 Flexible pavement design catalogue for low-volume roads: recommended ranges of structural number (SN) for six U.S. climate regions, three levels of axle load traffic and five levels of roadbed soil quality: inherent reliability, 75%

Relative Quality of Roadbed Soil	Traffic Level	U.S. Climatic Region					
		I	II	III	IV	V	VI
Very Good	High	2.6-2.7[1]	2.8-2.9	2.8-3.0	2.1-2.3	2.4-2.6	2.8-3.0
	Medium	2.3-2.5	2.5-2.7	2.5-2.7	1.9-2.1	2.2-2.4	2.5-2.7
	Low	1.6-2.1	1.8-2.3	1.9-2.4	1.4-1.8	1.6-2.1	1.9-2.4
Good	High	2.9-3.0	3.0-3.2	3.0-3.2	2.5-2.7	2.7-2.9	3.0-3.2
	Medium	2.6-2.8	2.7-3.0	2.8-3.0	2.2-2.4	2.5-2.7	2.7-2.9
	Low	1.9-2.4	2.0-2.6	2.0-2.7	1.6-2.1	1.8-2.4	2.0-2.6
Fair	High	3.2-3.3	3.3-3.4	3.1-3.3	2.8-3.0	2.9-3.1	3.1-3.3
	Medium	2.8-3.1	2.9-3.2	2.9-3.1	2.5-2.7	2.6-2.8	2.8-3.0
	Low	2.1-2.7	2.2-2.8	2.1-2.8	1.9-2.4	1.9-2.5	2.1-2.7
Poor	High	3.5-3.6	3.6-3.7	3.4-3.6	3.1-3.3	3.2-3.4	3.4-3.6
	Medium	3.1-3.4	3.2-3.5	3.1-3.4	2.8-3.0	2.9-3.2	3.1-3.3
	Low	2.4-3.0	2.4-3.0	2.3-3.0	2.1-2.7	2.2-2.8	2.3-3.0
Very Poor	High	3.8-3.9	3.8-4.0	3.5-3.7	3.3-3.5	3.4-3.6	3.5-3.7
	Medium	3.4-3.7	3.3-3.5	3.3-3.5	3.1-3.3	3.1-3.3	3.2-3.4
	Low	2.6-3.2	2.4-3.1	2.4-3.1	2.3-3.0	2.3-3.0	2.4-3.1

Source: AASHTO 1985

[1] Recommended range of structural number (SN).



Source: AASHTO 1985

Table 18.13(a) The six climatic regions in the United States

REGION	CHARACTERISTICS
I	Wet, no freeze
II	Wet, freeze-thaw cycling
III	Wet, hard-freeze, spring thaw
IV	Dry, no freeze
V	Dry, freeze-thaw cycling
VI	Dry, hard freeze, spring thaw

18.6 OVERLAY DESIGN (Based on J.P. Mahoney, 1985)

Pavements constitute about 15-25 per cent of the total cost of hill roads. Once the road is constructed, most of the works during maintenance are related to the pavement. The maintenance of roads in developing countries incur annual expenditure as costs for maintenance agencies in the range of 1.5 to 3 per cent of the updated construction costs. In addition there are user costs such as vehicle operation, time, and accident costs, associated with the road construction.

The general trends in the maintenance of roads in developing countries are limited to i) repair of drains, potholes, shoulders, retaining walls, and culverts and ii) resealing the surface with sand seal, chip seal, or single or double bituminous surfacing at intervals of from 4 to 8 years.

Traffic growth, axle load increase, and design life, in terms of total load rather than the number of years, is seldom analyzed in deciding the periodic maintenance levels. There are instances in which the roads, that are mostly designed for a lifespan of 10 to 15 years, realize the designed load much earlier rendering the original pavement structurally inadequate. Under such circumstances, resealing or surface treatment do not contribute much since they do not enhance the structural capability of the pavement. Overlaying based on traffic and axle load study, then, is the proper answer to considerably improve the life of the pavements and cost effectiveness of the investments. Periodic maintenance decisions, after about 5 years of service, of any important road, must therefore be based on proper evaluation of pavement conditions and alternatives.

It must be remembered that pavements on weak sub-grades have a much smaller load capacity to withstand both fatigue and rutting failures, compared to pavements of a similar thickness but on stronger sub-grades. Double bituminous surface treatment (DBST) may last 4 to 6 years provided the sub-grade is strong enough to withstand failure against rutting (deformation of sub-grade) for at least that period. The purpose of DBST would then be to prevent the weakening of base, sub-base, and sub-grade by moisture seeping down from the top. Thus, if the existing pavement has 40 years of rutting life,then DBST could be good enough for periodic maintenance for seven, 5 year cycles of DBST. The purpose of periodic maintenance in this case would be to save the underlying layers from accelerated weakening. The seven, 5 year cycles of DBST could be replaced by designing the surface course for a fatigue life of 40 years at the outset. This, however, is not possible because i) asphalt pavements are subject to aging of bitumen after 12 to 15 years, ii) the initial investment would be excessively high, and iii) traffic growth is not predictable. The next choice would be to initially design for 12-15 years of fatigue life and overlay every 12-15 years.

The important thing is to assess the difference in the maintenance efforts in terms of costs, investment levels, practicability, and reliability in choosing among 4 to 6 year cycles of DBST, 12-15 year cycle overlays, or 40 year cycle rigid pavements. In the case of a pavement on weak sub-grades, having a shorter life in terms of failure from rutting, there is no choice other than overlaying or new construction. DBST on these pavements will have a much shorter life than the normal 4-6 years for DBST. This section aims to present an example of practical situations in dealing with periodic maintenance of road pavements through overlay design by various methods. The example of overlay design presented here is based on the data and investigations relating to a 30 kilometre section of a highway in Nepal. Traffic and axle load data (Table 18.14) have been taken from existing studies for similar roads in Nepal. Pavement tests (Table 18.15) were carried out for a representative test section of 500 metres for each 5 km road length.

The purposes of this example are i) to emphasize the need for a systematic design approach to the design of overlays, ii) to familiarize the readers with some of the existing design methods, iii) to illustrate the need for engineering judgement and experience, along with the use of existing design charts and analysis, in the final selection of overlay design types and thickness appropriate to specific conditions. This example serves as a useful guide to the concepts and approaches for those pavement maintenance agencies where ad hoc decisions or experience-based judgement alone, rather than tests and analyses, are the practices in deciding pavement rehabilitation designs.

In this section, overlay design examples are presented using some of the several existing methods which are given below:

- a component analysis based on the Asphalt Institute,
- a component analysis based on the AASHTO Design Guide, and
- deflection based designs using:
  - the Asphalt Institute Design charts,
  - the TRRL Design charts,
  - the Canadian Good Road Association (CGRA) Design charts, and
  - the Mechanistic Design.

18.6.1 *Overlaying Design by Component Analysis Based on the Asphalt Institute*

This component analysis approach to overlay design involves the development of a total pavement structure as a new design for the specified service conditions, and then a comparison of the existing pavement structure (taking into account pavement condition, type, and thickness of pavement layers). A review of current component design procedures quickly reveals that substantial judgement is required to use them effectively. This judgement is mainly associated with selection of 'weighting factors' to use in evaluating the structural adequacy of the existing pavement layers.

The Asphalt Institute Method of component analysis (called "effective thickness") uses the relationships of sub-grade strength, pavement structure, and traffic. The existing structural integrity of pavement is converted to an equivalent thickness of asphalt concrete which is then compared to that required for a new design.

The three essential parts of this overlay design procedure will be briefly described and will include:

1. sub-grade analysis,
2. pavement structure thickness analysis, and
3. traffic analysis.

Sub-grade Analysis

Testing of sub-grade materials is encouraged, even if original design records are available. Use of resilient modulus (M_r), soaked CBR or R-value tests appear to be the easiest to use with this procedure. For actual design, the design strength of the sub-grade must be characterized in terms of resilient modulus. Associated correlations for CBR and R-value are :

$$M_r \text{ (psi)} = 1{,}500 \text{ (CBR)}$$
$$= 1{,}155 + 555 \text{ (R-value)}.$$

If test data in terms of M_r, CBR, or R-value are not available, sub-grades can be placed into one of the three classes for design purposes as given.

1. Poor soils. Soft and plastic when wet, generally composed of silts or clays. Typical properties : M_r = 4,500 psi, CBR = 3, R-value = 6.

2. Medium soils include soils such as loams, silty sands, and sand-gravels that contain moderate amounts of clay and silt. These soils can be expected to lose only a moderate amount of strength when wet. Typical properties: M_r = 12,000 psi, CBR = 8, R-value = 20.

3. Good soils. These soils can be expected to retain a substantial amount of their strength when wet and include clean sands and sand-gravels. Typical properties: M_r=25,000 psi, CBR=17, R-value=43.

Pavement Structure Thickness Analysis

The goal of this portion of the design method is to determine the "Effective Thickness (Te)" of the existing pavement structure. The Asphalt Institute has two approaches that can be used, only one will be illustrated in this section. First, the significant pavement layers are identified and their conditions determined. Second, "conversion factors" are selected for each layer (judgement by the designer is very important at this point). Third, the effective thickness for each layer is determined by multiplying the actual layer thickness by the appropriate conversion factor. The effective thickness of the complete pavement structure is the sum of the individual effective thickness. Typical layer thickness conversion factors are shown in Table 18.16.

Traffic Analysis

The Asphalt Institute treatment of traffic includes consideration of volume composition and axle weights with the goal being to develop the equivalent number of 18,000 to equivalent single axle loads (18-KEAL). Because of the trend of loading trucks heavily in the developing countries, the equivalency of trucks in terms of 18-KEAL or ESA tend to be much higher than in developed countries. It is, therefore, suggested that the equivalency factor for traffic load should be established based on axle load surveys of existing studies relevant to the situation concerned.

Table 18.17 is an example of overlay design based on the Asphalt Institute Component analysis. Conversion factors from Table 18.16 are used to convert the existing pavement to the effective thickness. The thickness of designed overlay is obtained for the given load and sub-grade strength from Figure 18.19.

