Table 5.14 Percent Change in Design Life Due to Variations in Tire Pressure at E_{SG} of 12000 psi and MAAT of 29°C

	W (%)								
	3			5			11		
% Variation in TP / TP Level (psi)	70	90	120	70	90	120	70	90	120
10	-1.1	-0.7	-0.4	-1.1	-5.1	-8.2	-19.3	-13.9	-8.6
-10	1.3	0.8	0.4	1.4	0.8	9.5	27.0	17.6	11.2
20	-2.1	-1.3	-0.8	-2.0	-15.9	-15.1	-33.7	-24.9	-16.[illegible]
-20	2.8	1.7	0.9	2.9	1.6	20.6	65.9	40.4	24.1
40	-3.7	-2.3	-1.3	-3.6	-32.2	-26.7	-95.2	-41.2	-29.3
-40	6.9	3.8	1.9	7.0	3.7	49.5	107.1	111.8	58.6

Table 5.15 Percent Change in Design Life Due to Variations in Tire Pressure at E_{SG} of 22500 psi and MAAT of 29°C

	W (%)								
	3			5			11		
% Variation in TP / TP Level (psi)	70	90	120	70	90	120	70	90	120
10	-4.2	-12.0	-8.0	-17.3	-12.4	-8.2	-19.6	-14.1	-9.4
-10	1.4	14.8	9.3	23.2	15.5	9.7	27.2	17.8	10.5
20	-18.7	-21.8	-14.9	-30.3	-22.6	-15.4	-33.8	-25.1	-17.1
-20	3.1	33.5	20.2	55.5	35.2	21.1	66.4	40.7	23.8
40	-39.1	-36.6	-26.3	-48.5	-37.8	-27.1	-53.1	-41.5	-29.8
-40	7.5	64.2	48.4	114.8	94.0	50.8	218.3	113.1	58.6

Table 5.16 Percent Change in Design Life Due to Variations in Volume of Air Voids at E_{SG} of 4500 psi and MAAT of 29°C

	Tire Pressure (psi)								
	70			90			120		
% Variation in VV \ VV Level (%)	3	5	11	3	5	11	3	5	11
6.0	-1.1	-1.1	-1.3	0.0	0.0	0.0	-1.2	-1.2	-8.5
-6.0	1.1	1.1	1.3	1.1	1.2	1.3	1.2	1.2	8.5
12.5	-1.1	-1.1	-1.3	-1.1	-1.2	-1.3	-2.3	-2.4	-15.3
-12.5	2.2	2.3	1.3	2.3	2.4	2.6	1.2	1.2	20.3
25.0	-3.3	-3.4	-2.5	-3.4	-3.5	-2.6	-3.5	-3.6	-27.1
-25.0	4.4	3.4	2.5	4.5	3.5	3.9	3.5	3.6	32.2

Table 5.17 Percent Change in Design Life Due to Variations in Volume of Air Voids at E_{SG} of 12000 psi and MAAT of 29°C

	Tire Pressure (psi)								
	70			90			120		
% Variation in VV \ VV Level (%)	3	5	11	3	5	11	3	5	11
6.0	-0.8	-0.6	-6.2	-0.7	-10.2	-6.9	-0.8	-17.5	-7.8
-6.0	0.8	0.7	7.3	0.8	0.7	8.6	0.8	23.0	9.5
12.5	-1.5	-1.4	-11.6	-1.5	-24.4	-13.1	-13.4	-31.1	-14.7
-12.5	1.5	1.5	16.2	1.6	1.4	18.4	1.6	53.4	20.7
25.0	-3.0	-2.7	-19.7	-2.9	-44.5	-22.9	-243.6	-50.2	-25.0
-25.0	3.2	3.0	38.9	3.3	3.0	44.1	3.3	77.2	49.1

Table 5.18 Percent Change in Design Life Due to Variations in Volume of Air Voids at E_{SG} of 22500 psi and MAAT of 29°C

VV Level (%) / % Variation in VV	Tire Pressure (psi) 70			90			120		
	3	5	11	3	5	11	3	5	11
6.0	-9.6	-16.2	-6.4	-21.9	-16.8	-7.1	-22.4	-17.3	-7.7
-6.0	0.7	21.0	7.4	30.5	21.9	8.4	31.4	22.7	8.8
12.5	-27.6	-28.8	-11.8	-37.9	-29.9	-13.1	-38.7	-30.8	-14.4
-12.5	1.5	48.6	16.2	60.1	50.8	18.0	76.2	52.8	19.9
25.0	-51.3	-46.8	-20.1	-58.8	-48.4	-22.7	-59.9	-49.7	-25.4
-25.0	3.0	105.3	39.1	62.6	142.6	43.6	197.8	149.2	47.5

Table 5.19 Percent Change in Design Life Due to Variations in the Subgrade Modulus at VV of 3% and MAAT of 29°C

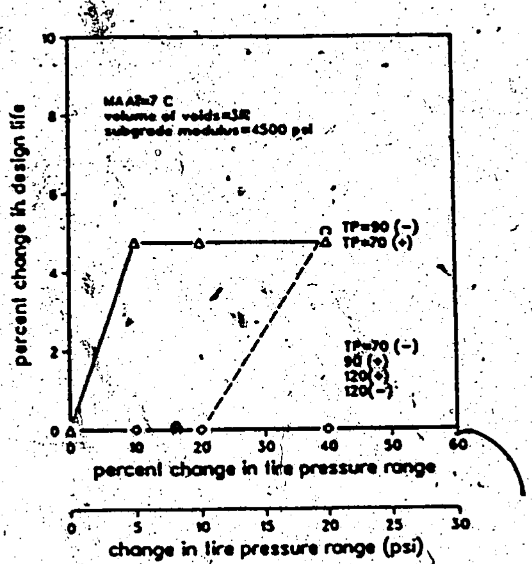
E_{SG} Level (psi) / % Variation in E_{SG}	Tire Pressure (psi) 70			90			120		
	4500	12000	22500	4500	12000	22500	4500	12000	22500
1	9.9	4.2	2.5	10.2	4.3	0.7	9.3	4.2	0.5
-1	-8.8	-4.1	-2.4	-9.1	-4.1	-0.5	-9.3	-4.0	-0.5
5	54.9	22.3	12.8	54.5	22.3	2.7	53.5	22.2	2.6
-5	-40.7	-19.2	-11.7	-39.8	-19.2	-2.8	-40.7	-19.2	-2.6
20	328.6	109.8	27.6	327.3	109.7	10.7	324.4	68.4	10.4
-20	-96.7	-61.7	-40.9	-26.6	-61.5	-11.3	-96.5	-61.5	-10.9

Table 5.20 Percent Change in Design Life Due to Variations in the Subgrade Modulus at VV of 5% and MAAT of 29°C

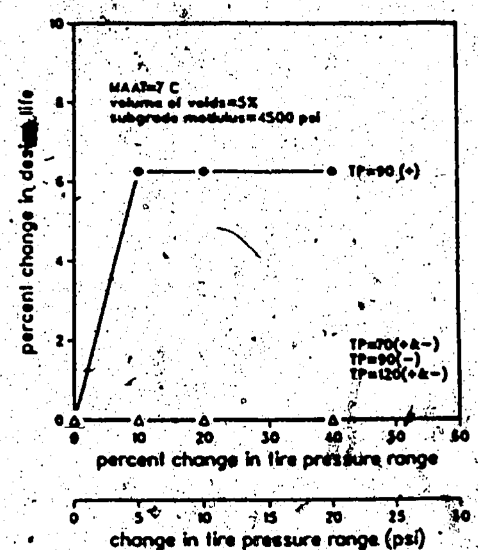
	Tire Pressure (psi)								
	70			90			120		
% Variation in E_{SG} / E_{SG} Level (psi)	4500	12000	22500	4500	12000	22500	4500	12000	22500
1	10.2	4.2	0.6	10.6	4.2	0.6	9.6	1.0	0.6
-1	-9.1	-4.1	-0.6	-9.4	-4.1	-0.5	-9.6	-1.0	-0.6
5	54.5	22.4	2.8	55.3	13.6	2.7	54.2	4.9	2.7
-5	-39.8	-19.3	-2.9	-40.0	-19.3	-2.7	-41.0	-5.1	-2.7
20	329.5	110.1	11.2	329.4	29.7	10.9	326.5	19.2	10.6
-20	-96.6	-61.7	-11.7	-96.5	-61.6	-11.4	-96.4	-21.3	-11.0

Table 5.21 Percent Change in Design Life Due to Variations in the Subgrade Modulus at VV of 11% and MAAT of 29°C

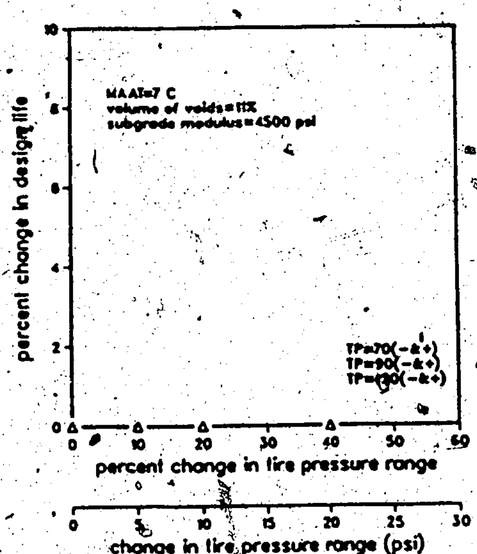
	Tire Pressure (psi)								
	70			90			120		
% Variation in E_{SG} / E_{SG} Level (psi)	4500	12000	22500	4500	12000	22500	4500	12000	22500
1	10.0	1.2	0.5	10.4	1.2	0.5	1.7	1.7	0.6
-1	-8.8	-1.2	-0.6	-9.1	-1.2	-0.5	-3.4	-0.9	-0.6
5	55.0	5.4	2.8	55.8	5.3	2.9	13.6	5.2	2.8
-5	-41.3	-5.6	-2.9	-40.3	-5.3	-2.9	-23.7	-5.2	-2.8
20	333.8	21.0	11.0	140.3	20.4	11.0	49.2	20.7	11.1
-20	-96.3	-22.9	-11.8	-96.1	-22.4	-11.7	-94.9	-21.6	-11.6



(a) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi, Volume of Air Voids of 3% and MAAT of 7°C

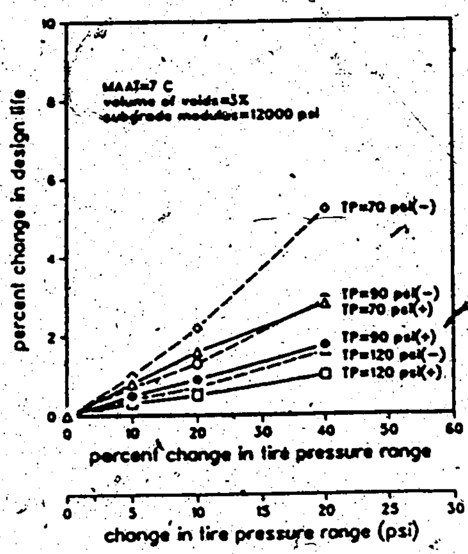


(b) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi, Volume of Air Voids of 5% and MAAT of 7°C



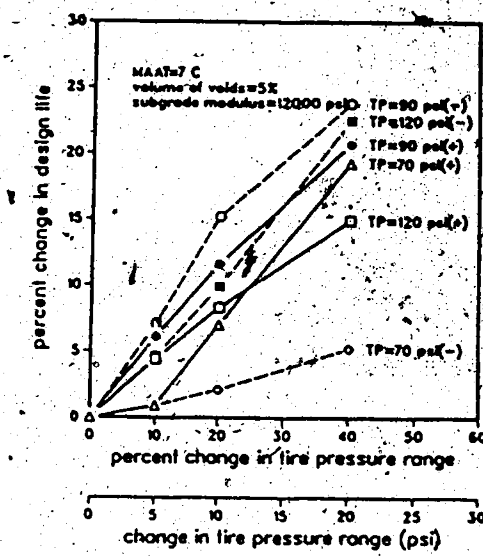
(c) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi, Volume of Air Voids of 11% and MAAT of 7°C

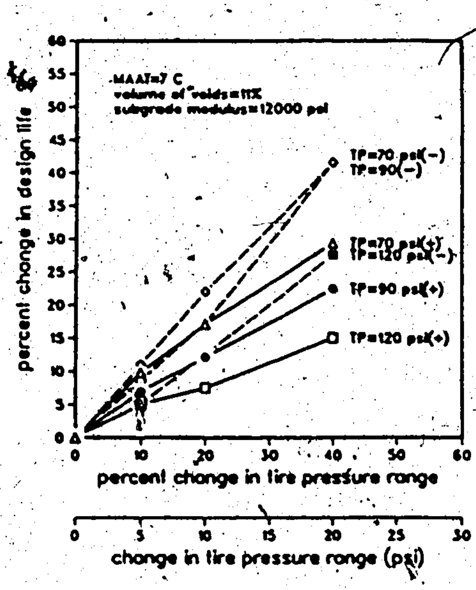
Fig. 5.1 Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi and MAAT of 7°C



(a) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi, Volume of Air Voids of 3% and MAAT of 7°C

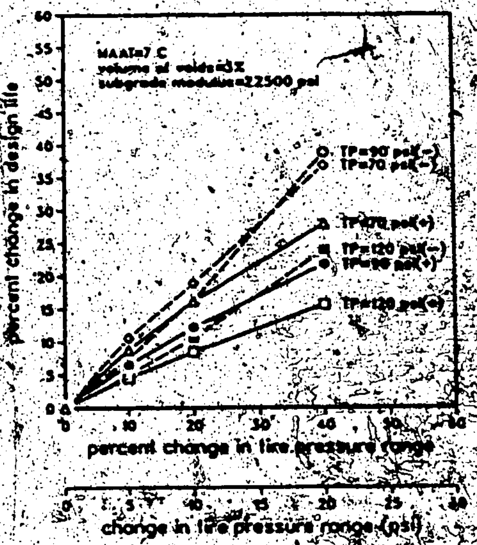
(b) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi, Volume of Air Voids of 5% and MAAT of 7°C



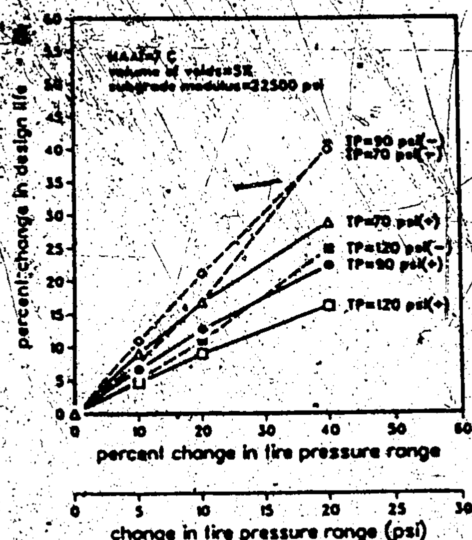


(c) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi, Volume of Air Voids of 11% and MAAT of 7°C

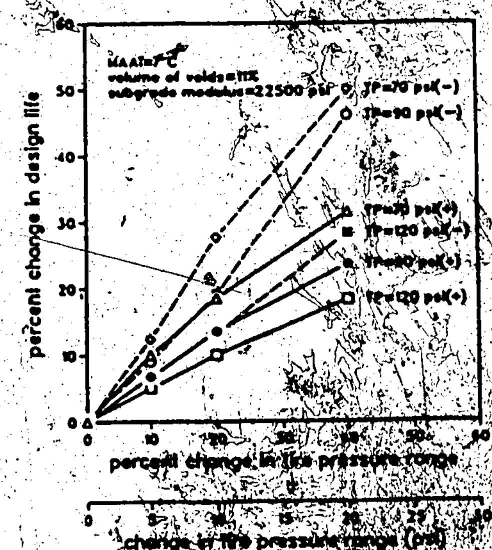
Fig. 5.2 Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi and MAAT of 7°C



(a) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi, Volume of Air Voids of 3% and MAAT of 7°C

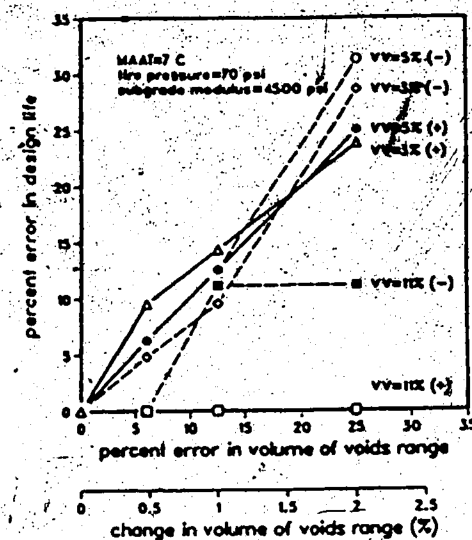


(b) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi, Volume of Air Voids of 5% and MAAT of 7°C