It should be noted that the asphalt concrete to be used in overlays by this method should be the same as those assumed in the development of design charts which are for U.S. conditions (assuming an asphalt concrete modulus of 400,000 to 500,000 psi).

Table 18.14 Traffic data and ESA calculation

S. No.	Test	AADT				Traffic Growth % Per Year	Source	Equiv, Factor			ESA OR 18 KEAL	Cumulative ESA	Cumulative ESA in million.	Cumulative RSA x 106 one Lane	Remarks
		Buses	Trucks	Cars Light	Total AADT			Bus	Truck	Cars Lights					
1	1978	71	213	72	356	7		.739	2.718	.002	230515	230680	.23	.12	
2	1979	76	224	75	371	7		.739	2.718	.002	242509	473023	.47	.24	
3	1980	80	240	80	400	7		.739	2.718	.002	259734	732757	.73	.37	
4	1981	86	257	85	428	7		.739	2.718	.002	278221	1010978	1.01	.51	
5	1982	92	275	91	458	7		.739	2.718	.002	297701	1308680	1.31	.65	
6	1983	98	294	98	490	7		.739	2.718	.002	318174	1626854	1.63	.81	
7	1984	105	315	105	525	7		739	2.718	.002	340901	1967755	1.97	.98	
8	1985	112	337	112	561	7		739	2.718	.002	364620	2332374	2.33	1.17	
9	1986	120	360	120	600	7		.739	2.718	.002	389601	2721975	2.72	1.36	
10	1987	129	385	128	642	7		.739	2.718	002	416836	3138812	3.14	1.57	
11	1988	138	412	137	687	7		.739	2.718	.002	446056	3584868	3.58	1 79	
12	1989	148	4441	147	735	7		.739	2.718	.002	577280	4062148	4.06	2.03	
13	1990	158	472	157	787	7		.739	2.718	.002	510690	4572838	4.57	2.29	
14	1991	169	505	168	842	7		.739	2.718	.002	546438	5119276	5.12	2.56	
15	1992	181	540	180	901	7		.739	2.718	.002	584689	5703965	5.70	2.85	
16	1993	194	578	192	964	7		.739	2.718	.002	625617	6329582	6.33	3.16	
17	1994	207	618	206	1031	7		.739	2.718	.002	669410	6998992	7.00	3.50	
18	1995	222	662	220	1103	7		739	2.718	002	716269	7715261	7.72	3.86	
19	1996	237	708	235	1180	7		739	2 718	002	766408	8481669	8.48	4.24	
20	1997	254	757	252	1263	7		.739	2.718	.002	820056	9301725	9.30	4.65	
21	1998	271	810	269	1351	7		.739	2.718	.002	877460	10179185	10.18	5.09	
22	1999	290	867	288	1446	7		.739	2.718	.002	938882	11118068	11.12	5.56	
23	2000	311	928	309	1547	7		.739	2.718	.002	1001604	12122672	12.12	6.06	

Source: University of Washington 1986

Note: 1. Equivalency factors for both the lanes are assumed to be same since deflection results do not show specific trend and this particular road carries loaded traffic in both directions.

2. Design traffic:
Case-1 = 10 years (end of 1988 to end of 1997) = 3.08:10^6 ESA
Case-2 = 5 years (end of 1988 to end of 1992) = 1.28:10^6 ESA

3. Base year traffic = 1986 = 3.14/2 =1.57 x 10^6 ESA (for one lane)

Table 18.15 Test and design parameter

TEST DATA												ADOPTED FOR DESIGN			
TEST SECTION	KM	CR. DEFLN. (15k axle) = x+2SD, x10^9 in	AV. γ_d (In situ) pcf	AV.W (In situ) %	γ_d max pcf	Wopt % SOAKED	AV Field CBR (Dcp) %	AV. Lab CBR Soaked %	PVT. TEMP °F	PVT. Thk in	SOIL TYPE	Design x10-3 15k axle	Defln. in 18k axle	CBR	MR ksi
Surface	0-5	11.42							84.2	2	Well-graded sandy gravel with little fines.	11.42	14.28	36	22
Base			111.39	24.2	135.19	7.4	36.0	64.00		13.71				36	15.80
Sub-grade			113.45	3.15	139	7.8	44.5	7.5							
Surface	6-10	39.76							86	2.2		39.76	49.70	34	21
Base			109.7	4.2	134.35	5.0	33.5	37.00		11.8				28	14.40
Sub-grade			112.09	6.58	127.35	8.8	32.5	11.00							
Surface	11-15	42.52							95	2.2	Well-graded sandy gravel with very little fines.	42.52	53.15	33	21
Base			125.06	4.56	139.38	7.5	33.0	53.50		8.3				28	14.40
Sub-grade			105.47	10.19	127.35	10.0	28.0	12.50							
Surface	16-20	36.22							87.8	2	well-graded sandy gravel with little fines.	36.22	45.28	45	22.50
Base			129.87	4.81	136.51	7.6	53.5	25.50		6.7				20	12.70
Sub-grade			105.31	10.88	136.2	9.8	18.5	15.50							
Surface	21-25	48.42							84.2	2	well graded sandy gravel with little fines.	48.42	60.53	32	20.50
Base			125.54	2.49	136.00	6.7	31.5	39.00		7.9				19	12.70
Sub-grade			104.64	10.84	139.88	6.5	19.0	3.50							

a) Field CRR values are adopted without any adjustments since:

1. No seasonal factor is considered because the tests were carried out when the sub-grade conditions were wetter.

2. The results of soaked CRR are erratic and too unreliable to be accepted for design.

3. The material properties and classification do not have a definite trend.

4. Tests are carried out for the worst 500 m length for each 5 km road length, therefore involve conservatism.

5. Deflection under 18,000 pound axle dual wheels should be taken as 1.25 times the deflection under 14,000 pound axle dual wheels, wherever necessary. This is based on PSADZA computer analysis carried out by the University of Washington, Seattle, during a study for Washington State Department of Transportation on "Evaluation of Frost Related Effects on Pavements", May 1984.

Table 18.16 Example of Asphalt Institute conversion factors for estimating thickness of existing pavement components to effective thickness

Description of Layer Material		Conversion Factor[1]
1.	Native sub-grade	0.0
2a.	Improved sub-grade - predominantly granular materials	
b.	Lime modified sub-grade of high PI soils	
3a.	Granular sub-base or base-CBR not less than 20	0.1 - 0.3
b.	Cement modified sub-bases and constructed from low PI soils	
4a.	Cement or lime-fly ash bases with pattern cracking	0.3 - 0.5
b.	Emulsified or cutback asphalt surfaces and bases with extensive cracking, rutting, etc	
c.	PCC pavement broken into small pieces	
5a.	Asphalt concrete surface and base that exhibit extensive cracking	0.5 - 0.7
6a.	Asphalt concrete - generally uncracked	0.9 - 1.0
b.	PCC pavement - stable undersealed and generally uncracked pavement	

Source: Asphalt Institute 1981

[1] Equivalent thickness of new asphalt concrete

Table 18.17 Overlay design by component analysis method - Asphalt Institute

- Design Traffic = 3.08×10^6 ESA, = (Case 1 = 1988-1997)
- Design Traffic = 1.28×10^6 ESA, = (Case 2 = 1988-1992)

Description	Section 1 0 - 5 km	Section 2 6 - 10 km	Section 3 11 - 16 km	Section 4 11 - 21 km	Section 5 22-26 km
Sub-grade, MR,					
ksi 15.8	14.4	14.4	12.7	12.7	
Total existing thickness	15.78in.	14.00in.	10.50in.	8.7in.	9.9in.
Effective AC thick. of existing pavt., in (from Table 18.16)	0.5 x 2 + 3x15.78" = 5.73	0.5 x 2 + 3x14 " = 5.20	0.5 x 2 + 3x10.5" = 4.15	0.5 x 2 + 3x8.1" = 3.61	0.5 x 2 + 3x9.9" = 3.97
Total thick. of AC reqd. for design ESA, in. (Fig 18.19)					
Case - 1	9.30	9.70	9.70	10	10
Case - 2	7.60	8	8	8.20	8.20
<u>Designed Overlay</u>					
Reqd. overlay (dense graded) thick., in.					
Case - 1	3.75	4.50	5.55	6.39	6.03
Case - 2	1.87	2.80	3.85	4.59	4.23

Source: Asphalt Institute Manual Series No. 1, 1981

The above AC may be converted to emulsifical AC and a two layer overlay with a combination of AC and untreated aggregate by using the following conversions.
1" Ac = 1.43" type II emulsified AC. 1"AC = 3" gravel.
(Type II emulsified asphalt mixes made with semi-processed all crusher run, pit run, or bank run aggregates.)

18.6.2 *Overlay Design Based on AASHTO Design Guide*

This method of design requires the determination of the total thickness of pavement based on traffic, reliability, drainage factors, serviceability loss, and layer co-efficients. The existing pavement structure is then deducted from the total thickness required by the new design, with the difference being the required overlay thickness. One of the significant assumptions in this method is that each layer of the pavement structure is assigned a layer co-efficient on the basis of experience.

Figure 18.12 illustrates the new design concept and Figures 18.13 to 18.18 and Tables 18.13 and 18.14 are nomographs and design parameters. Table 18.18 is an example of design by this method. It may be noted that the existing thickness of pavement with bituminous surfacing (about 2") and base course of untreated aggregates have been converted to the equivalent thickness of 400,000 psi asphalt concrete by assuming that 1" of existing bituminous surfacing is equivalent to 2" of untreated base course and 3" of untreated base course is equivalent to 1" of 400,000 psi asphalt concrete. Similarly, the total design thickness, in terms of 400,000 psi asphalt concrete, is obtained by converting the untreated base to asphalt concrete by assuming an equivalency of 3:1. Thus the difference of the total equivalent thickness of existing and new designs gives the thickness of overlay expressed in terms of 400,000 psi asphalt concrete.

One should be very careful in comparing the results of design by various methods. It must be ensured that the conditions of materials and assumptions in the designs are similar. For example, the design for a reliability of 50 per cent would be quite different from the design for a reliability of 80 per cent. Similarly, the design for an asphalt concrete of stiffness 200,000 psi would give much greater thickness in comparison to a design for 400,00 psi AC.

18.6.3 *Overlay Based on Deflection Criteria*

The objective of deflection testing is to measure the structural properties of the pavement by non-destructive testing. This is done by imposing a known load on the pavement and measuring its response (i.e., surface deflection). Thus, an overall or effective strength is measured that combines all influencing factors such as material properties (including sub-grade), thickness of pavement layers, and environmental effects. The most commonly used, deflection-based, overlay design procedures do not attempt to isolate material properties of individual pavement layers.

The dominant type of measurement used for non-destructive, overlay design procedures is surface deflection (or deflection basins) obtained with known load conditions (i.e., contact pressure, force, and time-loading). Each of these factors can influence the pavement response to loading. Surface deflection measurements can be categorized into three types of non-destructive test: static deflections, steady-state deflections, and impact load response. Some examples of equipment associated with these tests are given below.

1. Static Deflections :
   Benkelman Beam, travelling deflectometer, and plate-bearing test (ASTM D 1196).

Table 18.18 Overlay design based on AASHTO design guide

Past traffic = 1.575×10^6 ESA
Design Future traffic, Case 1 = 3.08×10^6 ESA

Case 2 = 1.28 x 106 ESA

Assuming three layers including sub-grade,
Structural Number, $SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$
n2 = 1.0 for good drainage quality
a1 = 0.42 assuming average annual pavement temperature of 90 F. (from Fig. 18.15)
E1 = 400,000 psi at 84 F (Fig. 18.14)
a2 = 0.249 (logEbase) - 0.977.

Description	Section-1	Section-2	Section-3	Section-4	Section-5
CBRbase	36	34	33	45	32
Ebase, ksi (Table 18.15)22	21	21	22.50	20.50	
a2 .104	.099	.099	.107	.097	
CBRsgr (Table 18.15)	36	28	28	20	19
Esgr, ksi (Fig 18.17)	15.80	14.40	14.40	12.70	12.70
From Fig. 18,13,					
for reliability 50%					
std. deviation of 0.45 SNI					
Case 1	1.9	1.9	1.9	1.8	2.0
Case 2	1.0	1.0	1.0	1.0	1.0
SN2					
Case 1	2.4	2.6	2.6	2.7	2.7
Case 2	1.5	1.6	1.6	1.7	1.7

Total thickness

D1 = SN1/a1 = snl/.275 D2 = (SN2-SN1)/a2 0.8 ; m_2 = 0.8 from Table 18.5 for Fair and > 25 % case.

Description	Section-1	Section-2	Section-3	Section-4	Section-5
Case 1					
D1, in	4.5	4.5	4.5	4.3	4.8
D2, in	4.8	7.1	7.1	8.4	7.2
Case 2					
D1, in	2.4	2.4	2.4	2.4	2.4
D2, in	4.8	6.1	6.1	6.5	7.2
Existing thickness, D2 in					
. In terms of untreated base material (assuming 1" existing surface = 2" base)	13.78+2x2 = 17.78	12+2x2 = 16	8.5+2x2 = 12.5	6.7+2x2 = 10.70	7.9+2x2 = 11.9
. In terms of new A.C (assuming 1" AC 400 ksi = 3"base)	5.93	5.33	4.16	3.56	3.96
Overlay thickness, in					
AC 400 ksi;					
DE - (D1 + D2/3)					
Case 1	0.2	1.5	2.7	3.6	4.7
Case 2				1.0	0.8

Source: AASHTO 1985

2. Steady-state Deflections :
   Dynaflect, Road Rater (several models), Waterways Experiment Station Plate Vibrators, and the Federal Highway Waterway Association (FHWA) Deflection Van (Cox 1981).

3. Impact Load Response :
   Falling Weight Deflectometer.

Figure 18.25 outlines the general approach used in most of the overlay design procedures based on deflection measurements. The three basic elements of such design procedures are :

1. deflection measurement,
2. pavement conditions, and
3. traffic.

The minimum elements to be encompassed in mechanistic overlay design are given in Figure 18.26. A widely used deflection-based overlay design procedure is the Asphalt Institute Method. It will be described to illustrate the approach for asphalt concrete overlays placed on existing flexible pavements.

Asphalt Institute Overlay Design by Deflection Analysis

The basic approach of the overlay design procedure is to identify continuous pavement sections of uniform performance, obtain 'static' pavement, surface deflections with the Benkelman Beam and an 18,000lb single axle, and determine the expected traffic by user-equivalent axle loads.

The Asphalt Institute recommends that a minimum of 20 deflection measurements be taken each mile and randomly located in the outer wheelpath. From this data for each 'uniform' pavement section, a "representative rebound deflection" (RRD), is determined as follows :

$$RRD = (x + 2s)\,(f)\,(c)$$

where,

RRD	=	representative rebound deflection (in.),
x	=	mean of the individual deflections (in.),
s	=	standard deviation of the deflections (in.),
f	=	temperature adjustment factor, and
c	=	critical period adjustment factor (where c= 1 if deflection tests made during the most critical period).

This calculation of RRD represents the upper bound of about 97 per cent of all deflections measured. The temperature adjustment factor used in the equation above adjusts the existing asphalt concrete surfacing to a standard temperature of 70° F (refer to Fig. 18.27).

The deflection measurements obtained by use of the Benkelman Beam can be used to estimate the remaining life of the pavement or the needed thickness of asphalt concrete overlay. To determine the required overlay thickness, Figure 18.28 is used with the RRD and 18 KEAL as the required input.

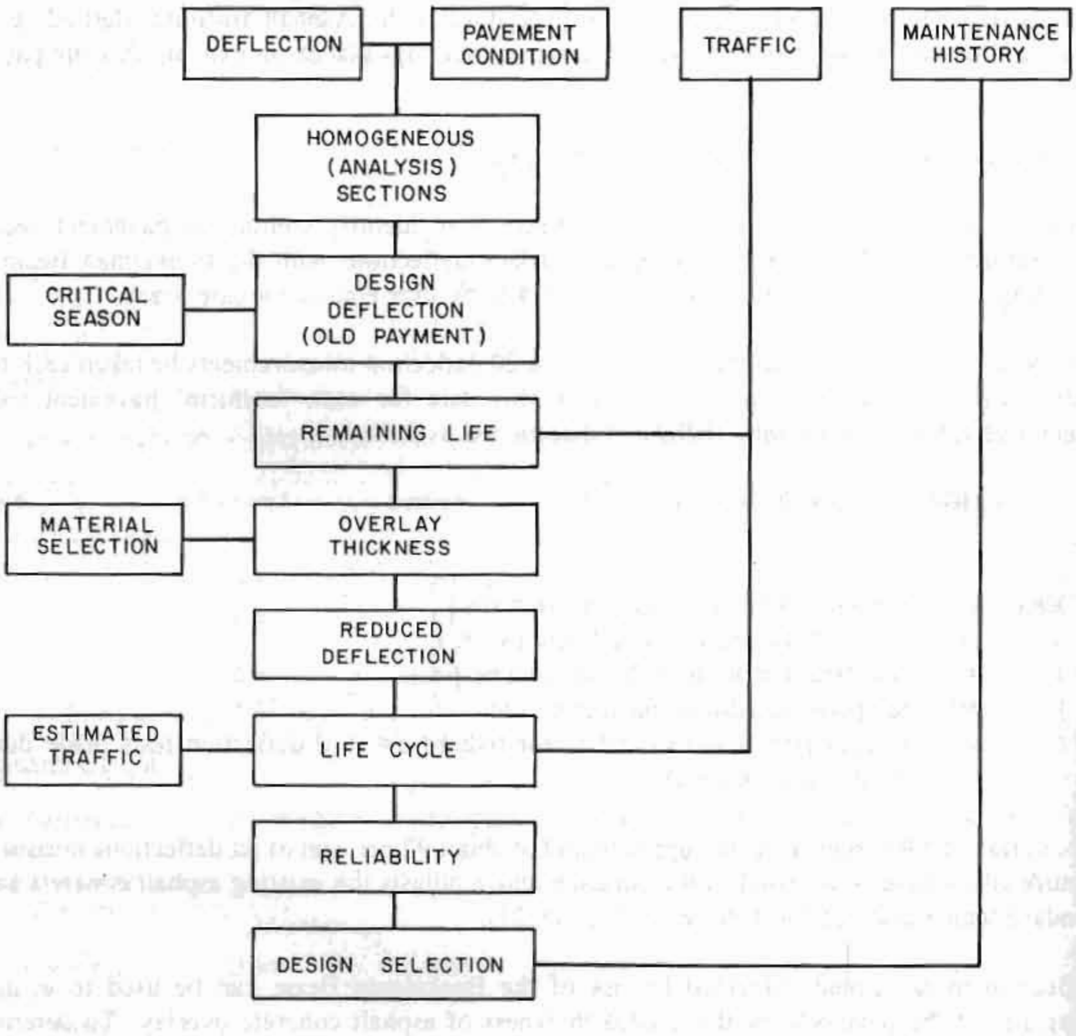
Table 18.19 presents an example of overlay design by the Asphalt Institute Method.

Allowable Deflection Criteria based on GGRA, Liste, and Cox 1981

Table 18.20 presents an example of overlay design based on allowable deflection criteria (Fig. 18.29 and Fig. 18.30) and GGRA design chart (Fig. 18.31).

The Transport and Road Research Laboratory (TRRL) Method

Table 18.21 presents an example of overlay design based on Transport and Road Research Laboratory Lab. Report No. 833 (1978). Figures 18.32, 18.33, and 18.34 are design charts for standard (allowable) deflection. This method involves determination of the remaining life of the existing pavement to ascertain whether overlay is required or not. The design charts for this method are applicable to sub-grade CBR up to 15 per cent only. However, these charts have been used here just to present an example only.