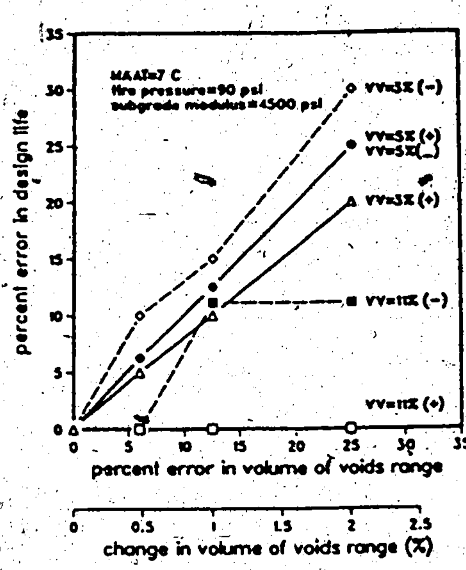


(c) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi, Volume of Air Voids of 11% and MAAT of 7°C

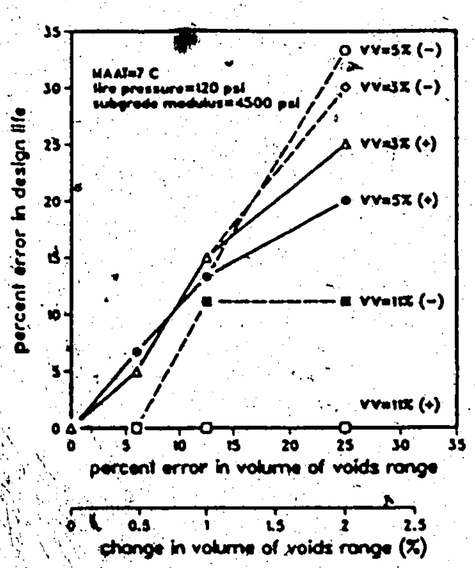
Fig. 5.3 Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi and MAAT of 7°C



(a) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 4500 psi, Tire Pressure of 70 psi, and MAAT of 7°C

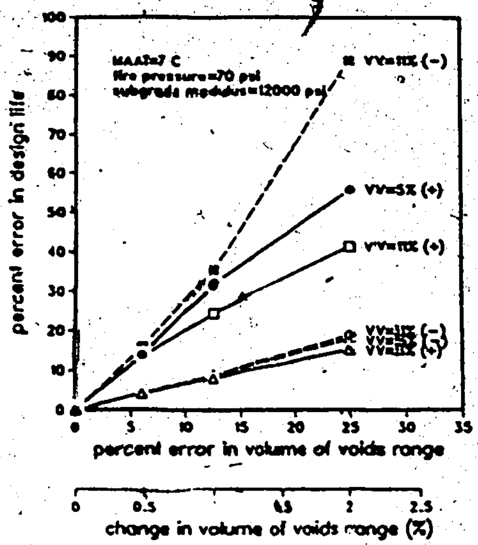


(b) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 4500 psi, Tire Pressure of 90 psi and MAAT of 7°C



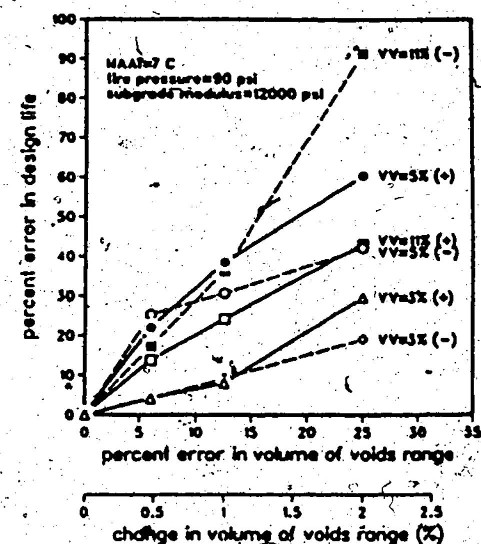
(c) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 4500 psi, Tire Pressure of 120 psi and MAAT of 7°C

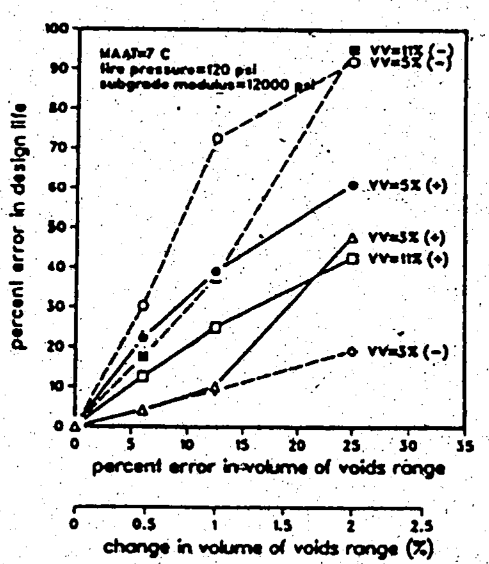
Fig. 5.4 Effect of Volume of Voids Variations on Design Life for Subgrade Modulus of 4500 psi and MAAT of 7°C.



(a) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 12000 psi, Tire Pressure of 70 psi, and MAAT of 7°C

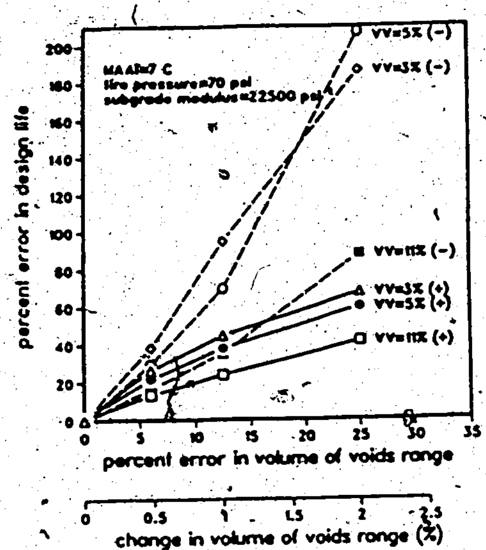
(b) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 12000 psi, Tire Pressure of 90 psi and MAAT of 7°C



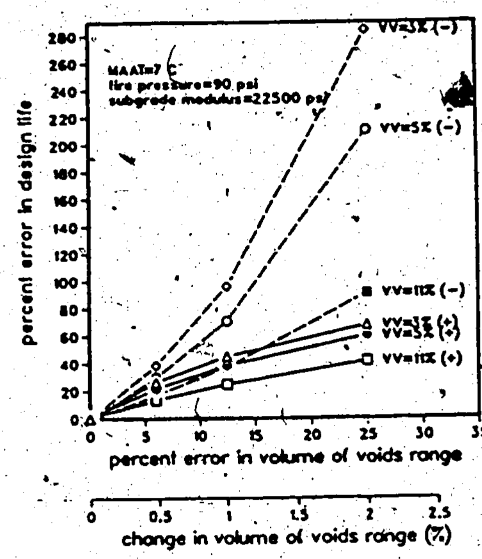


(c) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 12000 psi, Tire Pressure of 120 psi and MAAT of 7°C

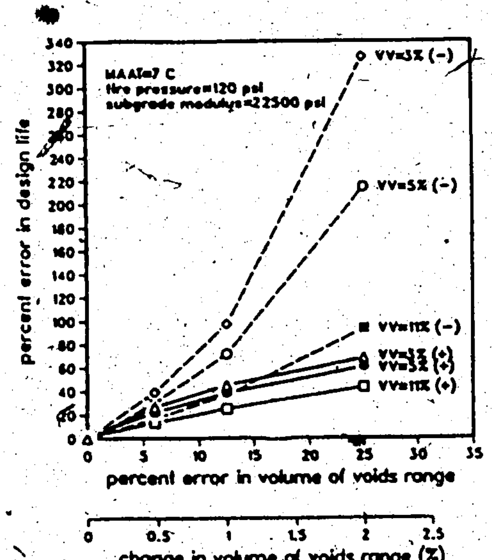
Fig. 5.5 Effect of Volume of Voids Variations on Design Life for Subgrade Modulus of 12000 psi and MAAT of 7°C



(a) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 22500 psi, Tire Pressure of 70 psi, and MAAT of 7°C

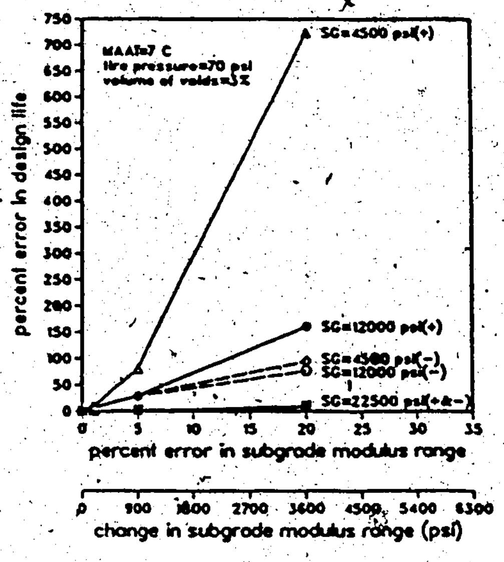


(b) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 22500 psi, Tire Pressure of 90 psi and MAAT of 7°C

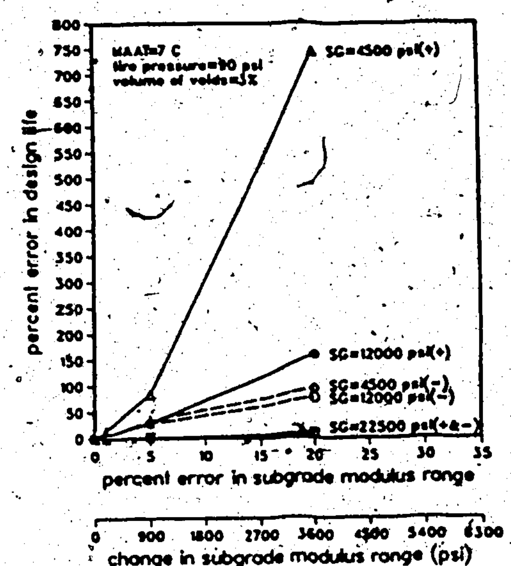


(c) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 22500 psi, Tire Pressure of 120 psi and MAAT of 7°C

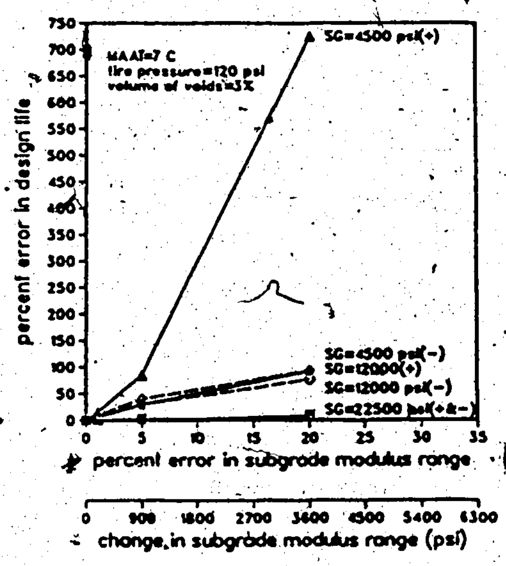
Fig. 5.6 Effect of Volume of Voids Variations on Design Life for Subgrade Modulus of 22500 psi and MAAT of 7°C



(a) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3%, Tire Pressure of 70 psi, and MAAT of 7°C

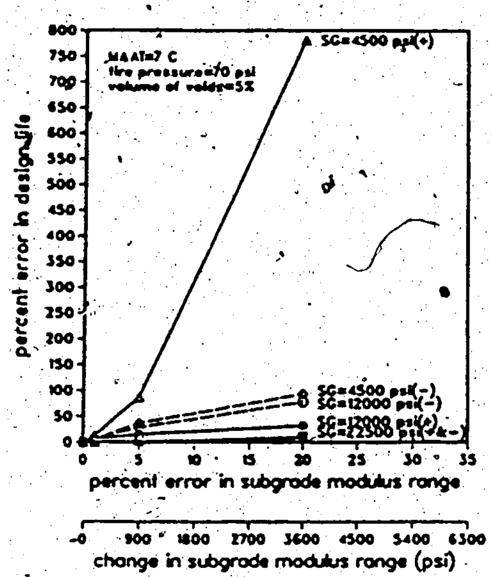


(b) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3%, Tire Pressure of 90 psi, and MAAT of 7°C



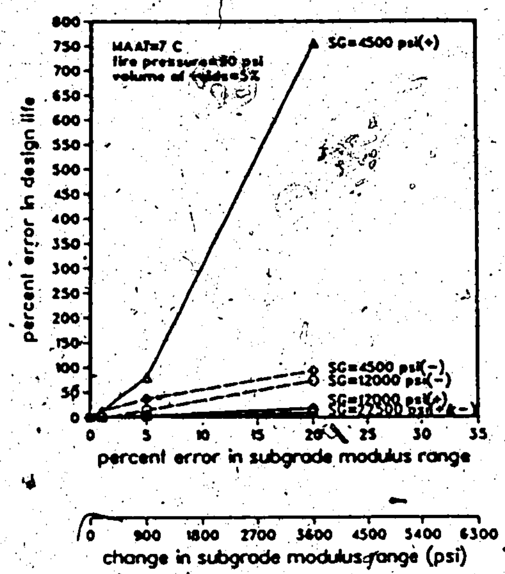
(c) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3%, Tire Pressure of 120 psi, and MAAT of 7°C

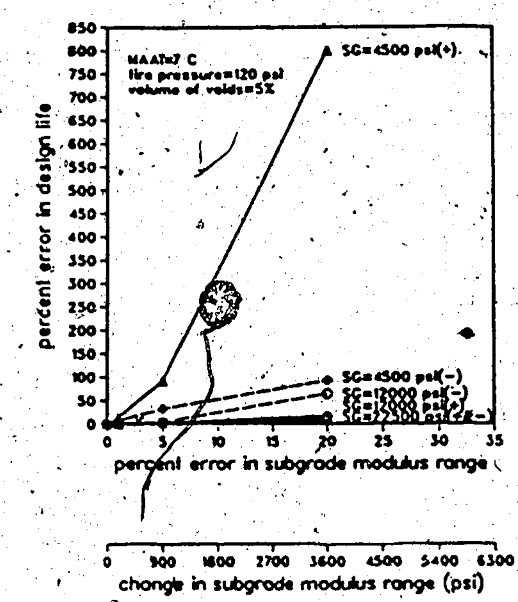
Fig. 5.7 Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3% and MAAT of 7°C



(a) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5%, Tire Pressure of 70 psi, and MAAT of 7°C

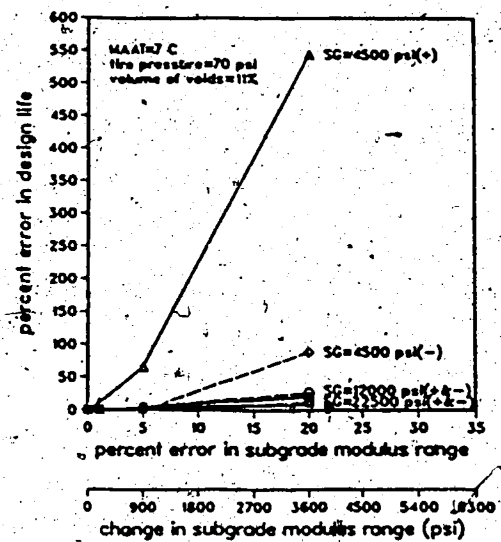
(b) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5%, Tire Pressure of 90 psi, and MAAT of 7°C



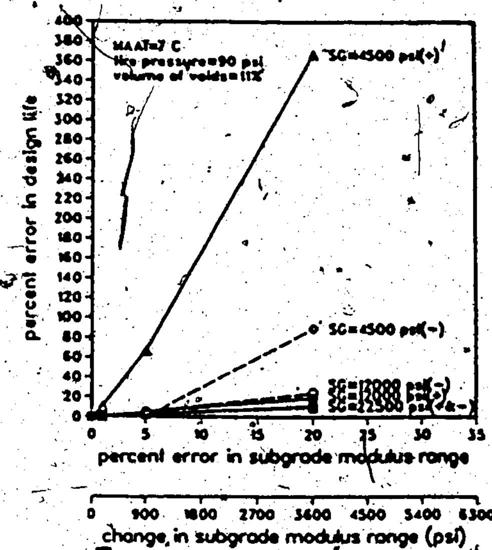


(c) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5%, Tire Pressure of 120 psi, and MAAT of 7°C