Source: University of Washington 1986

Figure 18.25 Overlay design with deflection measurements

18.6.4 *Overlay Design by Mechanistic Analysis*

Mechanistic Analysis

Significant interest has developed in the use of mechanistic overlay design procedures. The term 'mechanistic' as defined in most dictionaries (such as Webster's Seventh New Collegiate) is "mechanically determined" or "pertaining to the doctrine of mechanism". In turn, 'mechanism' is defined as the "fundamental physical processes involved in or responsible for an action, reaction, or other natural phenomenon". This roughly translates to pavement engineers as determining the fundamental **stresses strains** and **deflections** caused by traffic and/or the environment in pavement structures. Knowledge of these stresses, strains, or deflections can in turn be used with limiting criteria to evaluate not only the need for an overlay but remaining pavement life as well.

The greatest advantage of mechanistic-based methods is the versatility provided in evaluating different materials under various environments and pavement conditions. The mechanistic procedures provide a basis for rationally modelled pavement systems. As these models improve, better correlations can be expected between design and performance parameters. It is anticipated that these procedures will replace limiting deflection, overlay methods since the latter do not account for sub-surface material properties. Mechanistic overlay design should, at a minimum, encompass the elements illustrated in Figure 18.26. Selected elements shown in this figure will be separately discussed.

Analysis Sections

A reasonable amount of uniformity should exist within a given pavement segment being considered for overlaying. These actions can be initially identified by use of condition surveys and, finally, deflection measurements. There exist numerous methods to determine the required number and location of such measurements. A minimum sample generally consists of deflection measurements every 250 to 500 ft. After collection of the deflection measurements, statistical measures can be used to delineate between analysis sections (along with the condition surveys).

Layer Characteristics

The mechanistic approach to overlay design can encompass both material characterization from the laboratory and non-destructive test data collected in the field. Total reliance on either laboratory or field data is generally felt to be inappropriate at the current stage of development. However, recent developments have provided estimates from field data of *in situ* moduli of the pavement layers. Illustrations of these approaches include :

1. FHWA - Resource International overlay design procedure,
2. BISDEF - computer programme developed by Bush (1980) at the Waterways Experiment Station (not an overlay design system),
3. ELMOD - computer programme by Ullidtz (1977), and
4. several other analysis procedures which use deflection basins from the Falling Weight Deflectometer, Dynaflect, or Road Rater.

Laboratory testing for mechanistic analysis generally implies the determination of resilient moduli (essentially a "modulus of elasticity" for pavement materials). Standard test methods such as the American Society for Testing and Materials (ASTM), D4123 - 82, are used for bituminous mixtures and triaxial procedures such as those recommended by Kalcheff and Hicks (1973) can be used for unbound granular materials. Laboratory determined moduli from unbound base, sub-base, and sub-grade materials are stress-sensitive and as such must be recognized.

The laboratory derived moduli are often adjusted by using layered-elastic analysis to calculate the resulting maximum deflection or deflection basin for a specified loading condition. In turn, field deflections are compared to the estimated deflections. If differences exist, the laboratory values are modified to reasonably match field measurements. More recently, computer programmes such as BISDEF have been used to estimate layer moduli for up to four pavement layers. Input data for this programme include the non-destructive test (NDT) load, measured deflection basin, layer thickness and limiting ranges, and expected values of moduli for each pavement layer. The programme then estimates the moduli for each layer which results in the best fit of the field deflection basin (within a user-specified error range).

Limiting Failure Criteria

Pavement sections deteriorate with time because of a progression of defects (because of traffic loads, environment, and other factors acting on the pavement structure). The pavement reaction to its total loading condition can be characterized by estimating the induced stress, strain, and deflections. When these pavement responses reach cumulative limiting value, distress results. The resulting serviceability loss can occur as a result of the accumulation of a single distress type (often fatigue), rutting, or a combination of several types.

Fatigue-related distress can be defined as the phenomenon of load-induced cracking caused by repeated stress or strain level below the ultimate strength of the material. The classical type of fatigue failure is commonly described as 'alligator' cracking, because of the pattern of cracks which appear on the pavement surface. These cracks appear to be best associated with tensile strains at the bottom of the asphalt concrete layers. A common expression used to relate the number of loads to fatigue failure as a function of tensile strain is:

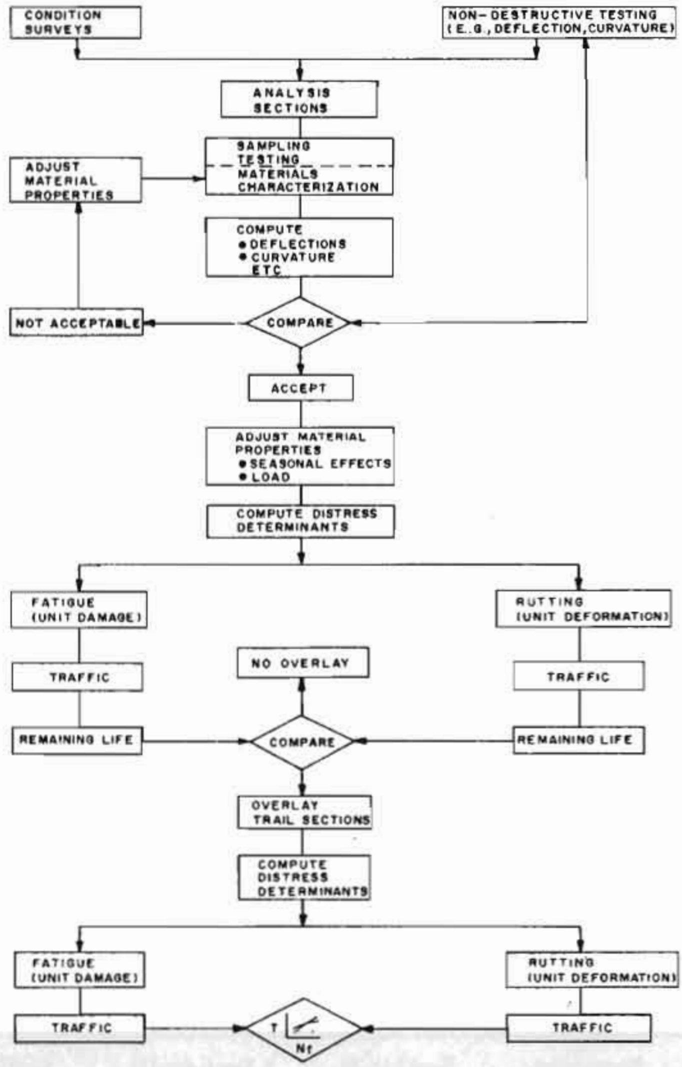
$$Nf = K1\ (1/E_i)^k$$

where,

Nf = load repetitions to failure,
E_i = initial tensile strain, and
K_i, K_2 = fatigue parameters.

The fatigue relationship developed by Majidzadeh and Ilves (1981) for the FHWA-RII Overlay Design System is used to illustrate 'typical' K_1, and K_2 parameters:

$$N_f = 7.56 \times 10^{-12}\ (1/\epsilon_t)4.68.$$



Source: Monsmith and Finn 1984

Figure 18.26 Overlay design based on mechanistic analysis

Analogous criteria have been developed for rutting failure, whereby the number of load repetitions to failure is generally made a function of vertical strain in the sub-grade (in place of tensile strains as for asphalt concrete fatigue).

In practice, flexible pavements are subjected to a variety of loads. Miner's rule is used for evaluating cumulative damage. The rule states that the condition at failure is given by :

where,

n_i	=	actual number of cycles of stress or strain applied to the pavement,
N_i	=	allowable number of cycles to failure based on failure criteria (such as fatigue or rutting), and
r	=	number of loading conditions considered.

In this example of mechanistic design, the CHEVPC computer programme has been used. Stress sensitivity is not considered for sub-grade and base course. The failure criteria for fatigue and rutting as per Finn's Model (see Section 18.5.7) have been used to calculate the failure loads from the strain obtained from the CHEVPC programme.

Table 18.22 shows the material properties adjusted by matching the observed defection with the calculated deflection (by CHEVPC). The considerable variation between the adjusted material properties and test data might be indicative of the inadequacy of test data. Table 18.23 shows the remaining life of the existing pavement and Table 18.24 shows the failure loads for different trial thicknesses of the overlay.

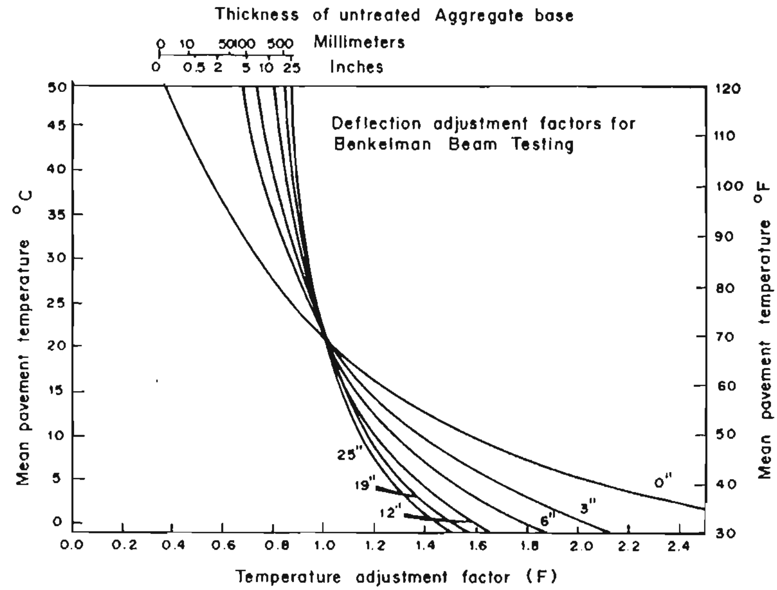
The results from this method indicate a much higher overlay thickness compared to other methods. It should be remembered that the reliability in most other methods is fifty per cent only compared to more than 50 per cent in this method. The deflection with mean plus two standard deviation, adopted as measured deflection for matching calculated deflection, further renders the design more conservative. The poor reliability of test results (obvious from the inconsistencies among material type, CBR field, and CBR lab [see Table 18.15]), and the uncertainty of the measured thickness of existing pavement, all tend to render the design more conservative, leading to a great deal of over-calculation in the thickness of the overlay.

Table 18.19 Overlay design by deflection analysis based on the Asphalt Institute

Case-1, traffic 3.08 x 10^6 ESA (1988 - 1997)
Case-2, traffic 1.28 x 10^6 ESA (1988 - 1982)

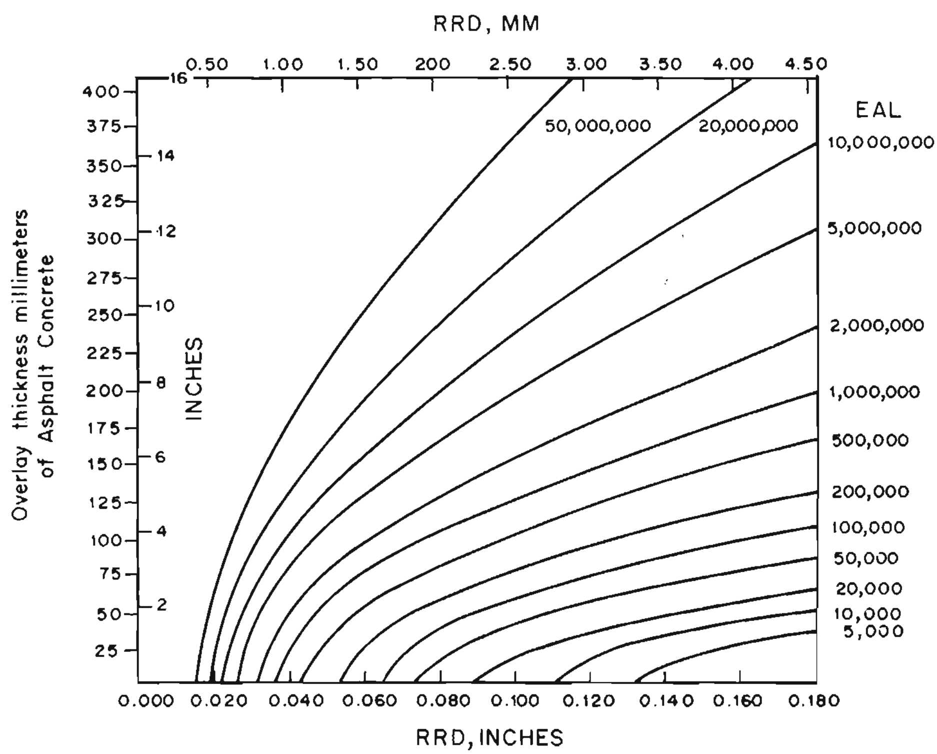
Description	Section-1	Section-II	Section-III	Section-IV	Section-V
RRD(dc in terms of 1800 lb. dual wheel load x 103 in. (from Table 18.15)	14.27	49.70	53.15	45.27	60.52
Designed thick. of overlay (from Figure 18.28) AC Nr = 400 ksi					
Case - 1	nil	3.70	4	3.80	5.60
Case - 2	nil	3.10	3.20	2.30	4.50

Source: Asphalt Institute 1983



Source: Asphalt Institute 1983

Fig. 18.27 Asphalt Institute temperature adjustment factors for Benkelman Beam deflections



Source: Asphalt Institute 1983

Fig. 18.28 Asphalt concrete overlay thickness required to reduce pavement deflection from a measured to a design deflection value

Table 18.20 Overlay design by allowable deflection criteria

Case - 1, traffic 3.08×10^6 ESA (1988 - 1997)
Case - 2, traffic 1.28×10^6 ESA (1988 - 1982)

Assumed Allowable Deflection under 18 K - axle :

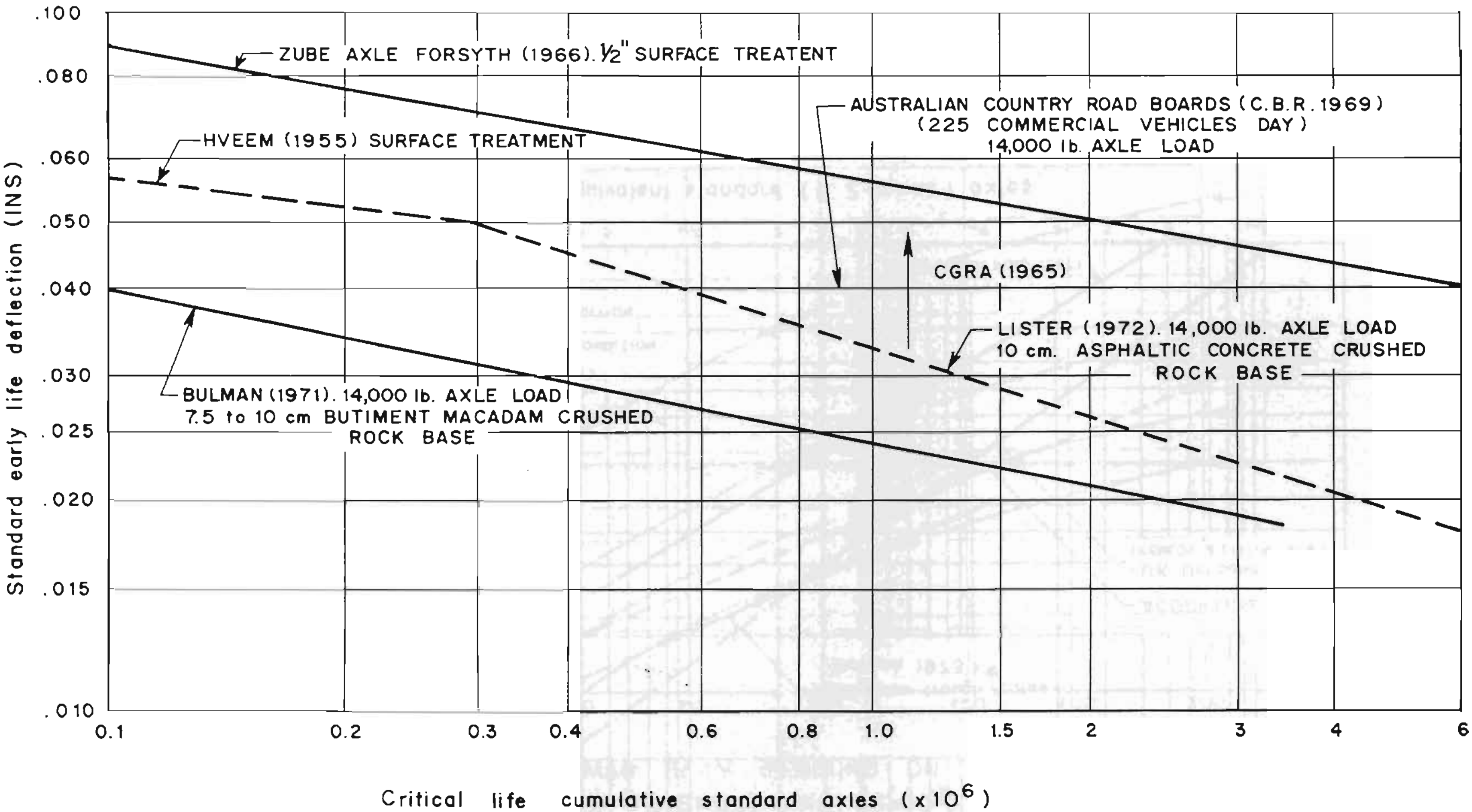
	Lister 1972 (Fig. 18.29)	Cox 1981 (Fig. 18.30)
Case - 1	33 x 10-3in.	39 x 10-3in.
Case - 2	46 x 10-3in.	50 x 10-3in.

Allowable Deflection under 14K-axle based on Lister, 1972

For case - 1	-	23 x 10-3in.
For case - 2	-	28 x 10-3in.