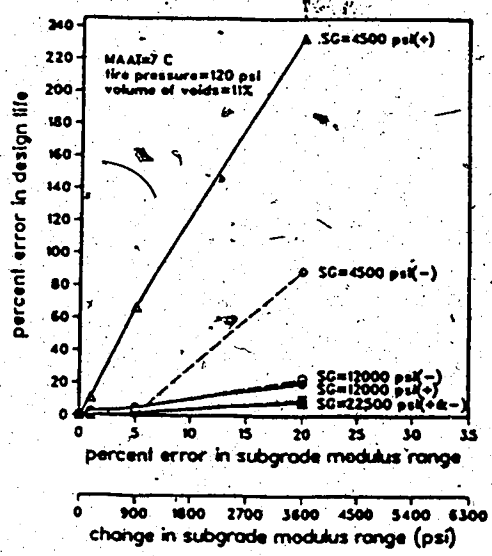
Fig. 5.8 Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5% and MAAT of 7°C



(a) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11%, Tire Pressure of 70 psi, and MAAT of 7°C

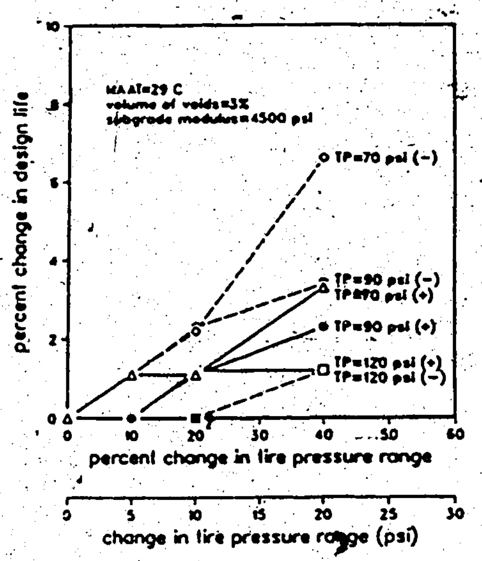


(b) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11%, Tire Pressure of 90 psi, and MAAT of 7°C



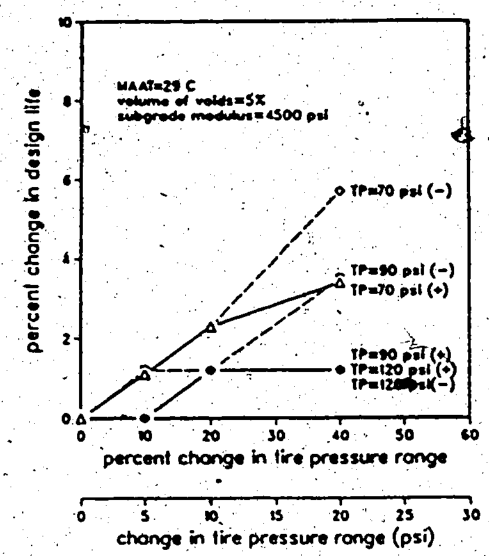
(c) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11%, Tire Pressure of 120 psi, and MAAT of 7°C

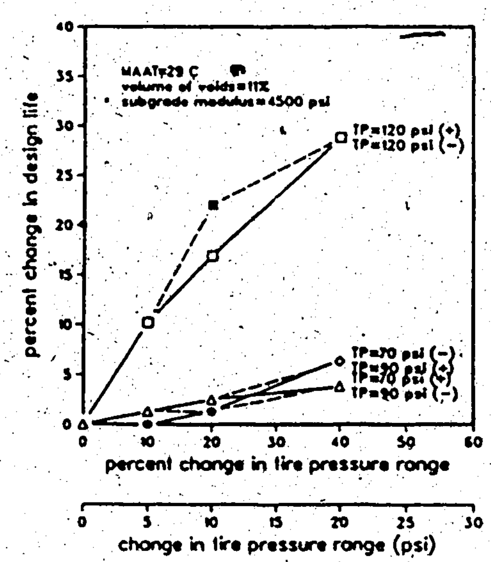
Fig. 5.9 Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11% and MAAT of 7°C



(a) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi, Volume of Air Voids of 3% and MAAT of 29°C.

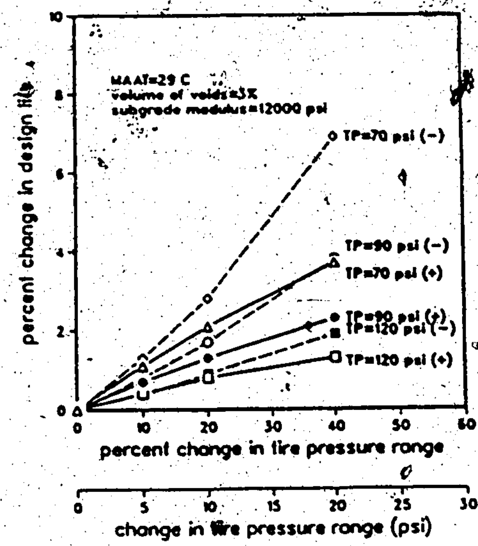
(b) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi, Volume of Air Voids of 5% and MAAT of 29°C



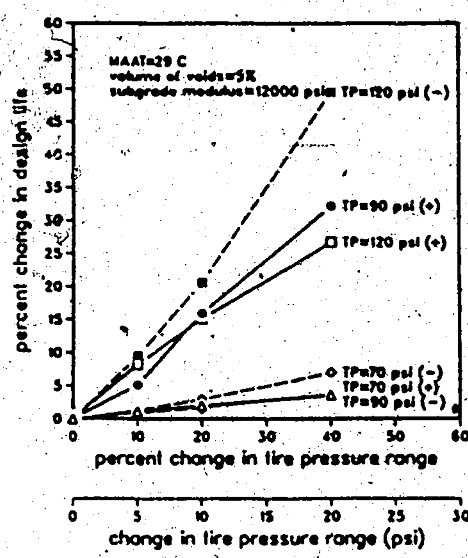


(c) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi, Volume of Air Voids of 11% and MAAT of 29°C

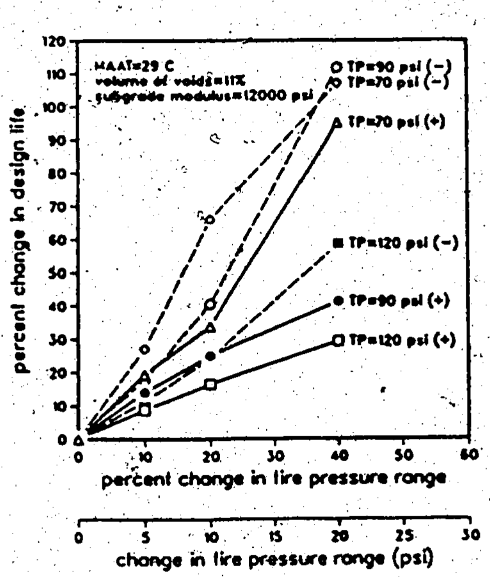
Fig. 5.10 Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 4500 psi and MAAT of 29°C



(a) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi, Volume of Air Voids of 3% and MAAT of 29°C

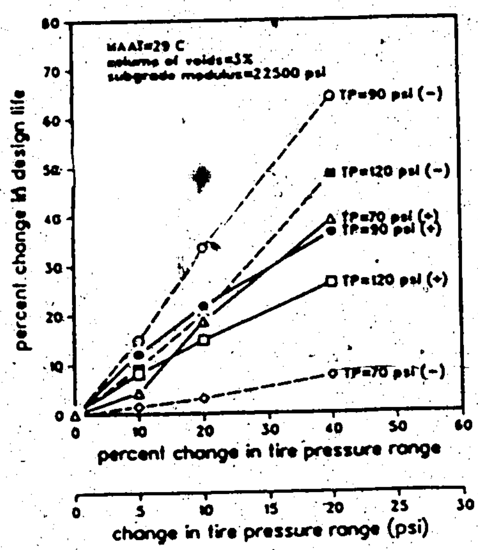


(b) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi, Volume of Air Voids of 5% and MAAT of 29°C



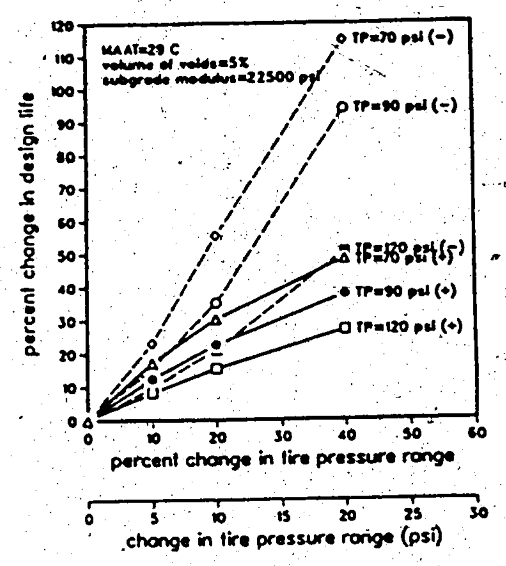
(c) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi, Volume of Air Voids of 11% and MAAT of 29°C

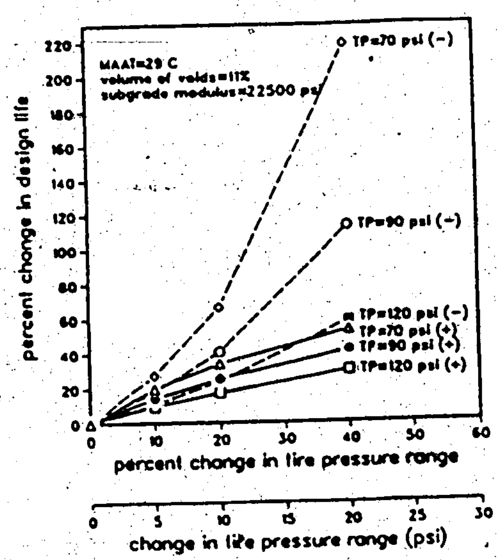
Fig. 5.11 Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 12000 psi and MAAT of 29°C



(a) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi, Volume of Air Voids of 3% and MAAT of 29°C

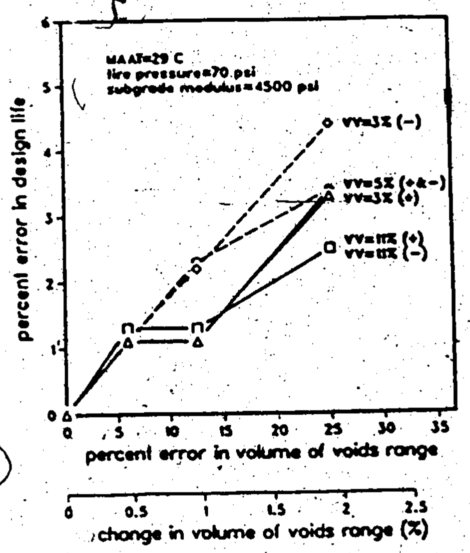
(b) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi, Volume of Air Voids of 5% and MAAT of 29°C



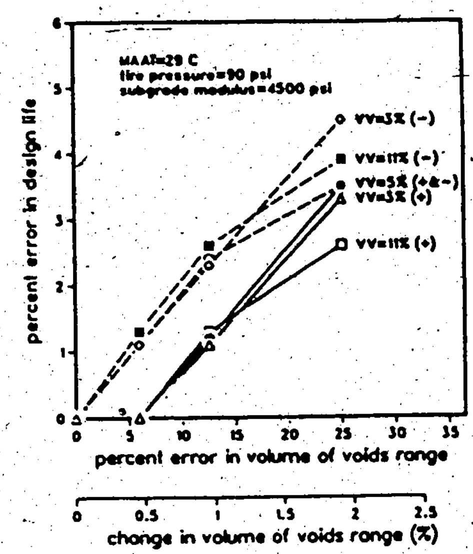


(c) Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi, Volume of Air Voids of 11% and MAAT of 29°C

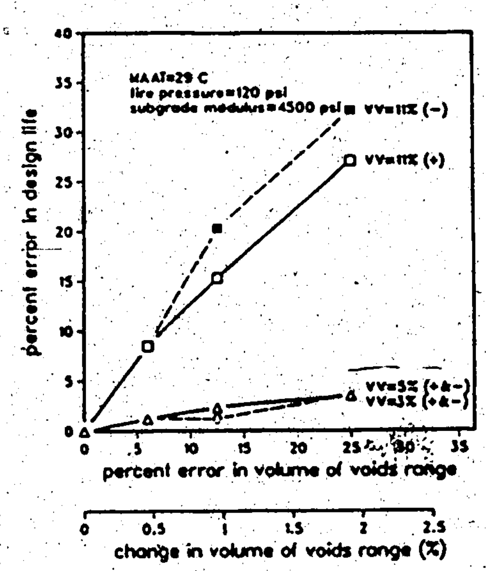
Fig. 5.12 Effect of Tire Pressure Variations on Design Life for Subgrade Modulus of 22500 psi and MAAT of 29°C



(a) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 4500 psi, Tire Pressure of 70 psi, and MAAT of 29°C

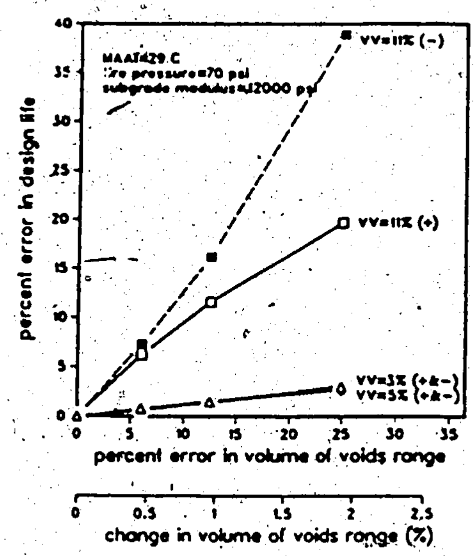


(b) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 4500 psi, Tire Pressure of 90 psi and MAAT of 29°C

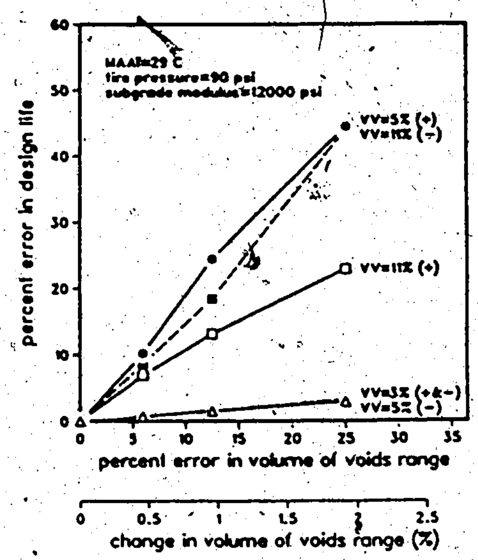


(c) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 4500 psi, Tire Pressure of 120 psi and MAAT of 29°C

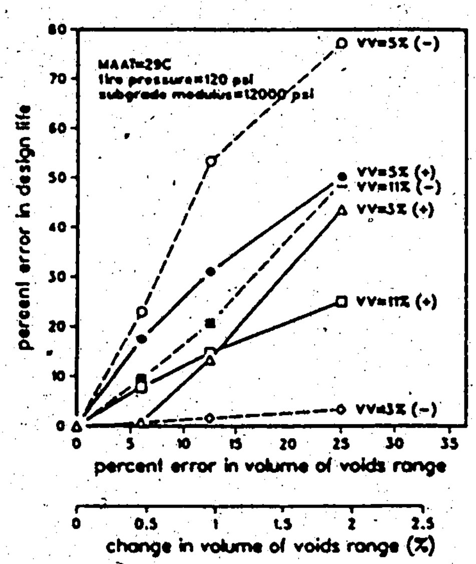
Fig. 5.13 Effect of Volume of Voids Variations on Design Life for Subgrade Modulus of 4500 psi and MAAT of 29°C



(a) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 12000 psi, Tire Pressure of 70 psi, and MAAT of 29°C

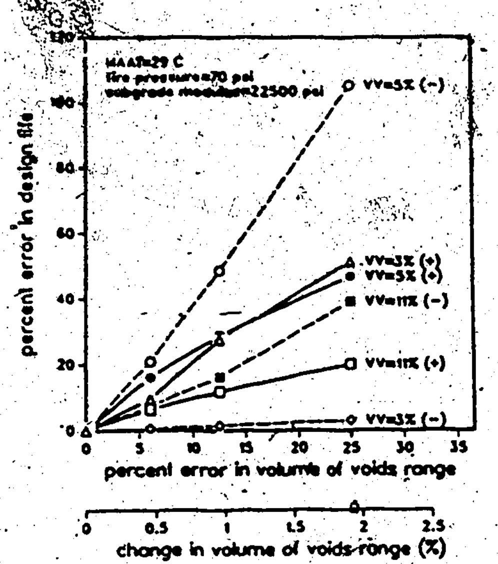


(b) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 12000 psi, Tire Pressure of 90 psi and MAAT of 29°C