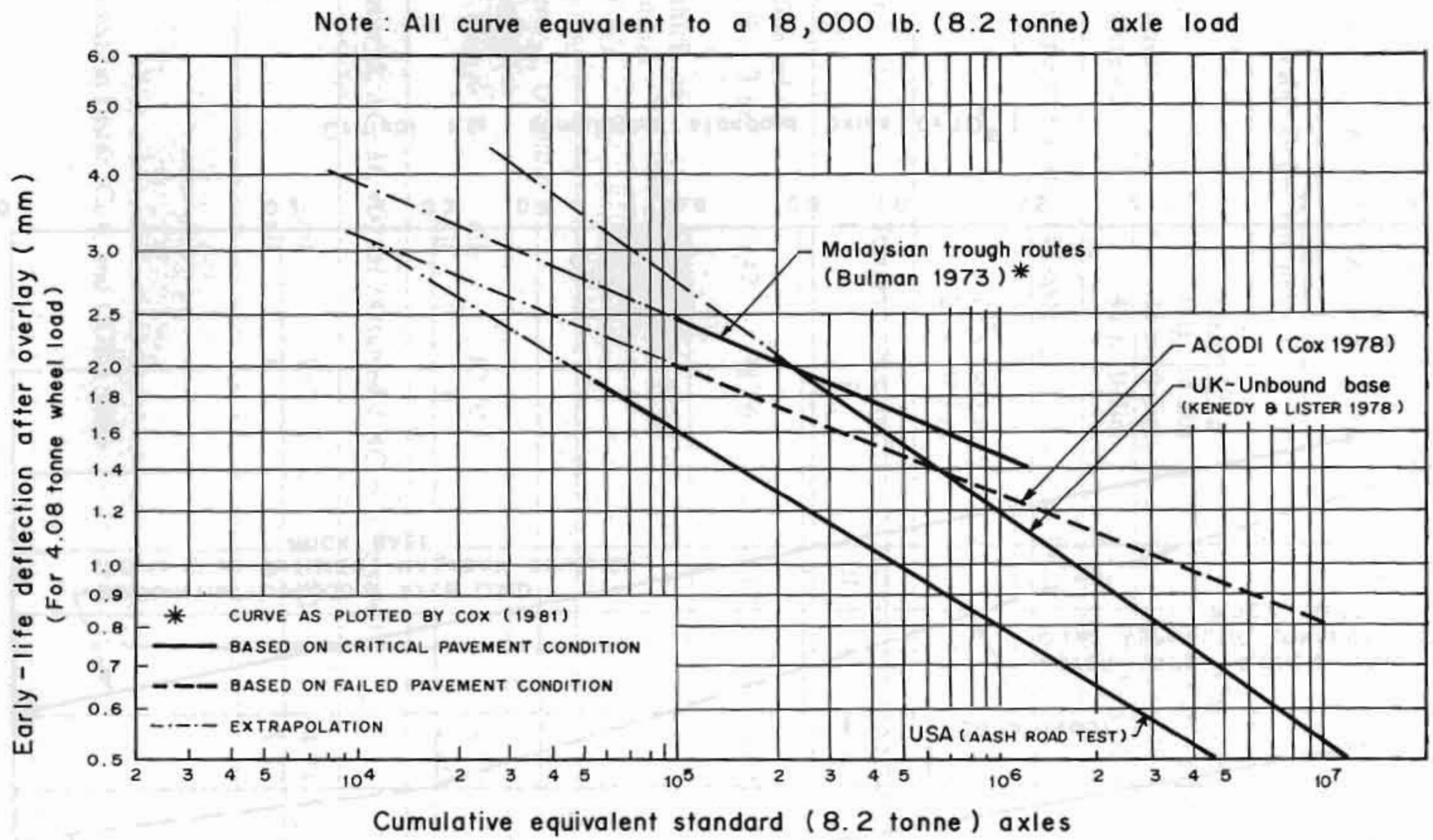
Description	Section I	Section II	Section III	Section IV	Section V
Design define before overlay x 10" - 3 in.	11.42	38.76	42.62	36.22	48.42
Designed thick. of Overlay, inches (Figure 18.31)					
a) *Alternative-1, Granular BC*					
Case - 1	nil	10.70	12	8.70	14.20
Case - 2	nil	9	8	4.40	8.80
b) *Alternative -2, AC, M, 400 ksi (Assumed 1 AC = 3" gravel)*					
Case - 1	nil	3.57	42.90	4.73	
Case - 2	nil	3	2.67	1.47	2.93

Source: Based on Lister, Cox, and CGRA 1981



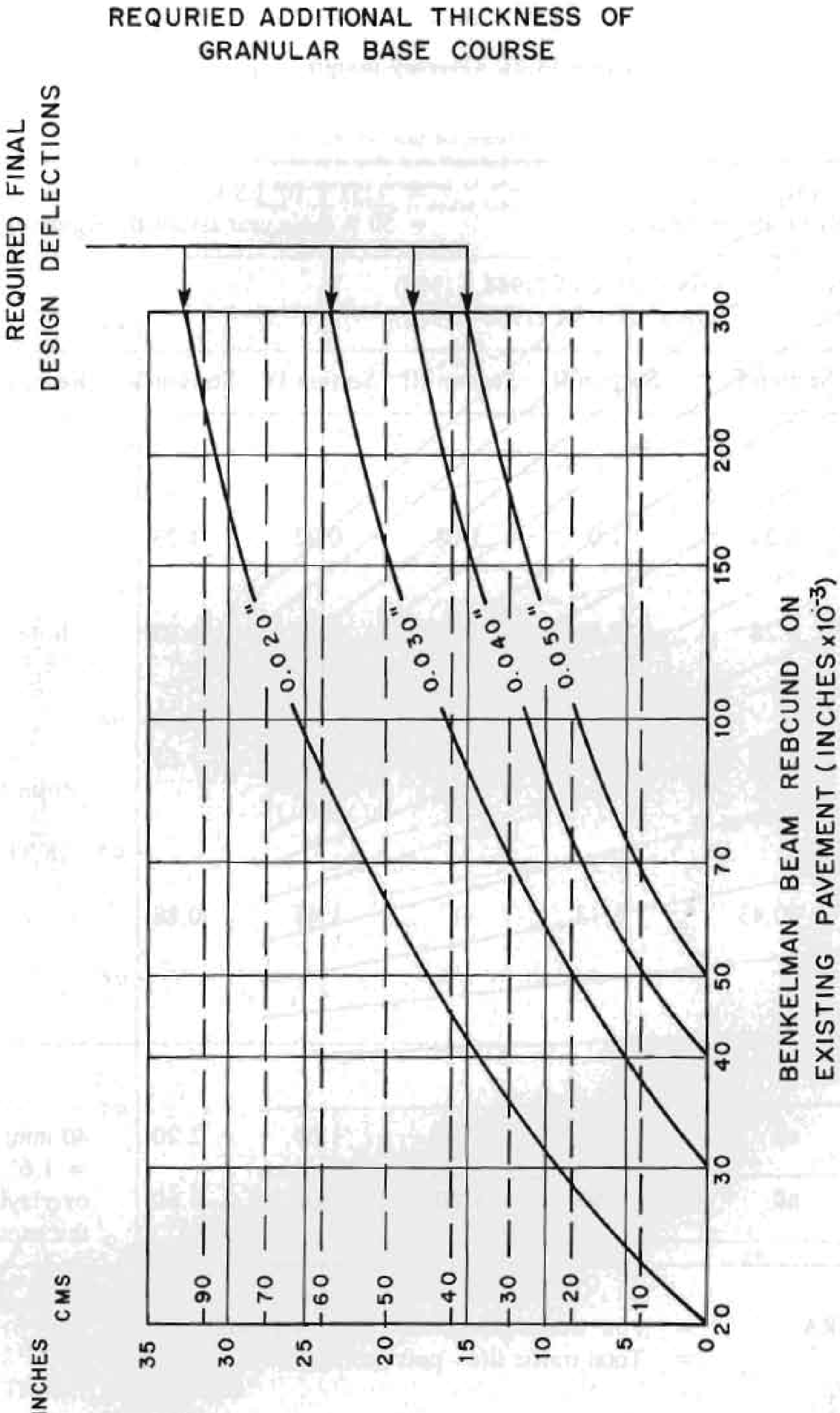
Source: Adapted from Lea and Associates 1977

Fig. 18.29 Design deflection curves



Source: Adapted from Corne 1983

Figure 18.30 Deflection/life relationships for dense plant-mix asphalt overlays



Source: Adapted from Lea and Associates and the Department of Roads (DOR) 1977

Figure 18.31 CGRA overlay design chart by deflection (after reference)

Table 18.21 Overlay design

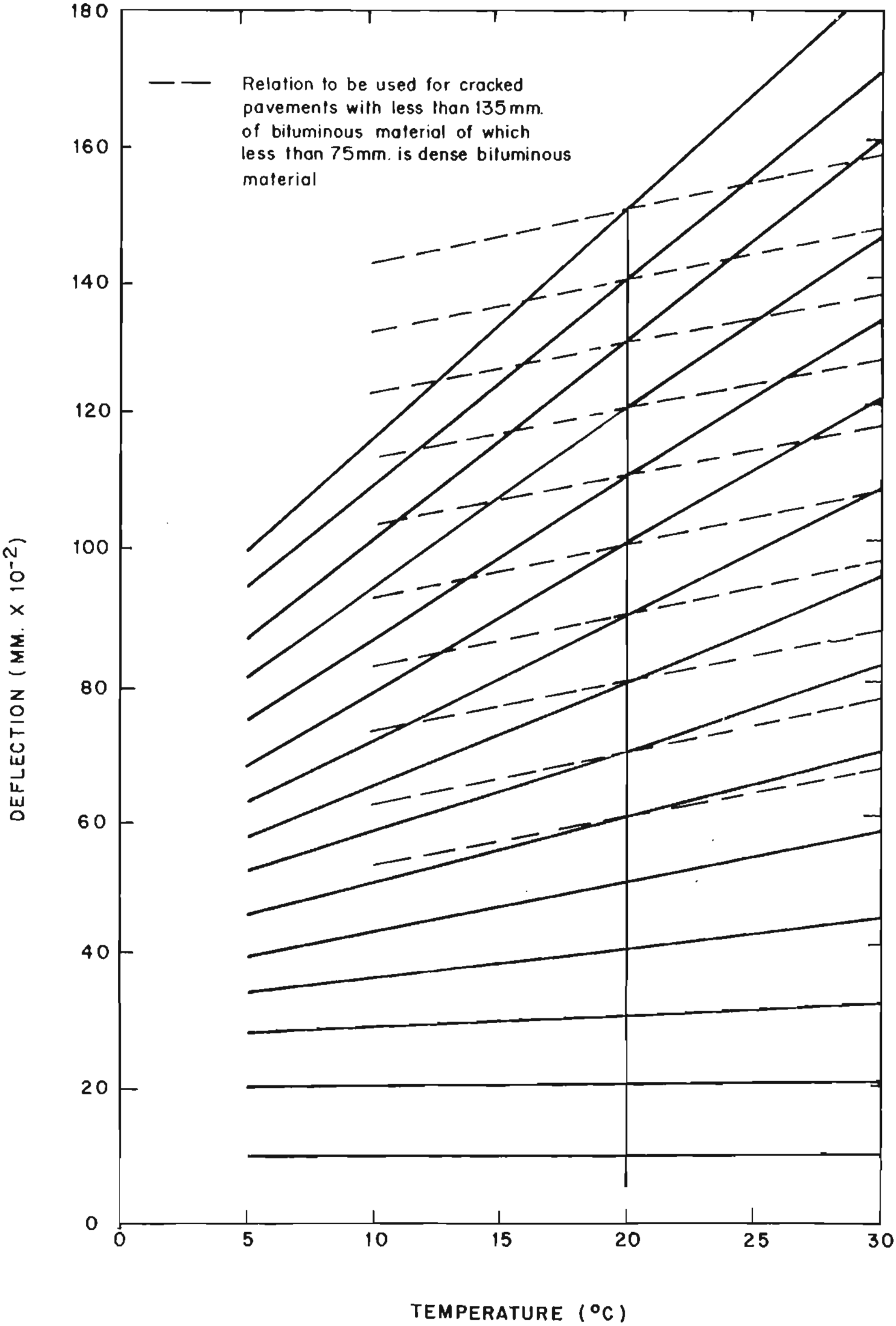
Past traffic	= 1.57×10^6 ESA,
assuming probability of achieving life	= 50 % (base year assumed - beginning of 1988).

Case - 1, traffic :	3.08×10^6 ESA (1988 - 1987)
Case - 2, traffic :	1.28×10^6 ESA (1988 - 1982)

Description	Section-I	Section-II	Section-III	Section-IV	Section-V	Remarks
1) Design deflection for 14R-axle at 30°C mm	0.29	1.0	1.08	0.92	1.23	
Standard defl. at 20°C mm	0.28	0.83	0.92	0.78	1.03	- from Fig. 18.33, 18.34
2) Total life (x 106 ESA)	32.0	2.7	2.05	3.0	1.65	- from Fig. 18.33, 18.34
3) Beginning life (x 106 ESA)	30.43	1.13	.48	1.43	0.88	
Overlay thickness required (inches)						
Alternative - 1 (HRA)						
Case - 1	nil	1.60	1.60	1.60	2.20	40 mm. = 1.6" min. overlaying thickness
Case - 2	nil	1.60	1.60	1.60	1.60	

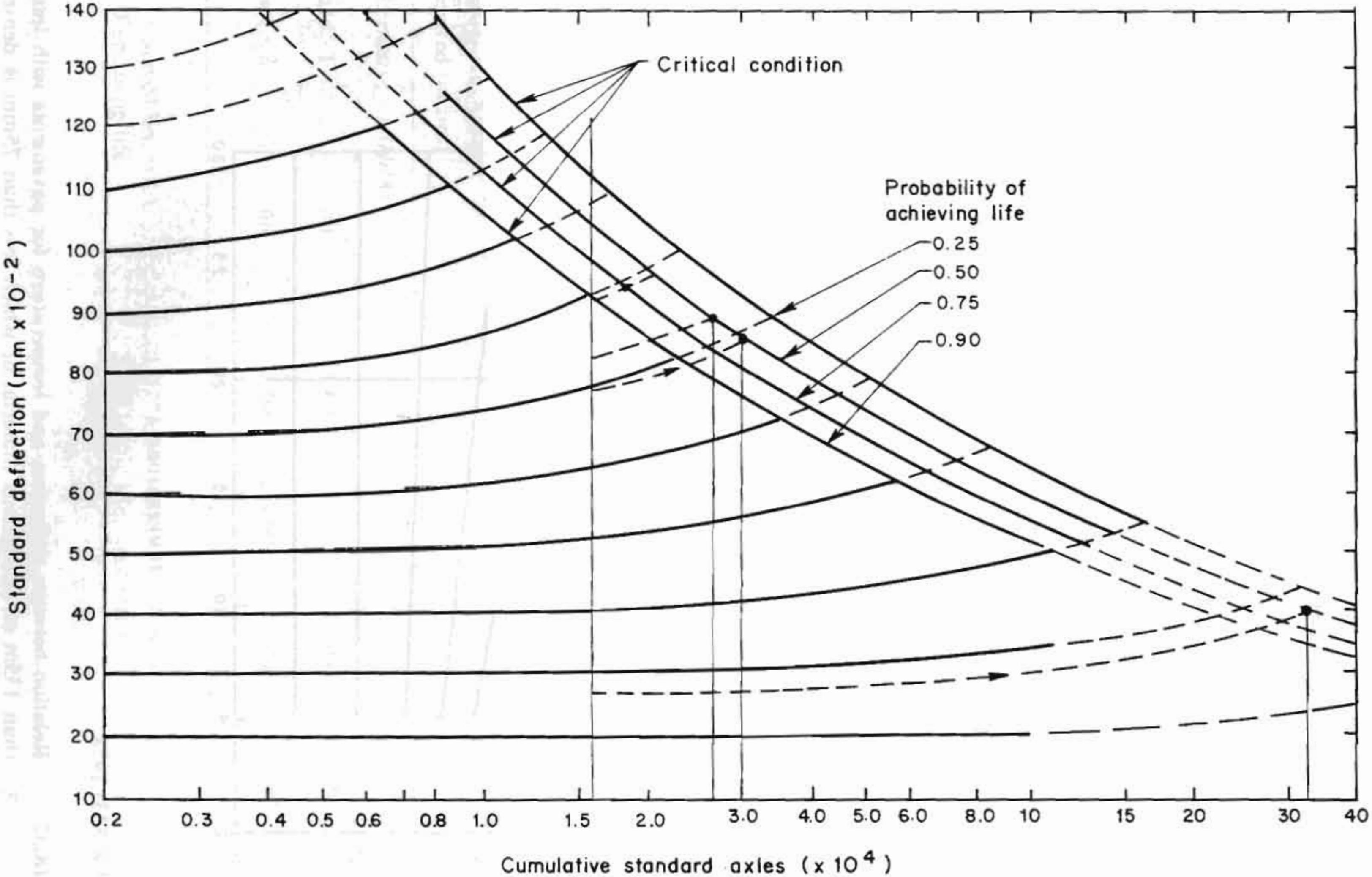
Assumed lin. HRA	=	1 in. dense graded AC. 3 in. of gravel
Remaining life	=	Total traffic life - past traffic.

Source: TRRL Lab Report 833, 1978



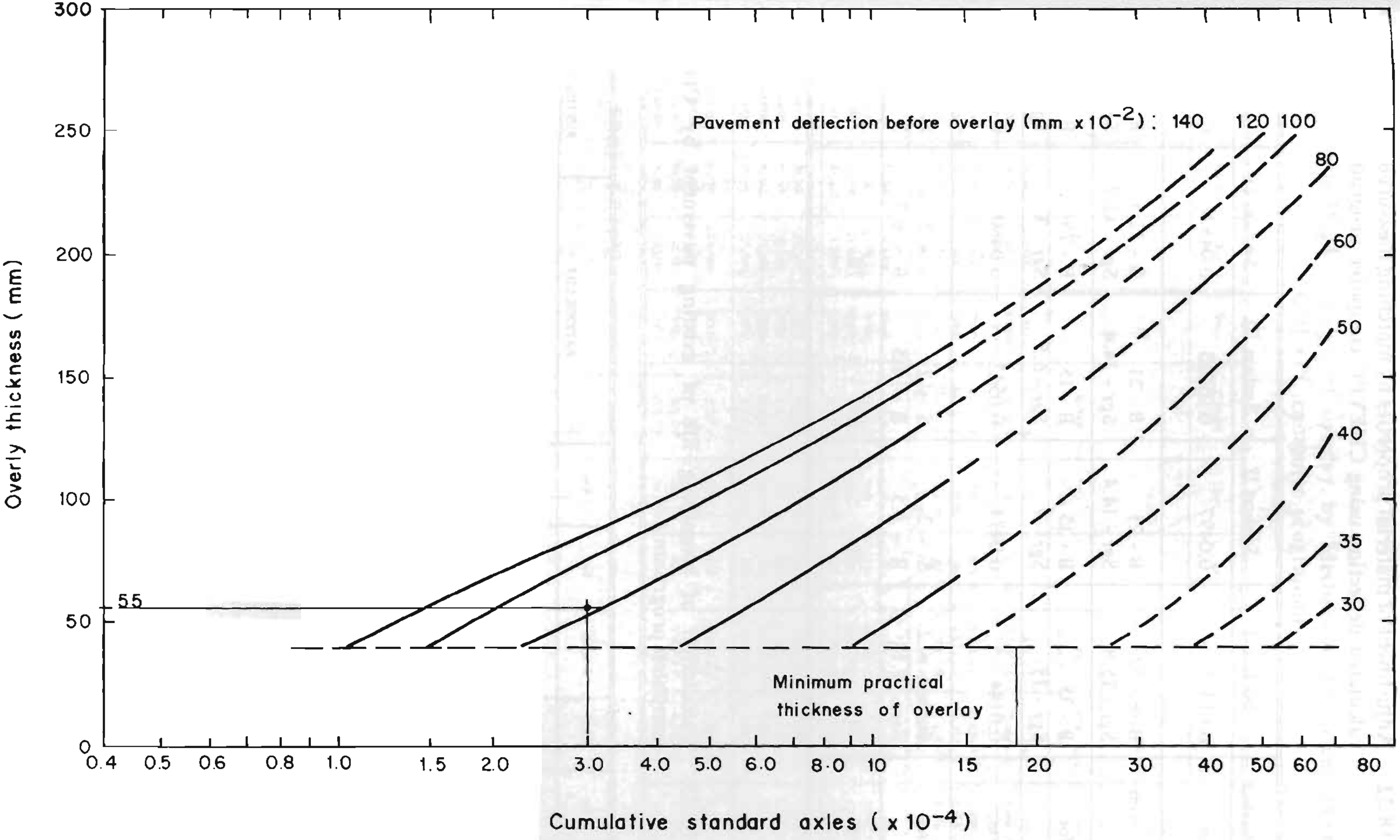
Source: TRRL (LR 833) 1978

Fig 18.32 **Relation between deflection and temperature for pavements with less than 135m of bituminous material of which less than 75mm is dense bituminous material**



Source: TRRL 1978

Fig 18.33 **Relation between standard deflection and life pavements with non-cementing granular road bases**



Source: TRRL (LR 833) 1978

Figure 18.34 Overlay design chart for pavements with non-cementing granular road bases (0.50 probability)

Table 18.22 Adjustment of material properties by matching measured deflection with calculated deflection using CHEVPC computer programme