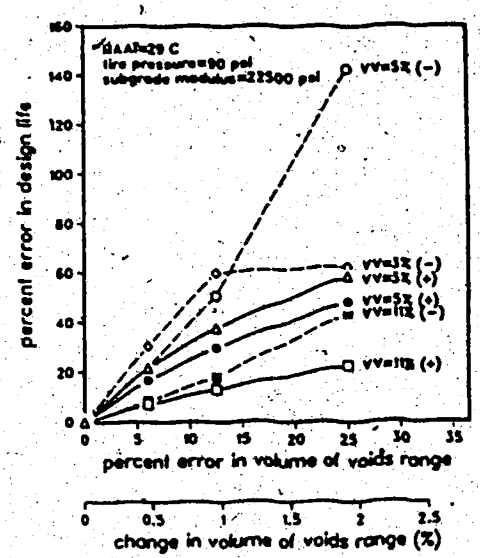
(c) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 12000 psi, Tire Pressure of 120 psi and MAAT of 29°C

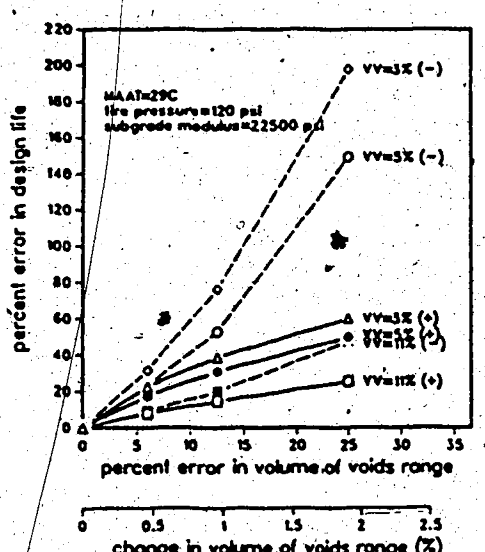
Fig. 5.14 Effect of Volume of Voids Variations on Design Life for Subgrade Modulus of 12000 psi and MAAT of 29°C



(a) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 22500 psi, Tire Pressure of 70 psi, and MAAT of 29°C

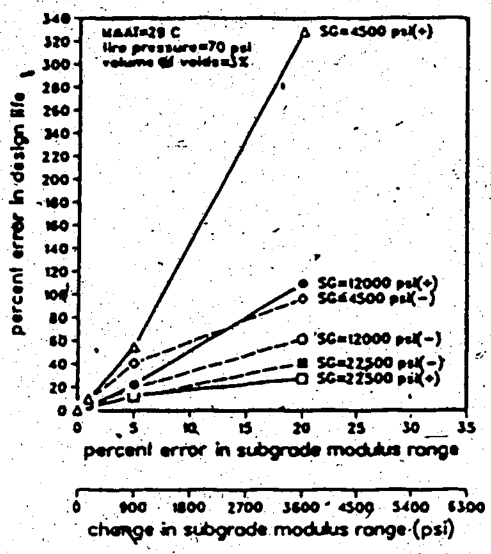
(b) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 22500 psi, Tire Pressure of 90 psi and MAAT of 29°C





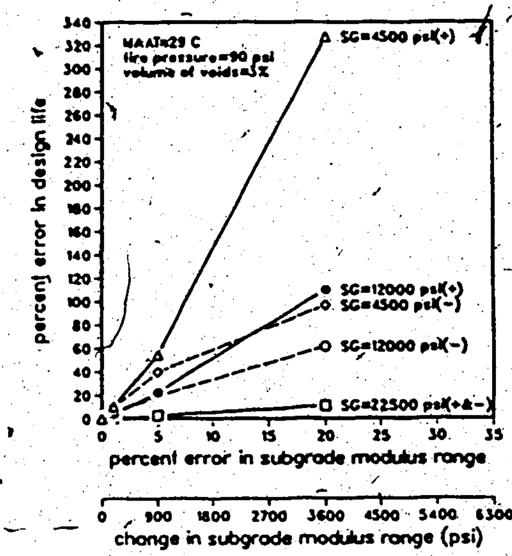
(c) Effect of Volume of Air Voids Variations on Design Life for Subgrade Modulus of 22500 psi, Tire Pressure of 120 psi and MAAT of 29°C

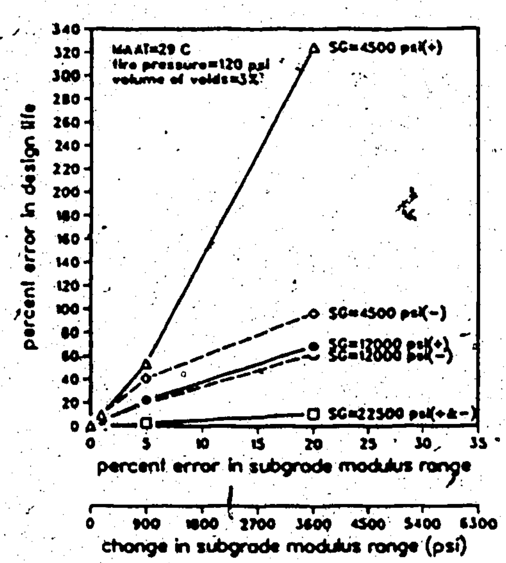
Fig. 5.15 Effect of Volume of Voids Variations on Design Life for Subgrade Modulus of 22500 psi and MAAT of 29°C



(a) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3%, Tire Pressure of 70 psi, and MAAT of 29°C

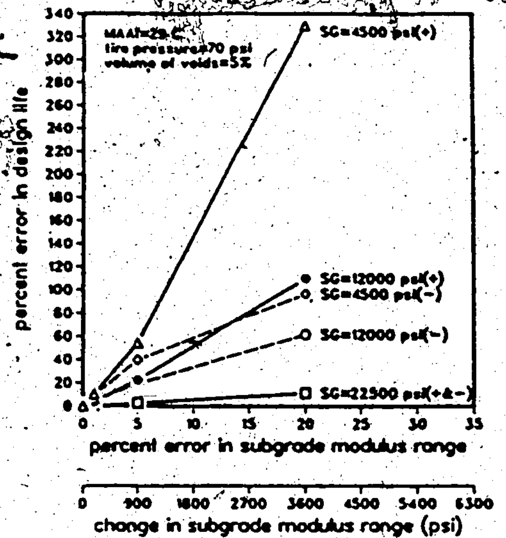
(b) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3%, Tire Pressure of 90 psi, and MAAT of 29°C



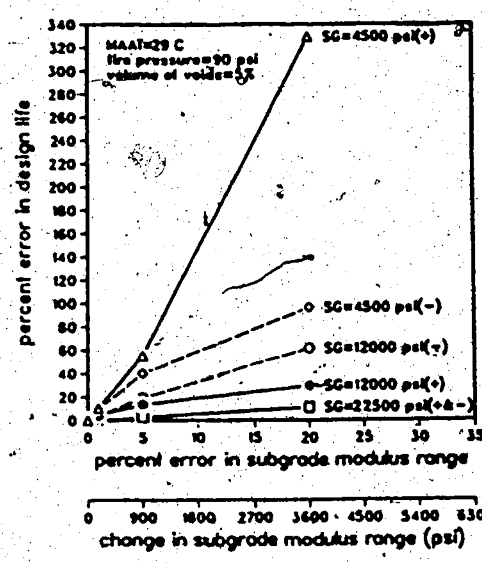


(c) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3%, Tire Pressure of 120 psi, and MAAT of 29°C

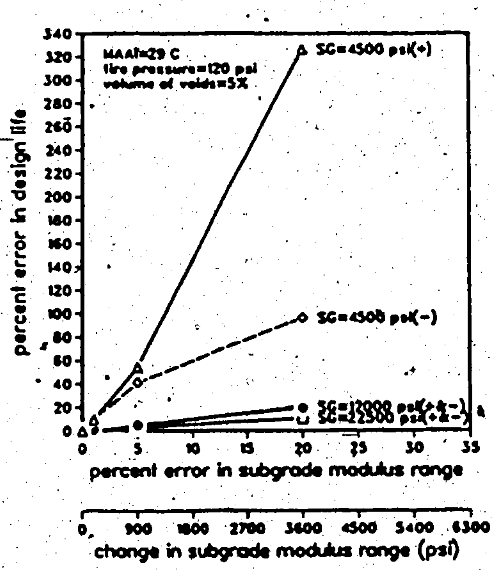
Fig. 5.16 Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 3% and MAAT of 29°C



(a) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5%, Tire Pressure of 70 psi, and MAAT of 29°C

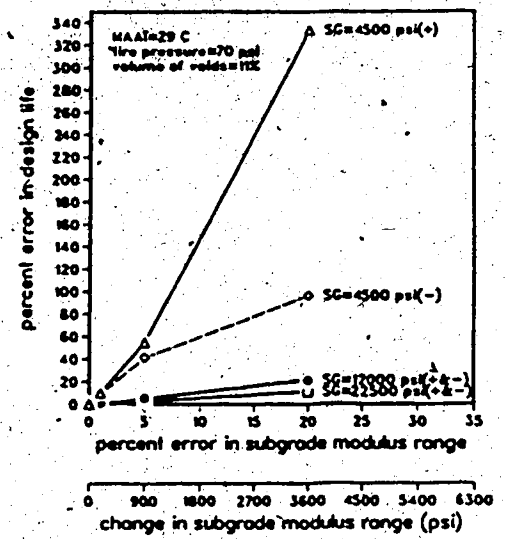


(b) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5%, Tire Pressure of 90 psi, and MAAT of 29°C

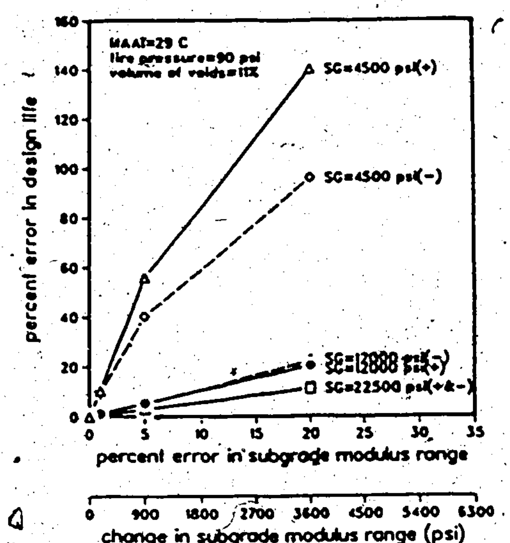


(c) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5%, Tire Pressure of 120 psi, and MAAT of 29°C

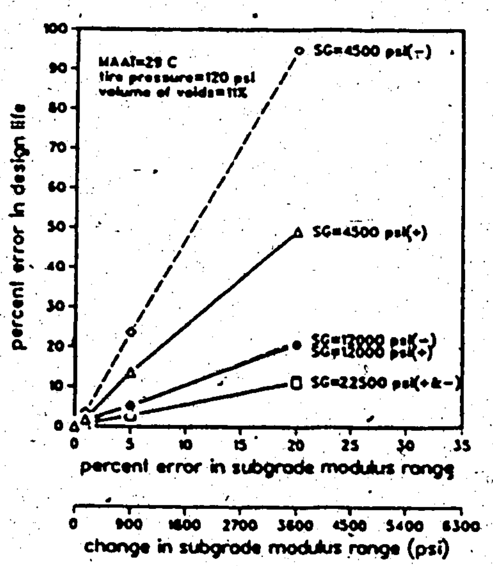
Fig. 5.17 Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 5% and MAAT of 29°C



(a) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11%, Tire Pressure of 70 psi, and MAAT of 29°C

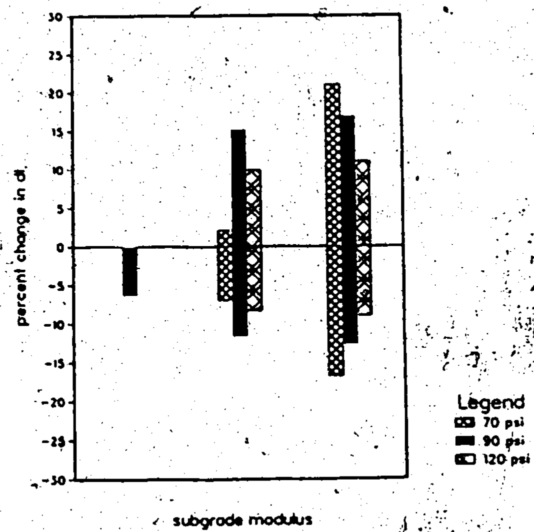


(b) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11%, Tire Pressure of 90 psi, and MAAT of 29°C



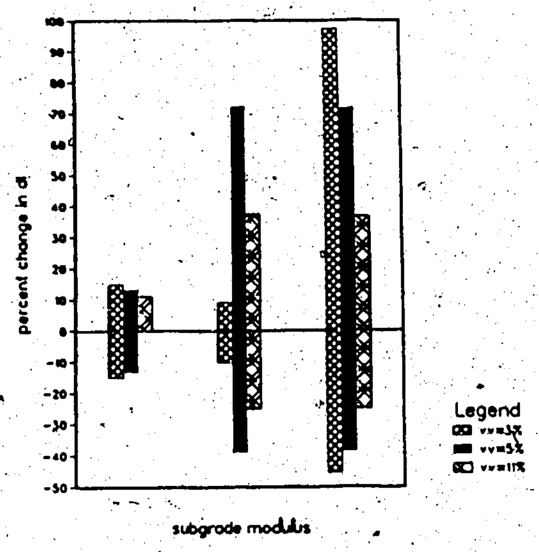
(c) Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11%, Tire Pressure of 120 psi, and MAAT of 29°C

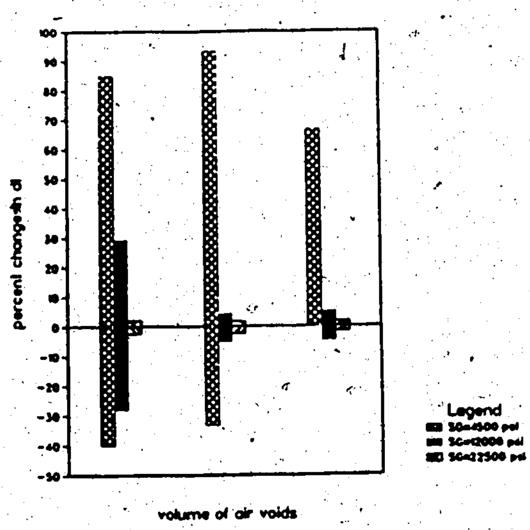
Fig. 5.18 Effect of Subgrade Modulus Variations on Design Life for Volume of Air Voids of 11% and MAAT of 29°C



(a) Effect of a 20% (+ and -) Variation in Tire Pressure on Design Life for Volume of Air Voids of 5%, Subgrade Moduli Values of 4500 psi, 12000 psi, 22500 psi, and MAAT of 7°C

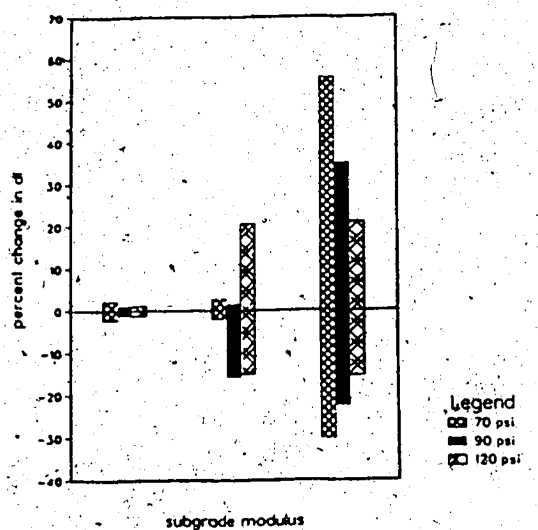
(b) Effect of a 12.5% (+ and -) Variation in Volume of Air Voids on Design Life for Tire Pressure of 120 psi, Subgrade Moduli Values of 4500 psi, 12000 psi, 22500 psi, and MAAT of 7°C





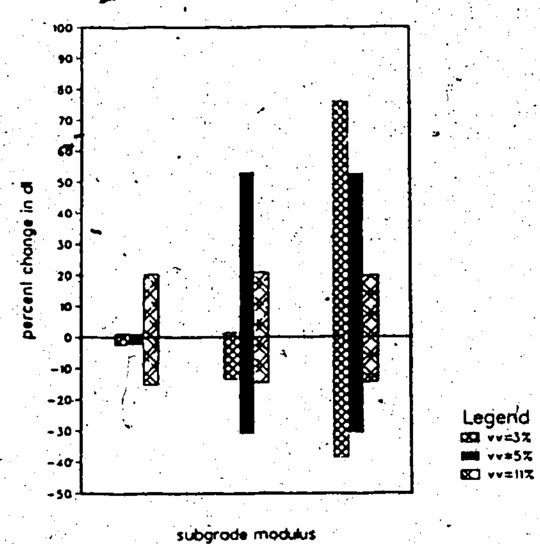
(c) Effect of a 5% (+ and -) Variation in Subgrade Modulus on Design Life for Tire Pressure of 120 psi, Volume of Air Voids of 3%, 5%, 11%, and MAAT of 7°C

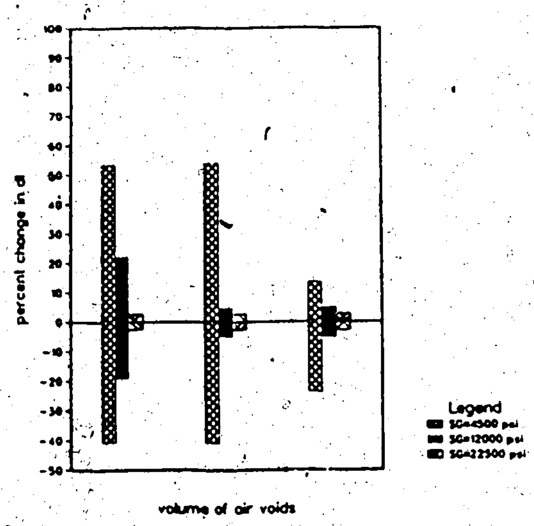
Fig. 5.19 General Trends in Design Life Resulting from Variations in Design Parameters under Cold Climate Conditions (MAAT of 7°C)



(a) Effect of a 20% (+ and −) Variation in Tire Pressure on Design Life for Volume of Air Voids of 5%, Subgrade Moduli Values of 4500 psi, 12000 psi, 22500 psi, and MAAT of 29°C

(b) Effect of a 12.5% (+ and −) Variation in Volume of Air Voids on Design Life for Tire Pressure of 120 psi, Subgrade Moduli Values of 4500 psi, 12000 psi, 22500 psi, and MAAT of 29°C





(c) Effect of a 5% (+ and −) Variation in Subgrade Modulus on Design Life for Tire Pressure of 120 psi, Volume of Air Voids of 3%, 5%, 11%, and MAAT of 29°C

Fig. 5.20 General Trends in Design Life Resulting from Variations in Design Parameters under Hot Climate Conditions (MAAT of 29°C)

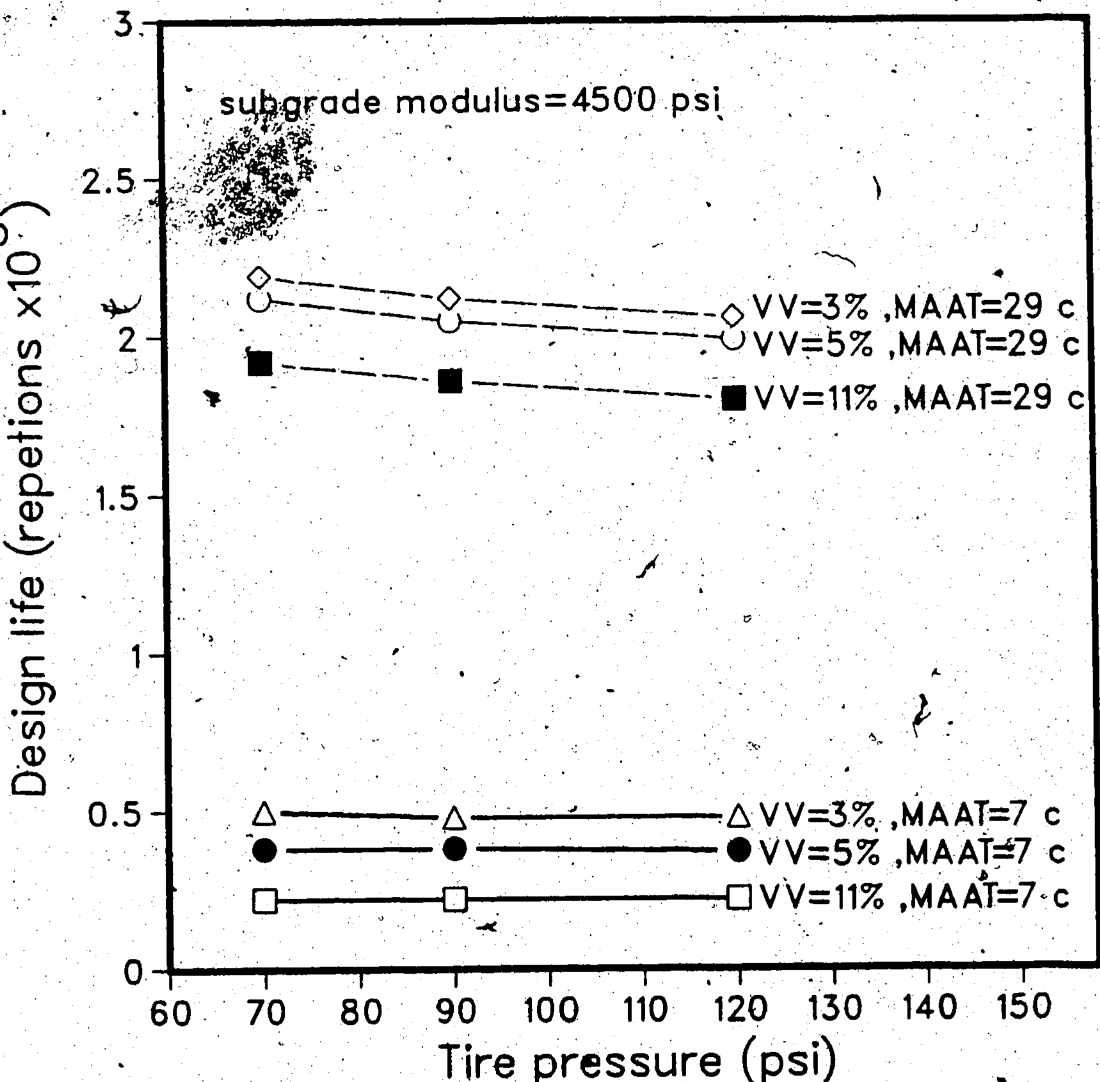


Fig. 5.21 Effect of Mean Annual Air Temperature on Design Life for Subgrade Modulus of 4500 psi

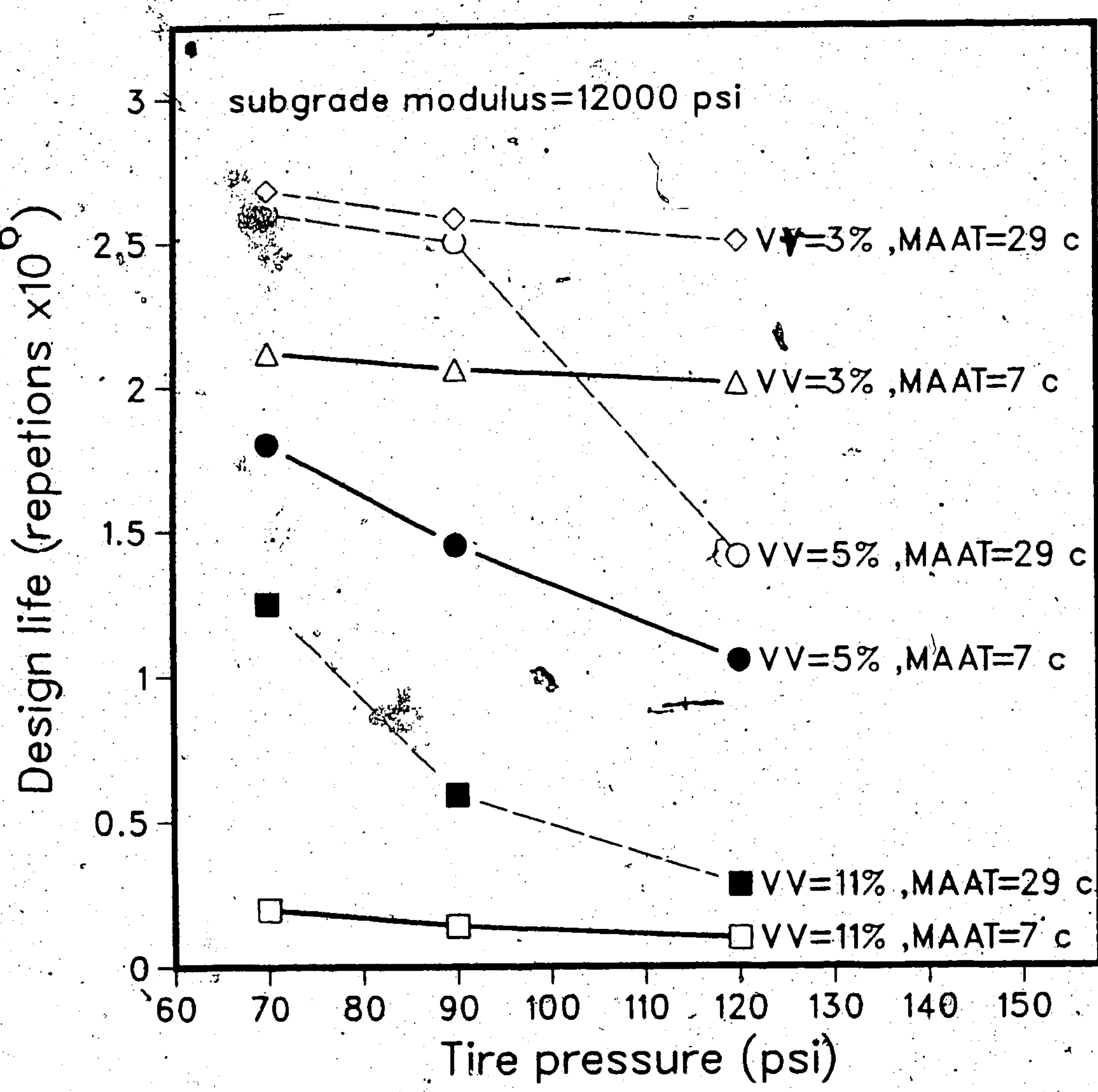


Fig. 5.22 Effect of Mean Annual Air Temperature on Design Life for Subgrade Modulus of 12000 psi

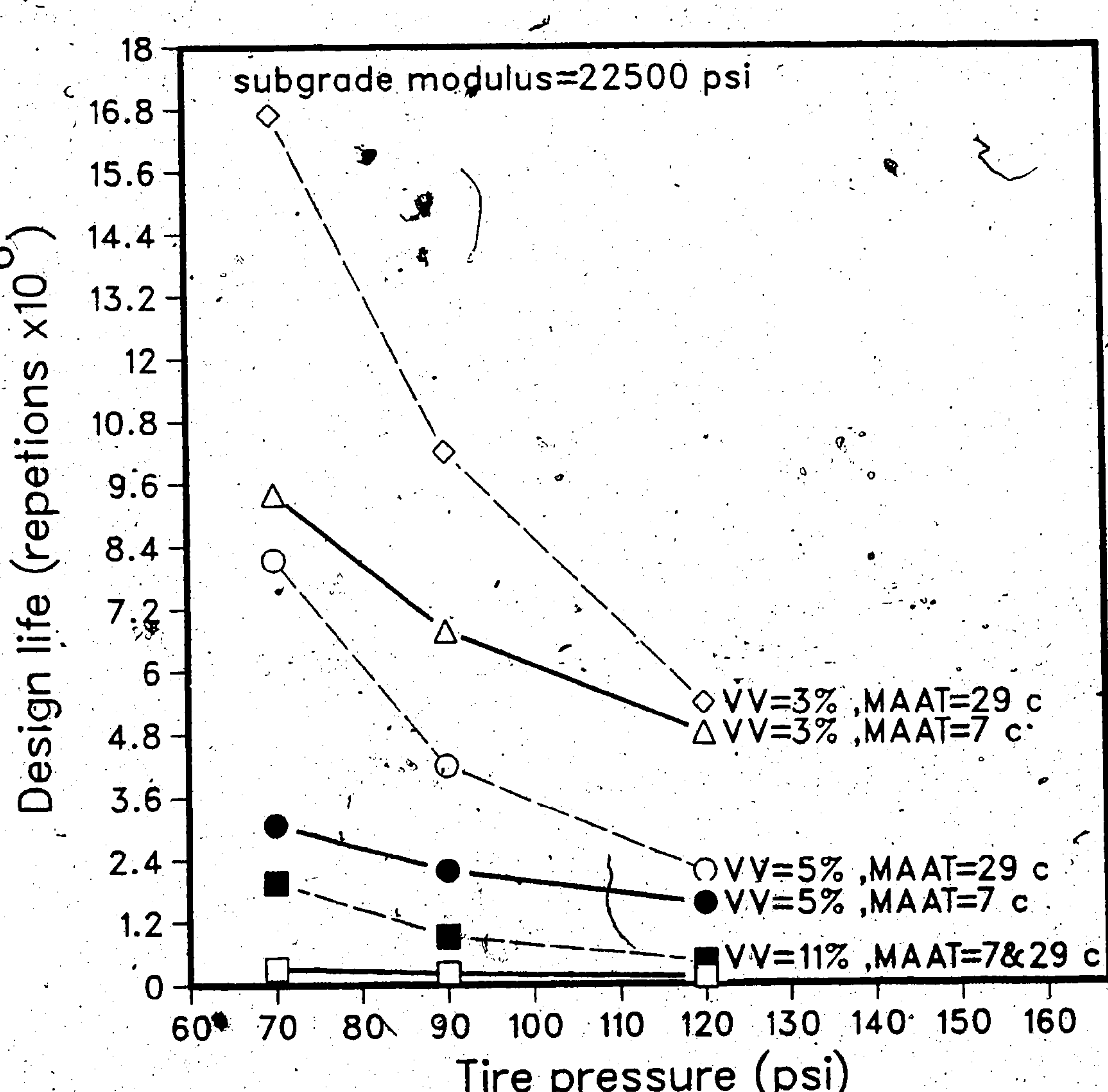


Fig. 5.23 Effect of Mean Annual Air Temperature on Design Life for Subgrade Modulus of 22500 psi

References

1. The Asphalt Institute, "Thickness Design - Asphalt Pavement Structures for Highways and Streets", The Asphalt Institute, College Park, Maryland, Asphalt Institute Manual Series No. 1, 9th Edition, 1981.

2. The Asphalt Institute, "Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition", The Asphalt Institute, College Park, Maryland, Asphalt Institute Research Report No. 82-2, Aug. 1982.

3. The Asphalt Institute, "Computer Program DAMA - User's Manual", The Asphalt Institute, College Park, Maryland, Asphalt Institute Computer Program Series CP-1, Oct. 1983.

4. B.P. Shields, K.O. Anderson, and J.M. Dacyszyn, "An Investigation of Low Temperature Transverse Cracking of Flexible Pavements", Proceedings of the 1969 Convention, Canadian Good Road Association, Edmonton, Sept. 1969, pp. 273-308.

5. K.O. Anderson and B.P. Shields, "Rheological Changes of Asphalt Cements in Service in Alberta", Proceedings, Twentieth Annual Conference, CTAA, Toronto, Nov. 1975, pp. 389-409.

6. K.O. Anderson and D.P. Palsat, "Recycling of Asphalt Pavements in Alberta", Proceedings, CTAA, Montreal, Quebec, Nov. 1982, Vol. 27, pp. 188-206.

7. G.E.P. Box, W.G. Hunter, and J.S. Hunter, "Statistics for Experimenters - An Introduction to Design, Data Analysis and Model Building", Wiley Interscience Publications, 1978, pp. 306-432.

6.0 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This investigation was aimed primarily at achieving a better understanding of the influence of the different design parameters on the design life of conventional asphalt concrete pavements. The study also included an evaluation of the potential applicability of a mechanistically-based design procedure to different climatic regions.

From the review of the literature, currently available theoretically-based design procedures for flexible pavements consist of two parts: an analytical part and an empirical one. The analytical part involves using the elastic or viscoelastic materials input to determine the pavement mechanical responses under the prevailing load and environmental conditions. The empirical part then correlates this information to the amount and extent of the structural distress induced in order to achieve a better understanding of the overall pavement performance.

In all mechanistic-empirical design models postulated, cracking of the asphalt layer due to fatiguing and/or sub-zero temperature effects, and rutting accumulated within the different pavement layers, were recognized as appropriate criteria governing the design. Limiting the amount of stresses and/or strains induced in the pavement system under the prevailing loading conditions, constitutes the basis for these criteria.

Despite the potential use of mechanistic-empirical design procedures, the implementation of such techniques is still under evaluation by many highway departments. This is primarily due to the difficulties encountered in obtaining precise estimates of some of the input parameters needed for such methods. Among these parameters is the estimation of the actual traffic to be anticipated to use the pavement facility during its design life, and the evaluation of the subgrade resilient characteristics.