Pavement Characteristics	Section I	Section II	Section III	Section IV	Section V
Measured Deflection, inches	0.01428	0.0497	0.05315	0.04528	0.0605
Material Properties from field CBR	Base - 22 Sgr - 15.8	B - 21 Sgr - 14.4	B - 21 Sgr - 14.4	B - 22.5 Sgr - 12.7	B - 20.5 Sgr - 12
Adjusted Material for Properties	B - 35 Sgr - 35	B - 15 Sgr - 7	B - 15 Sgr - 6.5	B - 25 Sgr - 7	B - 18 Sgr - 5
Calculated Deflection, inches	0.0144	0.0474	0.0532	0.0460	0.06000
Thickness of Existing Pavement (inches)	Surface = 2 Base 13.18	S = 2 B = 8.5	S = 2 B = 8.5	S = 2 B = 6.7	S = 2 B = 7.9
Assumed Stiffness of Existing Surface, (ksi)	70	70	70	70	70

Table 18.23 Calculation of remaining life of existing pavement by CHEVPC computer programme

Section	Eex.ac.psi	Eb_1	Esgr	T-layers	Eac	Evs	FATIGUE LIFE			RUTTING LIFE	
							Finn model		TRRL mode	Finn model	TRRL model
							A.C.		HPA		
				Inches			10% crack Nf	45% crack Nf	Nf	Nr	Nr
I	100,000	35000	35000	2,13.78	319	302	9.98E+5	1.09E+6	2.11E+5	8.15E+6	4.94E+6
II	70,000	15000	7000	2,11.8	1385	248.30	1.08E+4	1.18E+4	3.71E+2	1.96E+7	1.07E+7
III	70,000	15000	6500	2,8.5	735.60	1958	8.65E+4	9.47E+4	5.70E+3	1.87E+3	3.07E+3
IV	70,000	25000	7000	2,6.7	365	1878	8.69E+5	9.50E+5	1.18E+5	2.25E+3	3.62E+3
V	70,000	18000	5000	2,7.9	566	2288	2.05E+5	2.24E+5	1.77E+4	9.28E+2	1.66E+3

Table 18.24 Design of overlay by Mechanistic-Empirical Method (MEM) using CHEVPC computer programme

Sect-ion	Eac	Eex.ac	Eb_1	Esgr	T-layers	Eac	Evs	FATIGUE LIFE			RUTTING LIFE	
								Finn model		TRRL mode	Finn model	TRRL model
								A.C.		HRA		
	psi				Inches			10% crack Nf	45% crack Nf	Nf	Nr	Nr
I	400000	100,000	35000	35000	2,2,13.78	370	225	1.87E+5	2.05E+5	1.11E+5	3.05E+7	1.58E+7
	400000	100,000	35000	35000	6,2,13.78	174	130.40	2.24E+6	2.46E+6	2.89E+6	3.52E+8	1.36E+8
II	400000	70,000	15000	7000	2,2,11.8	748	918.70	1.85E+4	2.02E+4	5.13E+3	5.55E+4	6.10E+4
	400000	70,000	15000	7000	6,2,11.8	292.30	200.70	4.07E+5	4.45E+5	3.07E+5	5.09E+7	2.48E+7
	400000	70,000	15000	7000	7,2,11.8	205.60	368.60	1.30E+6	1.42E+6	1.41E+6	3.33E+6	2.25E+6
	400000	70,000	15000	7000	8,2,11.8	159.40	248.30	3.00E+6	3.28E+6	4.22E+6	1.96E+7	1.07E+7
III	400000	70,000	15000	6500	2,2,8.5	777	1249	1.63E+4	1.78E+4	4.50E+3	1.40E+4	1.81E+4
	400000	70,000	15000	6500	4,2,8.5	469	810	9.59E+4	9.40E+4	3.99E+4	9.77E+4	1.00E+5
	400000	70,000	15000	6500	6,2,8.5	304	545	3.58E+5	3.91E+5	2.53E+5	5.77E+5	4.80E+5
	400000	70,000	15000	6500	8,2,8.5	164.60	384	2.70E+6	2.95E+6	3.67E+6	2.78E+6	1.91E+6
IV	400000	70,000	25,000	7000	2,2,6.7	572.60	1244.40	4.45E+4	4.07E+4	1.63E+4	1.42E+4	1.84E+4
	400000	70,000	25,000	7000	4,2,6.7	377	826	1.76E+5	1.93E+5	1.02E+5	8.95E+4	9.29E+4
	400000	70,000	25,000	7000	6,2,6.7	256.60	563	6.25E+5	6.94E+5	5.40E+5	4.99E+5	4.22E+5
	400000	70,000	25,000	7000	8,2,6.7	152.40	400.40	3.47E+6	3.80E+6	5.12E+6	2.30E+6	1.62E+6
V	400000	70,000	18,000	5000	2,2,7.9	720	1458	2.10E+4	2.29E+4	6.26E+3	7.00E+3	3.84E+3
	400000	70,000	18,000	5000	4,2,7.9	452	949	9.70E+4	1.06E+5	4.68E+4	4.08E+4	5.37E+4
	400000	70,000	18,000	5000	6,2,7.9	298	400	3.82E+5	4.18E+5	2.83E+5	2.31E+6	1.63E+6
	400000	70,000	18,000	5000	8,2,7.9	165.30	445	2.66E+6	2.19E+6	3.61E+6	1.43E+6	1.07E+6

Source: TRRL, LR1132

Failure Criteria:

Finn,

Fatigue:

TRRL LR1132

$\leq$ 10% logNf = 15.947 - 3.291 x log $\epsilon_t/10^{-6}$ 854 x log MR/1000 Fatigue:

logNf = -9.78 - 4.32 x log ϵ_t

$\leq$ 45° logNf = 15.986 - 3.291 x log $\epsilon_t/10^{-6}$ 854 x log MR/1000

Rutting:

logNr = -7.21 - 3.95 x log ϵ_{vs}

Rutting:

$$Nr = 1.077 \times 10^{18} \left(\frac{1}{10^6 \times \epsilon_{vs}}\right)^{4.4843}$$

Summary of Overlay Designs by Various Methods

Table 18.25 and 18.26 present a summary of thickness of overlay in terms of asphalt concrete of about 400,000 psi modulus. The considerable variation in thickness obtained, from different methods of design, suggests the need for exercise of engineering judgement to select the method that is most appropriate to the actual conditions. For the purpose of this example, the results from the AASHTO Method are suggested for adoption because the assumptions of design and material properties made are likely to represent actual conditions. The deflections and material properties revealed from Table 18.15 are not consistent for meaningful applications. The results from the TRRL LR 833 Method are not recommended because the design charts are applicable to HRA, sub-grade CBR not greater than 15 per cent, and involve past traffic (which is difficult to assess accurately).

The use of equivalencies may be made in order to convert the overlay of AC in terms of other choices such as emulsified AC, DBST + untreated aggregate, or lower stiffness AC. The following equivalencies are suggested for the purpose of this exercise.

1" AC (400 ksi)	=	1.43" Emulsified AC of Asphalt Institute Type II,
	=	3 " untreated gravel,
	=	1.53 " AC of 175 ksi, and
1" DBST	=	1" AC of 175 ksi.

It may be noted that the probability of achieving designed life in methods other than mechanistic are based on 50 per cent. The mechanistic method may be assumed to give a probability of achieving a designed life of more than 90 per cent.

Recommendations

In view of the variation in the design thickness from different methods, it is suggested that more than one method be tried and the selection be made by experienced judgement.A systematic design method should always be adopted rather than *ad hoc* judgements in deciding overlay thickness for pavement improvements.

The decisions concerning the type of overlay depends on available technology, materials, time, and cost economy. Asphalt concrete or hot, rolled asphalt overlay may not always be possible in developing countries. Emulsified concrete may, in some instances, be more desirable because of considerations of fuel for heating. Overlays of DBST with untreated aggregate may sometimes be feasible and desirable because of equipment and material constraints. In the absence of detailed analysis for a specific type of overlay, it should still be possible to design the overlays in terms of asphalt concrete used in most design charts and subsequently adjusting to the desired type can be done by using equivalencies from the available literature.

The field and laboratory tests for material properties, including the deflection test, should not only be adequate in numbers but also be consistent and acceptable to experienced judgement before they are used for design inputs. Each user agency should, wherever possible, try to develop design charts or methods appropriate to their own conditions.

Table 18.25 Summary of designs - Case 1 (10 year design life - 3.08 x 106 ESA)
(Thickness in inches)

S.No.	Design Method	Section I	Section 2	Section 3	Section 4	Section 5
	Type - 1 Dense Graded Asphalt Conc., M_r=400,000 psi					
1.	Component Analysis	3.57	4.50	5.55	6.39	6.03
2.	AASHTO Design Guide 1985	0.2	1.50	2.70	3.6	4.7
3.	Asphalt Institute Deflection	nil	3.70	4	3.8	5.60
4.	Allowable Deflection Criteria, Cox & CGRA	nil	3.57	4	2.90	4.70
5.	TRRL Lab Report 833, 1978	nil	1.6	1.6	1.6	2.2
6.	Mechanistic Method	6.5	8	8.5	8.0	8.5

Table 18.26 Summary of designs - case 2 (5 yr. design life - 1.28 x 106 ESA)
(Thickness in inches)

S. No.	Design Method	Section 1	Section 2	Section 3	Section 4	Section 5
	Type - 1 Dense, Graded Asphalt Conc., M_r=400,000 psi					
1.	Component Analysis	1.87	2.8	3.85	4.59	4.23
2.	AASHTO Design Guide 1985	1.0	0.8			
3.	Asphalt Institute Deflection	nil	3.10	3.20	2.9	4.50
4.	Allowable Deflection Criteria, Lister & CGRA	nil	3	2.70	1.47	2.90
5.	TRRL Lab Report 833, 1978	nil	1.60	1.60	1.60	1.60
6.	Mechanistic Method	nil	7	7.5	7.0	7