The Asphalt Institute design method was selected as the semi-mechanistic model to achieve the objectives of this investigation. Two separate studies were made using this procedure. In the first study, a pavement section of known performance history located within a primary highway in central Alberta was analyzed using the Asphalt Institute computer program DAMA. Subsequently a comparison of the predicted and actual pavement life was made which revealed reasonable agreement. This phase satisfied the first objective of evaluating the potential applicability of the design procedure for implementation to other geographic areas.

The second study involved conducting a parametric evaluation of the different design inputs to identify and quantify the effects of such factors on the pavement design life. The same pavement structure included in the first study was used again.

Fractional factorial experimental design techniques were used in the simulation to isolate and identify the most significant factors that influence the design life. Six factors evolved from this phase, namely:

- subgrade resilient modulus
- mean annual air temperature of the particular climatic region
- tire pressure
- percent air voids in the asphalt concrete layer
- percent bitumen used in asphalt concrete layer
- quality of aggregate material used in base and subbase layers.

The first four factors which evolved from phase one of the parametric study were chosen for inclusion in the second phase which comprised the sensitivity analysis. In this part, the individual and interactive influences of the design parameters on the design life were quantified.

From the investigation, although it appears that mechanistic-empirical techniques are potential tools for designing and evaluating asphalt pavement structures, precise estimation of the input parameters to be used in such procedures is indispensible for obtaining realistic design alternatives.

8.2 Conclusions

The following conclusions can be disclosed from the results of this investigation:

1. The Asphalt Institute method appears to have potential to reasonably predict the structural performance of both in-service and newly constructed asphalt concrete pavements under Albertan conditions provided that appropriate design inputs are available.

2. The most significant factors that influence the pavement design life are:
   - subgrade resilient modulus, E_{SG}
   - mean annual air temperature of the climatic region in which the pavement structure is situated, MAAT
   - percent volume of air voids contained in the asphalt concrete layer, VV
   - tire pressure of the wheel load system, TP

   It has also been identified that the percent volume of bitumen contained in the asphalt concrete layer, and the quality of aggregate used in base and subbase layers may also influence the design life. However, these latter two factors are more controlled by the mix design requirements.

3. Of the four factors considered in the sensitivity analysis, a variation in the subgrade modulus has the most pronounced effect on the design life especially at low levels of the design parameter.

4. Changes in the volume of air voids and tire pressure have approximately equal influence on the design life, and come next in order to the effect of changing the subgrade modulus.

5. The change in design life resulting from a change in any of the design parameters is not only dependent on the change induced in that factor, but also on the magnitude of all other factors.

6. Longer design life was obtained for the same structure when placed in a hotter climate.

7. The failure mode was found to be controlled by the type of support provided by the subgrade layer. Excessive rutting is associated with weak subgrades while strong subgrades will indicate the mode of failure to be one of fatigue cracking.

6.3 Recommendations

1. The reasonable agreement which resulted between predicted and observed performance of the pavement section used in the case study is indicative of the potential applicability of the design method to Albertan conditions. However, further verification using several in-service pavement sections

throughout the province is still needed before any general conclusion can be drawn in this regard.

2. In using the design procedure to obtain a realistic estimation of the anticipated design life of a certain pavement structure, the stochastic nature of the significant design parameters should be realized. This means that estimation of the magnitudes of the different significant design parameters as well as the reliability of such estimations must be taken into consideration.

3. Among the four design parameters considered in the sensitivity analysis, tire pressure, volume of air voids and subgrade modulus are factors that can be controlled through proper mix and construction procedures and adequate load enforcement strategies. However, the influence of the mean annual air temperature of the climatical regime for which the pavement structure was to be designed is not a controllable factor and care should be exercised when designing or analyzing a pavement structure under different climatic conditions.

4. In constructing asphalt concrete pavements, special regard should be made to achieve and maintain the highest possible subgrade strength since such factor

appears to be the most influential in affecting the design life. Also proper caution should be exercised to adequately compact the asphalt concrete layer to reduce the volume of air voids and thus to provide for maximum resistance against fatigue cracking.

5. The Asphalt Institute design procedure can be incorporated in the Alberta pavement management system as a tool for estimating the remaining life of in-service pavements and thus aid in establishing the required maintenance and rehabilitation needs.

6. Further research examining the mechanism(s) governing permanent deformation within the asphalt concrete layer is needed to check candidate designs selected by the design procedure.

7. Further research is required to examine the effects of seasonal variations of the subgrade strength on pavement design life.

8. Further research is also needed to quantitatively correlate the structural aspects of pavement design to the overall performance of asphalt concrete pavements.

APPENDIX A

PROGRAM DAMA - INPUT AND OUTPUT FORMATS

A.1 Introduction and Definition of Terms

Program DAMA is a computer program developed by the Asphalt Institute to analyze and design asphalt concrete pavements. The program considers the pavement structure as an elastic multi-layered system. Any pavement structure composed of asphalt concrete (AC), asphalt emulsion (AE), untreated aggregate materials (AGG), and subgrade soils (SG) can be analyzed by DAMA provided that the maximum numbers of layers does not exceed five.

In general terms, DAMA utilizes the Chevron N-layer program, developed by Chevron Research Corporation, to obtain the required elastic solutions within any pavement structure under either single or dual wheel loading conditions. These mechanistic responses are then related to user-defined deformation and fatigue-cracking distress criterion to compute the damage induced within the pavement structure and consequently determines the pavement design (or remaining) life.

In analyzing a given pavement section, DAMA utilizes several input parameters. Such inputs include materials properties, environmental and traffic inputs, and two user-defined distress criteria. Different materials inputs pertaining to the individual constituent materials are used. A general description of all inputs together with the program outputs will be given in Sections A.3 and A.4 respectively.

Because program DAMA analyzes a given pavement structure, it is not a design program per se. However, structural designs can be obtained by performing a factorial analysis on a set of pavement structures.

A.2 How Program DAMA Works

The following steps describe how the program works:

Step 1: DAMA utilizes the Chevron N-layer subroutine, and materials, and environmental inputs to compute the mechanical responses of stress, strain, and displacement induced in the pavement system by traffic loadings. This is done for each month within a typical calendar year to account for changes in moduli values of all layers brought about by temperature, curing (AE), freeze-thaw and moisture conditions.

Step 2: Critical strains developed on a monthly basis are then used to predict the repetitions to failure (N_f) for each month and for each distress type by using the appropriate damage equation. For AC and AE layers the principal tensile strain (ϵ_t) at the bottom of each layer is used for the damage computation, whereas for SG soils the vertical compressive strain at the top of the subgrade layer is used.

Step 3: Unit damages are calculated for each month and for each distress type. The unit damage due to a single load repetition is $d = 1/N_f$. Monthly damage for each distress mode is the product of the unit damage and average monthly traffic volume in ESAL's.

Step 4: The total damages for the different layers and at the different radial points (at centre of one tire, edge of one tire, mid-point between two tires) are accumulated month by month to determine the yearly cumulative damage. When an AE layer exists, the damage computation is done for 12 months beyond the time of cure.

Step 5: The program assumes the yearly cumulative damage for the last 12-month analysis period, evolved from step 4 above, to be the same for all future years until a damage of 1.0 (or 100%) is reached. Using this assumption, the layer that first attains a total damage of 1.0 controls the design life of the pavement structure.

A.3 DAMA Input Data

As mentioned previously, the input data needed by the program DAMA to analyze a certain pavement structure, covers a wide range of materials, traffic, environmental and user-defined distress inputs. Figure A.1 shows a typical input data sheet. Following is a description of each of the input parameters contained in Fig. A.1:

1. <u>INOPT, NSUB, STRESS (3I1)</u>

INOPT: This variable is used to initiate or end an input file. A zero value will terminate a runstream and 1 will initiate a runstream.

NSUB: This parameter refers to the number of subgrade layers within the pavement structure. A maximum number of subgrade layers is four.

STRESS: This variable is used to either print or suppress the printing of stress and strain values that are not routinely printed. A value of zero will suppress the printing of these stresses and strains, while any integer value will cause those responses to be printed.

2. <u>TITLE (20A4)</u>

This is the heading of the problem. Up to 80 alphanumerics can be used to define the specific problem.

3. <u>NS, MOPEN, MCURE, MTRAFF (3I5, I10)</u>

NS: Indicates the number of layers in the pavement structure. Maximum NS=5, including the subgrade (S).

MOPEN: Month at which the roadway is opened to traffic. For example, 7 indicates that the pavement has been opened to traffic in July.

MCURE: Cure time of AE layer in months before the road is opened to traffic. The value of MCURE is greater than 0 and less than the total cure time needed for the AE layer to attain full strength (normally 6 months).

MTRAFF: Is the average monthly traffic expressed in terms of equivalent single axle loads (ESAL's).

4. WGT, PSI, SD (3F 10.0)

WGT: Refers to load per single wheel, lb.

PSI: Is the contact pressure, psi.

SD: Is the center to center distance between dual tires, in.

5. ATEMP (I), I = 1, 12 (12F 5.1)

This is the mean monthly air temperatures from January to December, °F.

6. ETEMP (I), I = 1, 12 (12F 5.1)

These are the pavement temperatures at which the AC and AE moduli are evaluated. ETEMP shall cover the expected pavement temperature range. A relationship relating air and pavement temperatures is used to obtain such range.

7. MCODE (I), ME (I), V (I), HH (I), TCURE (I), VV (I), VB (I), P200 (I), FR (I), U(I), (2I1, 8F8.2) (Subscript I denotes layer numbers)

MCODE: This indicates material code. 1 for AE, 2 for AC, 3 for AGG, and 4 for SG.

ME: This is the material modulus code. For AC, if ME=0, then temperature modulus values must be provided as input. If ME=1 (for AC also) then the program uses the temperature-modulus developed by the Asphalt Institute to compute such values. ME=0 should be used for AE, AGG, and SG materials since E-values of such materials are always provided as inputs.

V: Is the Poisson's ratio. Typical values are 0.35 for AE, AC, and AGG, and 0.45 for SG.

HH: Indicates the layer thickness, in. Omit for subgrade layer since it is assumed infinite in the vertically downward direction.

TCURE: Designates the total cure time in months (including MCURE). TCURE is given only for AE layer, and is equal to zero for AC layer.

VV: This is the percent volume of air voids in either the AC or AE layer.

VB: Is the percent volume of bitumen in AC or AE layers.

P200: Is the percent aggregate passing #200 sieve, (80 μm). This parameter is provided for AC layer only.

FR: Indicates the expected loading frequency, Hz. It is provided for AC layer only.

U: Is the asphalt viscosity @ 70°F (21°C), 10^6

poises. It is also provided for AC layer only.

Note:
a. Only VV and VB are provided for AE layer and AC layer with ME=0. For AC layer with ME=0, provide all items from VV to U.

b. For AGG layer only MCODE, ME, V and H are provided.

c. For SG layer only MCODE, ME, and V are provided.

d. Card 7 is required for each layer from the top layer to the bottom.

8. EI (I,J), J=1, 12 (8F 10.0/4F 10.0)

These are the 12 moduli values provided for each layer depending on the type of material. 12 modulus values are provided for AC layer with ME=0. Not applicable for AC layer with ME=1. 12 initial modulus values at 12 ETEMP values before cure are provided for AE layer. 12 monthly material quality values (K1) for AGG layer are provided. 12 monthly modulus values for SG layer are provided. All the aforementioned moduli values are from January to December.

9. EF (I,J), T = 1, 12 (8F 10.0/4F 10.0)

These are the final modulus values provided for AE layer from January to December. They are not applicable for any other layer.

Note: Cards 7, 8 and 9 are required as a set for each layer from layer 1 to NS.

~~10.~~ F0, F1, F2, F3, F4, F5 (6F 10.0)

These are the coefficients for fatigue damage model. This card (10) is required for each AC and AE layer within the pavement structure being analyzed.

11. SDFG, SDF1 (2E 10.5)

These are the coefficients of the deformation damage model for subgrades. Note that, although there may exist more than one subgrade, this card is provided only once.

12. The final card should set INOPT to zero as well as NSUB and STRESS.

It should be noted that, the numbers and letters between the brackets in each of the input cards described above, denotes the format of the FORTRAN write statement used to input the specific parameter.

A.4 Format of Program Output

The general description of program DAMA output is explained briefly in the following paragraphs.

The first page of the program output, following the general state, contains the input information pertaining to the proposed pavement design, the curing conditions, monthly air temperatures, and traffic and load conditions. Computational points are calculated from load data.

Materials modulus values are printed next. Asphalt concrete moduli are either calculated from the mix properties or input directly as mentioned previously. Moduli for asphalt emulsion mixes are input individually. The program then interpolates these moduli at the required ETEMP temperature values. The monthly K1 values for aggregate materials and the subgrade modulus values may also be input as constants or varied to reflect seasonal changes.

Damage models are shown on the following output pages. These models are calculated using the fatigue and permanent deformation equations and coefficients that are input by the user.

When the stress option is selected, stress and strain values are printed next. These are vertical, radial, shear and bulk stresses and radial strain and deflection values for each radial distance and depth under one load for each month.

Subsequent pages summarize the combined structural responses for the dual-wheel loading. These responses include surface deflection, DZ, tensile strain at bottom of asphalt (or emulsified asphalt) layer, ET, and compressive strain at top of subgrade layer, EC. Damage factors calculated using these responses and the damage equations are then printed for each month. When an AE layer exists, these damage factors are calculated for 12 months plus the curing time in months. In the absence of AE layers, the damage factors are calculated for 12 months only.

The final output is the design life for a cumulative damage factor of 1.0. This life is based on the cumulative damage calculated for the last 12 month period. Design life is expressed both in years, at the specified average monthly load application rate, and as total repetitions of the input wheel load expressed in terms of equivalent single axle load applications (ESAL's).

An example illustrating the use of program DAMA for a highway problem will be described next.

A.5 Example Problem

This example, shown in Table A.1 and A.2, illustrates the use of program DAMA to predict the design life of a conventional flexible pavement structure. The analysis is made for a pavement cross-section consisting of 102 mm-thick (4 in.) asphalt concrete layer, underlain by 51 mm-thick (2 in.) asphalt emulsified layer and a 306 mm-thick aggregate base layer. This structure is resting on a subgrade soil of relatively low strength (normal resilient modulus of 7500 psi). Thicknesses and Poisson's ratio for these materials are presented on the first page of Table A.2. A six-month cure period is assumed for the asphalt emulsion layer. Traffic is expected to start immediately after construction, in July.

Page one of Table A.2 also contains the mean monthly air temperatures. This input information is used in calculating pavement temperatures, and is taken into consideration in making the seasonal variations (either reduction or increase) in aggregate K1 values and subgrade moduli. Pavement temperatures derived from mean monthly air temperatures are also used to compute the asphalt concrete moduli.

Load input information are also shown in page one of Table A.2. Two 20 kN (4500 lb) single wheel loads, spaced 343 mm (13.5 in.) apart, are used. Contact pressures are 483 kPa (70.0 psi). The 115 mm (4.52 in.) radius of loading and the computational points are calculated by the program and listed at the bottom of page one of Table A.2.

An average of 2000 ESAL's per month has been input. This is also reproduced on page one of Table A.2.

The temperature-modulus relationship for the asphalt concrete was calculated using mix and asphalt properties and the equation described earlier (Eq. 3.6). The input parameters and the resulting moduli values are printed on page two of Table A.2.

Also shown on page two of Table A.2 are the initial and final moduli for the asphalt emulsion layer.

Monthly K1 values and subgrade moduli are shown on page two and three of Table A.2. The variations in K1 values reflect the dependency of such parameter on variations in bulk stress brought about by temperature effects on asphalt concrete moduli and variation in subgrade moduli that are specified as input.

The variations in subgrade moduli indicate the effect of seasonal freezing and thawing of the subgrade layer and the accompanied increase or reduction in the subgrade strength.

Equations for failure criteria are shown on pages three and four of Table A.2.

Results of program computations are summarized on pages four through nine of Table A.2. Pages four to six contain the monthly structural responses. These responses include calculated pavement temperatures, asphalt concrete and asphalt emulsion moduli at the given pavement temperatures, calculated aggregate moduli, and subgrade moduli for each month and location under the dual wheels. Also shown are surface deflections, tensile strains at bottom of both asphalt concrete and emulsified asphalt mixture, and vertical compression strains at top of subgrade layer. Results of damage calculations based on these structural responses are given on pages seven to nine for each month, each layer and each location under the wheel loads.

The output of structural responses and monthly damages can be useful in answering questions like what month is the most severe for accumulating fatigue and/or permanent deformation damages? The structural response output can also be used to show the effect of reduced subgrade moduli values on strains and deflections.

The last page of Table A.2 contains the final output of program DAMA. This output is the predicted design life of the pavement section.

In this example, the predicted subgrade deformation life of 7.4 years is found to be the controlling design life. Predicted fatigue lives for asphalt concrete layer (1100.2 years) and asphalt emulsified layer (8.7 years) are also listed but do not control the design life. The program also prints the design lives in terms of the ESAL's repetitions required to produce the specific mode of failure.

```
 1       1     1     0
 2   EXAMPLE PROBLEM
 3       4     7     0.      2000
 4   4500.     70.    .    13.5
 5   40.5 42.2 49.2 59.5 67.4 74.4 77.5 76.5 70.6 60.2 50.0 41.2
 6   40.  50.  55.  60.  65.  70.  75.  80.  85.  90.  95.  100.
 7   21       .35      4.       0.       5.       11.      5.8      10.      1.5
 8   10.      .35      2.       6.       8.       9.
 9   400000.   290000.   230000.   190000.   160000.   130000.   110000.   90000.
10   70000.    56000.    45000.    35000.
11   1400000.  1000000.  800000.   690000.   560000.   460000.   380000.   320000.
12   250000.   200000.   155000.   125000.
13   30       .35      6.
14   12000.    24000.    36000.    3000.     5250.     7500.     9750.     12000.
15   12000.    12000.    12000.    12000.
16   40       .45
17   7500.     28750.    50000.    3150.     4237.     5325.     6412.     7500.
18   7500.     7500.     7500.     7500.
19   18.4      .004325   3.291     .854      4.84      .69
20   18.4      .004325   3.291     .854      4.84      .69
21     .1365E-8    .4477E1
22      0     0     0.
```

Table A.1 Input data for example problem

1 EXAMPLE PROBLEM

LAYER AND MATERIAL PROPERTIES

LAYER NUMBER	MATERIAL TYPE	POISSON'S RATIO	THICKNESS (IN)
1	ASPH. CONC.	0.35	4.00
2	ASPH. EMUL.	0.35	2.00
3	AGGREGATE	0.35	6.00
4	SUBGR. SOIL	0.45	

CURING CONDITIONS

LAYER NUMBER	MATERIAL TYPE	CURE TIME (MONTHS)	MONTH OPENED TO TRAFFIC	MONTHS CURED BEFORE OPENING
1	ASPH. CONC.	[illegible]	[illegible]	0
2	ASPH. EMUL.	[illegible]	[illegible]	0

TRAFFIC CONDITION

NUMBER OF REPETITIONS PER MONTH 2000

ENVIRONMENTAL CONDITIONS
(MEAN MONTHLY AIR TEMPERATURES, DEG. F)

JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT	NOV.	DEC.
40.5	42.2	49.2	59.5	67.4	74.4	77.5	76.5	70.6	60.2	50.0	41.2

LOAD CONFIGURATION AND COMPUTATIONAL POINTS

LOAD PER TIRE = 4500. LBS
CONTACT PRESSURE = 70.00 PSI
RADIUS OF LOAD = 4.52 IN
LOAD SPACING = 13.50 IN

COMPUTATIONAL POINT 1 X = 0.0 IN (CENTER OF ONE TIRE)
COMPUTATIONAL POINT 2 X = 4.52 IN (EDGE OF ONE TIRE)
COMPUTATIONAL POINT 3 X = 6.75 IN (MIDPOINT OF TWO TIRES)

Table A.2 Output for example problem

1 MODULI CONDITIONS

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
1	ASPH. CONC.				
		1	40.0	1653384.	
		2	50.0	1238306.	
		3	55.0	1051518.	
		4	60.0	881973.	
		5	65.0	730867.	
		6	70.0	598480.	
		7	75.0	484356.	
		8	80.0	387486.	
		9	85.0	306471.	
		10	90.0	239675.	
		11	95.0	185359.	
		12	100.0	141779.	

MODULUS PARAMETERS: NU= 1.500 P200= 5.8 FREQ=10.0HZ VV= 5.0 VB=11.0

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
2	ASPH. EMUL.				
		1	40.0	400000.	1400000.
		2	50.0	290000.	1000000.
		3	55.0	230000.	800000.
		4	60.0	190000.	690000.
		5	65.0	160000.	560000.
		6	70.0	130000.	460000.
		7	75.0	110000.	380000.
		8	80.0	90000.	320000.
		9	85.0	70000.	250000.
		10	90.0	56000.	200000.
		11	95.0	45000.	155000.
		12	100.0	35000.	125000.

UNTREATED GRANULAR BASE/SUBBASE

LAYER NUMBER	MATERIAL TYPE	MONTH	K1 FACTOR
3	AGGREGATE		
		JAN.	12000.
		FEB.	24000.
		MAR.	36000.
		APRIL	3000.
		MAY	5250.
		JUNE	7500.
		JULY	9750.

Table A.2 Continued

```
                    AUG.        12000.
                    SEPT.       12000.
                    OCT.        12000.
                    NOV.        12000.
                    DEC.        12000.
                       MODULI CONDITIONS


  SUBGRADE LAYER

   LAYER     MATERIAL      MONTH        MODULUS
   NUMBER      TYPE                      (PSI)
     4       SUBGR. SOIL

                           JAN.          7500.
                           FEB.         28750.
                           MAR.         50000.
                           APRIL         3150.
                           MAY           4237.
                           JUNE          5325.
                           JULY          6412.
                           AUG.          7500.
                           SEPT.         7500.
                           OCT.          7500.
                           NOV.          7500.
                           DEC.          7500.




                                DAMAGE  MODELS


FATIGUE DAMAGE    NF = (FO) * (F1) * (10**M) * (ET)**(-F2) * (MOD)**(-F3)

          WHERE

          NF IS LOAD REPETITIONS TO FAILURE
          FO IS DISTRESS TO PERFORMANCE FACTOR
          10**M IS MIX FACTOR (M= F4*(VB/(VB+VV)-F5)

                  VV IS VOLUME OF VOIDS IN ASPHALT MIX (PERCENT)
                  VB IS VOLUME OF BITUMEN IN ASPHALT MIX (PERCENT)

          ET IS TENSILE STRAIN IN ASPHALT LAYER
          MOD IS MODULUS OF ASPHALT
          F1, F2 AND F3 ARE COEFFICIENTS OF LAB FATIGUE EQUATION
                  GIVEN BY    NF = F1 * ET**(-F2) * MOD**(-F3)


          PARAMETERS OF LAYER 1

          FO =.18400E+02  F1 =.43250E-02  F2 =.32910E+01  F3 =.85400E+00

          F4 =.48400E+01  F5 =.69000E+00  VB = 11.00      VV =  5.00
```

Table A.2 Continued

```
FINAL FATIGUE EQUATION: NF=.77393E-01*(ET)**(-.32910E+01)*MOD**(-.85400E+00)

            PARAMETERS OF LAYER 2

            FO =.18400E+02  F1 =.43250E-02  F2 =.32910E+01  F3 =.85400E+00

            F4 =.48400E+01  F5 =.69000E+00  VB =  9.00      VV =  8.00

FINAL FATIGUE EQUATION: NF=.13291E-01*(ET)**(-.32910E+01)*MOD**(-.85400E+00)


 DEFORMATION DAMAGE    NF = DO * EC**(-D1)

            WHERE

            NF IS LOAD REPETITIONS TO FAILURE
            DO AND D1 ARE COEFFICIENTS FOR SUBGRADE DEFORMATION MODEL
            EC IS VERTICAL COMPRESSIVE STRAIN AT TOP OF SUBGRADE LAYER(S)


            DO =.13650E-08        D1 =.44770E+01

                   ***** MONTHLY STRUCTURAL RESPONSE *****


                      TYPES OF STRUCTURAL RESPONSES

                 DZ   VERTICAL DEFORMATION AT THE TOP OF LAYER (IN)
                 ET   TENSILE STRAIN AT THE BOTTOM OF LAYER (IN/IN)
                 EC   COMPRESSIVE STRAIN AT THE TOP OF LAYER(IN/IN)
```

STRUCTURAL RESPONSES

MON	L	PVT. TEMP	MODULUS (PSI)	RESP TYPE	COMPUTATIONAL POINTS CENTER	EDGE	MID. PT.
1	1	91	220117.	DZ	0.3018E-01	0.3044E-01	0.3013E-01
1	1	91	220117.	ET	0.2136E-03	0.2029E-03	0.1920E-03
1	2	88	93812.	ET	0.3598E-03	0.3609E-03	0.3545E-03
1	3		28093.				
1	4		6412.	EC	0.8242E-03	0.8738E-03	0.8750E-03
2	1	90	233970.	DZ	0.2550E-01	0.2567E-01	0.2538E-01
2	1	90	233970.	ET	0.1441E-03	0.1361E-03	0.1280E-03
2	2	87	148110.	ET	0.2901E-03	0.2898E-03	0.2842E-03
2	3		34346.				
2	4		7500.	EC	0.6905E-03	0.7301E-03	0.7307E-03

Table A.2 Continued

3	1	83	329393.	DZ	0.2387E-010.2420E-010.2406E-01
3	1	83	329393.	ET	0.1078E-030.1044E-030.9951E-04
3	2	80	243854.	ET	0.2516E-030.2537E-030.2498E-03
3	3		32534.		
3	4		7500.	EC	0.6159E-030.6539E-030.6562E-03
4	1	71	570960.	DZ	0.2159E-010.2201E-010.2199E-01
4	1	71	570960.	ET	0.8189E-040.8120E-040.7851E-04
4	2	69	415044.	ET	0.1978E-030.2016E-030.1995E-03
4	3		30163.		
4	4		7500.	EC	0.5075E-030.5406E-030.5452E-03
5	1	58	913539.	DZ	0.1953E-010.1994E-010.1996E-01
5	1	58	913539.	ET	0.6058E-040.6077E-040.5933E-04
5	2	57	679451.	ET	0.1507E-030.1543E-030.1535E-03
5	3		28227.		
5	4		7500.	EC	0.4116E-030.4382E-030.4447E-03
6	1	48	1291315.	DZ	0.1795E-010.1830E-010.1830E-01
6	1	48	1291315.	ET	0.4572E-040.4612E-040.4533E-04
6	2	48	1019757.	ET	0.1178E-030.1208E-030.1207E-03
6	3		26831.		
6	4		7500.	EC	0.3413E-030.3628E-030.3702E-03
7	1	47	1322717.	DZ	0.1782E-010.1817E-010.1817E-01
7	1	47	1322717.	ET	0.4445E-040.4486E-040.4411E-04
7	2	47	1057937.	ET	0.1153E-030.1182E-030.1182E-03
7	3		26727.		
7	4		7500.	EC	0.3359E-030.3570E-030.3645E-03
8	1	49	1247737.	DZ	0.6673E-020.6795E-020.6758E-02
8	1	49	1247737.	ET	0.3140E-040.3099E-040.2980E-04
8	2	49	992592.	ET	0.7724E-040.7846E-040.7738E-04
8	3		72336.		
8	4		28750.	EC	0.1680E-030.1794E-030.1799E-03
9	1	58	944573.	DZ	0.[illegible]E-020.4597E[illegible]0.4509E-02
9	1	58	944573.	ET	0.[illegible]E-040.2888E[illegible]0.2718E-04
9	2	56	727987.	ET	0.6666E-040.6646E-040.6509E-04
9	3		125483.		
9	4		50000.	EC	0.1288E-030.1365E-030.1363E-03
10	1	70	591399.	DZ	0.4298E-010.4386E-010.4392E-01
10	1	70	591399.	ET	0.1075E-030.1088E-030.1073E-03
10	2	68	471552.	ET	0.2777E-030.2853E-030.2853E-03
10	3		6997.		
10	4		3150.	EC	0.7570E-030.8048E-030.8229E-03
11	1	79	393367.	DZ	0.3698E-010.3775E-010.3780E-01

Table A.2 Continued

11	1	79	393367.	ET	0.1163E-030.1165E-030.[illegible]
11	2	77	339115.	ET	0.3090E-030.3159E-030.3133E-03
11	3		13059.		
11	4		4237.	EC	0.8070E-030.8598E-030.869[illegible]E-03
12	1	87	264767.	DZ	0.3296E-010.3351E-010.3337E-01
12	1	87	264767.	ET	0.1229E-030.1206E-030.1155E-03
12	2	85	240339.	ET	0.3287E-030.3328E-030.3282E-03
12	3		20032.		
12	4		5325.	EC	0.8210E-030.8731E-030.8770E-03
13	1	91	220117.	DZ	0.2897E-010.29[illegible]E-010.2900E-01
13	1	91	220117.	ET	0.1161E-030.[illegible]E-030.1052E-03
13	2	88	205971.	ET	0.3077E-030.3085E-030.3030E-03
13	3		27186.		
13	4		6412.	EC	0.7622E-030.8071E-030.8084E-03
14	1	90	233970.	DZ	0.2502E-010.2522E-010.2495E-01
14	1	90	233970.	ET	0.1029E-030.9768E-040.9176E-04
14	2	87	216484.	ET	0.2670E-030.2668E-030.2616E-03
14	3		33782.		
14	4		7500.	EC	0.6659E-030.7037E-030.7043E-03
15	1	83	329393.	DZ	0.2360E-010.2394E-010.2380E-01
15	1	83	329393.	ET	0.9019E-040.8756E-040.8340E-04
15	2	80	296211.	ET	0.2376E-030.2396E-030.2359E-03
15	3		32255.		
15	4		7500.	EC	0.6010E-030.6377E-030.6401E-03
16	1	71	570960.	DZ	0.2143E-010.2185E-010.2183E-01
16	1	71	570960.	ET	0.7524E-040.7466E-040.7220E-04
16	2	69	457276.	ET	0.1910E-030.1946E-030.1926E-03
16	3		30036.		
16	4		7500.	EC	0.4990E-030.5313E-030.5360E-03
17	1	58	913539.	DZ	0.1946E-010.1986E-010.1988E-01
17	1	58	913539.	ET	0.5835E-040.5855E-040.5716E-04
17	2	57	708941.	ET	0.1481E-030.1516E-030.1508E-03
17	3		28175.		
17	4		7500.	EC	0.4079E-030.4341E-030.4406E-03
18	1	48	1291315.	DZ	0.1793E-010.1828E-010.1828E-01
18	1	48	1291315.	ET	0.4528E-040.4568E-040.4490E-04
18	2	48	1030526.	ET	0.1172E-030.1203E-030.1201E-03
18	3		26819.		
18	4		7500.	EC	0.3404E-030.3618E-030.3692E-03

1 ***** MONTHLY DAMAGES *****

Table A.2 Continued

TYPES OF STRUCTURAL RESPONSES

DZ VERTICAL DEFORMATION AT THE TOP OF LAYER (IN)
ET TENSILE STRAIN AT THE BOTTOM OF LAYER (IN/IN)
EC COMPRESSIVE STRAIN AT THE TOP OF LAYER(IN/IN)

MON	L	PVT. TEMP	MODULUS (PSI)	RESP TYPE	COMPUTATIONAL POINTS CENTER	EDGE	MID., PT.
1	1	91	220117.	DZ			
1	1	91	220117.	ET	0.7861E-03	0.6644E-03	0.5533E-03
1	2	88	93812.	ET	0.1229E-01	0.1242E-01	0.1171E-01
1	3		28093.				
1	4		6412.	EC	0.2285E-01	0.2969E-01	0.2988E-01
2	1	90	233970.	DZ			
2	1	90	233970.	ET	0.2267E-03	0.1881E-03	0.1538E-03
2	2	87	148110.	ET	0.8944E-02	0.8906E-02	0.8353E-02
2	3		34346.				
2	4		7500.	EC	0.1035E-01	0.1328E-01	0.1333E-01
3	1	83	329393.	DZ			
3	1	83	329393.	ET	0.1170E-03	0.1053E-03	0.8983E-04
3	2	80	243854.	ET	0.8563E-02	0.8806E-02	0.8367E-02
3	3		32534.				
3	4		7500.	EC	0.6200E-02	0.8110E-02	0.8240E-02
4	1	71	570960.	DZ			
4	1	71	570960.	ET	0.7567E-04	0.7358E-04	0.6588E-04
4	2	69	415044.	ET	0.6115E-02	0.6505E-02	0.6289E-02
4	3		30163.				
4	4		7500.	EC	0.2606E-02	0.3458E-02	0.3592E-02
5	1	58	913539.	DZ			
5	1	58	913539.	ET	0.4193E-04	0.4236E-04	0.3913E-04
5	2	57	679451.	ET	0.3807E-02	0.4115E-02	0.4041E-02
5	3		28227.				
5	4		7500.	EC	0.1021E-02	0.1352E-02	0.1443E-02
6	1	48	1291315.	DZ			
6	1	48	1291315.	ET	0.2232E-04	0.2297E-04	0.2170E-04
6	2	48	1019757.	ET	0.2393E-02	0.2602E-02	0.2593E-02
6	3		26831.				
6	4		7500.	EC	0.4414E-03	0.5801E-03	0.6351E-03
7	1	47	1322717.	DZ			
7	1	47	1322717.	ET	0.2076E-04	0.2139E-04	0.2024E-04
7	2	47	1057937.	ET	0.2297E-02	0.2499E-02	0.2494E-02
7	3		26727.				
7	4		7500.	EC	0.4111E-03	0.5397E-03	0.5920E-03

Table A.2 Continued

8	1	49	1247737.	DZ	
8	1	49	1247737.	ET	0.6296E-050.6025E-050.5299E-05
8	2	49	992592.	ET	0.5830E-030.6137E-030.5864E-03
8	3		72336.		
8	4		28750.	EC	0.1846E-040.2479E-040.2511E-04
9	1	58	944573.	DZ	
9	1	58	944573.	ET	0.4474E-050.3766E-050.3085E-05
9	2	56	727987.	ET	0.2755E-030.2728E-030.2547E-03
9	3		125483.		
9	4		50000.	EC	0.5620E-050.7283E-050.7236E-05
10	1	70	591399.	DZ	
10	1	70	591399.	ET	0.1907E-030.1986E-030.1895E-03
10	2	68	471552.	ET	0.2082E-010.2275E-010.2276E-01
10	3		6997.		
10	4		3150.	EC	0.1562E-010.2054E-010.2269E-01
11	1	79	393367.	DZ	
11	1	79	393367.	ET	0.1747E-030.1759E-030.1605E-03
11	2	77	339115.	ET	0.2233E-010.2400E-010.2336E-01
11	3		13059.		
11	4		4237.	EC	0.2079E-010.2762E-010.2900E-01
12	1	87	264767.	DZ	
12	1	87	264767.	ET	0.1492E-030.1401E-030.1218E-03
12	2	85	240339.	ET	0.2039E-010.2124E-010.2028E-01
12	3		20032.		
12	4		5325.	EC	0.2246E-010.2958E-010.3018E-01
13	1	91	220117.	DZ	
13	1	91	220117.	ET	0.1058E-030.9204E-040.7636E-04
13	2	88	205971.	ET	0.1438E-010.1450E-010.1367E-01
13	3		27186.		
13	4		6412.	EC	0.1611E-010.2081E-010.2096E-01
14	1	90	233970.	DZ	
14	1	90	233970.	ET	0.7497E-040.6311E-040.5137E-04
14	2	87	216484.	ET	0.9416E-020.9382E-020.8802E-02
14	3		33782.		
14	4		7500.	EC	0.8794E-020.1126E-010.1131E-01
15	1	83	329393.	DZ	
15	1	83	329393.	ET	0.6500E-040.5896E-040.5023E-04
15	2	80	296211.	ET	0.8381E-020.8610E-020.8181E-02
15	3		32256.		
15	4		7500.	EC	0.5558E-020.7249E-020.7370E-02
16	1	71	570960.	DZ	

Table A.2 Continued

```
        16 1  71   570960.  ET  0.5726E-040.5583E-040.4998E-04
        16 2  68   457276.  ET  0.5922E-020.6292E-020.6082E-02
        16 3        30036.
        16 4         7500.  EC  0.2418E-020.3201E-020.3329E-02


        17 1  58   913539.  DZ
        17 1  58   913539.  ET  0.3708E-040.3747E-040.3462E-04
        17 2  57   708941.  ET  0.3726E-020.4024E-020.3953E-02
        17 3        28175.
        17 4         7500.  EC  0.9789E-030.1296E-020.1384E-02


        18 1  48  1291315.  DZ
        18 1  48  1291315.  ET  0.2162E-040.2225E-040.2102E-04
        18 2  48  1030526.  ET  0.2376E-020.2584E-020.2576E-02
        18 3        26819.
        18 4         7500.  EC  0.4363E-030.5732E-030.6277E-03





                            DAMAGE SUM FOR 18 MONTHS


                     LAYER 1   0.2177E-020.1972E-020.1708E-02

                     LAYER 2   0.1530E+000.1601E+000.1544E+00

                     LAYER 4   0.1371E+000.1792E+000.1846E+00
1                 ********** DESIGN LIFE OF PAVEMENT **********

              DAMAGE      CUMULATIVE     CRITICAL      DESIGN        DESIGN
   LAYER       TYPE         DAMAGE       POSITION    LIFE(YEARS)   REPETITIONS

     1      FATIGUE         1.000           1          1100.2      0.2640E+08

     2      FATIGUE         1.000           2             8.7      0.2086E+06

     4      DEFORMATION     1.000           3             7.4      0.1775E+06


                                                 LAYER 4 CONTROLS DESIGN LIFE
```

Table A.2 Continued

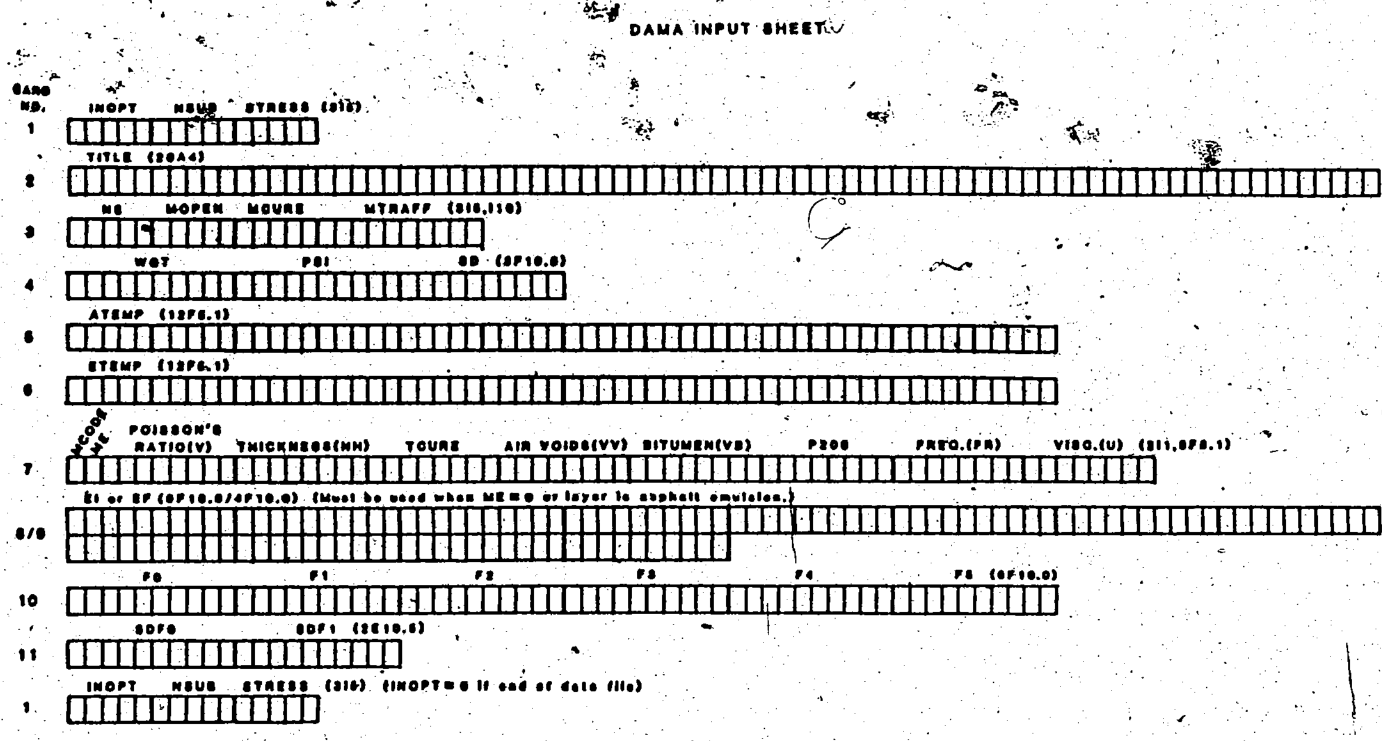


Fig. A.1 DAMA Input Sheet (Courtesy of the Asphalt Institute)

APPENDIX B

CASE STUDY - TRAFFIC, MATERIALS AND ENVIRONMENTAL INPUTS

B.1 Existing Pavement

The pavement cross-section selected for the case study is a test section within the Primary Highway 2 located in central Alberta. This test section was constructed during the period of 1964-1966 and has the following existing pavement structure:

100 mm of 16 000 μm topsize ACP (1965-1966)

50 mm of 20 000 μm topsize asphalt stabilized base course (1964-1965)

50 mm of 20 000 μm topsize granular base course (1964-1965)

250 mm of 40 000 μm topsize granular base course (1964-1965)

Field condition surveys in 1982 by Alberta Research Council and Alberta Transportation showed that severe rutting, in the order of 20-25 mm, existed within the boundaries of the northbound direction of the roadway section from km 2.4 to km 14.4. This necessitated the initiation of cold milling and recycling of a distance of 14.97 km of this highway section to restore its structural adequacy.

Since the performance history of the aforementioned highway section has been known for about 16 years i.e. from 1966-1982, it is considered appropriate to utilize such information and program DAMA of the Asphalt Institute to analyze that pavement section and check the applicability of the Asphalt Institute design methodology to Albertan conditions.

In the following sections, the information pertaining to the roadway section that has been used as inputs for program DAMA is presented.

B.2 Traffic Input

The traffic data needed as input for the analysis include the average monthly ESAL's per design lane, tire pressure of the wheel load system, and the spacing between the dual tires of the wheel load system. From the pavement inventory data base prepared by Alberta Research Council, daily ESAL's for primary and secondary highways can be obtained. Using such information, the distribution of the daily ESAL's for highway control section 2:28 was obtained for the period 1966-1982. Because there has been a trend of increasing ESAL's per day during this period, three different average daily ESAL's were selected as realistic estimates of the traffic that used the pavement facility during its service life. These averages are converted to monthly average ESAL's per design lane and used in the analysis. This information is contained in Table B.1.

Other traffic characteristics like the tire pressure and distance between the dual wheels are also selected based on information obtained by private communication with Christison of the Alberta Research Council. Typical values for such traffic parameters are shown in Table B.2.

B.3 Environmental Input

Two environmental inputs pertaining to the roadway section that are used in the analysis were obtained from a paper presented by Shields et al. (Reference 2 in Chapter 4) in the 1969 Convention of the Canadian Good Roads Association. These are the mean monthly air temperature profile and depth of frost penetration within the subgrade soil measured during the period 1966-1968 at the roadway section.

The mean monthly air temperatures shown in Table B.3, are used to reflect the influence of such parameter on the moduli values of the different pavement materials.

Furthermore, the different periods characterizing the seasonal variations in subgrade resilient moduli brought about by freezing and thawing cycles are obtained from the depth of frost penetration information contained in Fig. B.1. From such information it was found that freeze starts in October and last up to 4 months. This is then followed by a critical thaw period of one month and a subsequent 4 month period of recovery to attain the normal modulus again.

B.4 Materials Input

All materials inputs used in the analysis are based on information found in References 2-5 of Chapter 4, and presented in Tables B.4 to B.6.

Table B.4 contains the information pertaining to the asphalt concrete layer.

Table B.5 shows the information dealing with the aggregate base layer.

Table B.6 presents the inputs regarding the subgrade soil layer.

It is worth mentioning that since the 2 in.-thick asphalt bound layer is not actually an asphalt concrete layer, then it is assumed to be acting like an additional 2 in. aggregate base layer. In other words, the pavement section is considered as being comprised of a 4 in. asphalt concrete layer underlain by a 14 in. aggregate base layer.

Also from the depth of frost penetration information contained in Fig. B.1, both aggregate base and subgrade layers were assigned different monthly moduli values. These moduli were based on a freeze period of 4 months (November, December, January, February), a critical thaw period of one month (March), a recovery period of 4 months (April, May, June, July), and a period of 3 months (August, September, October) of normal modulus value.

B.5 Analysis

Seven runs of program DAMA were made using different values for monthly ESAL's per design lane, volume of air voids, and tire pressures. These three parameters were varied to reflect the changes which occurred in such variables during the service life of the pavement and thus provide a realistic estimate of the predicted design life of the pavement section. The values of ESAL's, VV's, and tire pressures (TP) used in the seven runs are as follows:

Run 1: Gives the damage accumulated within the first 6 months after construction. ESAL = 5400; VV = 8.3%; TP = 80 psi.

Run 2: Gives the damage accumulated between month 7 and month 12 after construction. ESAL = 6800; VV = 5.0%; TP = 80 psi.

Run 3: Gives the damage accumulated between month 13 and 18 after construction. ESAL = 6800; VV = 2.8%; TP = 80 psi.

Run 4: Gives the accumulated damage within the period of month 19 to month 24 after construction. ESAL = 6800; VV = 2.5%; TP = 80 psi.

Run 5: Gives the damage accumulated within the third year of service. ESAL = 6800; VV = 2.3%; TP = 80 psi.

Run 6: Gives the typical yearly damage for years 4 to 8 after construction. ESAL = 6800; VV = 2.2% ([2.2+1.8]/2); TP = 80 psi.

Run 7: Gives the typical yearly damage for years 9 to 16 after construction. ESAL = 12500; VV = 1.8%; TP = 95 psi.

Note that in all aforementioned runs, all other inputs are kept constant (except for ESAL's, VV's and TP's).

The results of the analysis and the computation of the predicted design life are mentioned elsewhere (specifically speaking, in Chapter 4).

Table B.1 Computation of the average monthly ESAL's per design lane - Hwy 2:28

- The following information was obtained from the pavement inventory data base - compressed version 1.6.

Year	Cumulative Daily ESAL's (one direction - two lanes)
1966	272
1968	858
1971	1709
1973	2332
1974	2756
1976	3799
1977	4589
1979	5839
1980	6464
1982	7814

- The following average monthly ESAL's/design lane were selected for the analysis.

Year	Average Monthly ESAL's/Design Lane
1966	$(272 \times 0.85^{*} \times 280^{**})/12 = 5400$
For 1966-74	$\left(\frac{2756 \times 0.85 \times 280}{8}\right) \times \frac{1}{12} = 6800$
For 1975-82	$\frac{(7814-2756) \times 0.85 \times 280}{8 \times 12} = 12500$

* 0.85 is the factor used to get the average daily ESAL's per design lane.

** 280 is the period, in days, during which the pavement structure will be mostly susceptible to the damage induced by traffic.

Table B.2 Other traffic characteristics of the roadway section - Hwy 2:28

- Spacing between the dual tires of the wheel load system is 34.3 cm (13.5 in.)

- Tire pressure values selected for the analysis are:

Period	Tire Pressure (psi)
1966-74	80
1975-82	95

Table B.3 Mean monthly air temperature profile measured at the roadway section during the period 1966-1968

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
15	20	20	30	60	80	80	70	60	45	25	20

Table B.4 Asphalt concrete layer input - Hwy 2:28

- % volume of voids (VV):
  - after construction = 8.3
  - after 6 months of service = 5.0
  - after 12 months of service = 2.8
  - after 18 months of service = 2.5
  - after 24 months of service = 2.3
  - after 36 months of service = 2.2
  - after 48 months of service = 1.8
- % volume of bitumen (VB)* = 12.85
- % agg. fraction passing the 80 μm sieve (P200) = 6.6
- asphalt viscosity @ 21°C (70°F, poises) (U)** = 0.6
- frequency of load application (FR)*** = 10.0 Hz

* Based on 5.2 bitumen content per 100 lb of dry agg.

** Computed from

$$U\Big|_{70^0F,\ 10^6 \text{ poises}} = 29508.2\ \text{Pen}_{77^0F}^{-2.1939}$$

where, Pen (@ 77°F) = 138 dmm

*** Assumed frequency.

Table B.5 Aggregate base layer input - Hwy 2:28

- Since the aggregate material was found to possess a good drainage potentiality (from soil classification), then a K1 value of 12000 is selected.
- Minimum K1 value (in March, the critical thaw month) is = 0.25 x 12000 = 3000
- Maximum K1 value (frozen subgrade support) is = (300/100) x 12000 = 36000
- The monthly distribution of K1 used in the analysis (i.e. in all runs) is as follows (x 10^3):

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
30.0	36.0	3.0	5.25	7.5	9.75	12.0	12.0	12.0	12.0	18.0	24.0

Table B.6 Subgrade layer inputs - Hwy 2:28

- From information pertaining to the roadway section (Ref. 2 in Chapter 4), the following are the properties of the subgrade soil:

Sieve Analysis (sieve sizes in μm)						Atterberg Limits		P.I.
40000	20000	5000	1250	315	80	L.L.	P.L.	
100	100	98.5	97.6	92.0	56.1	31.3	14.8	16.5

- From the above information, the Group Index (GI) is calculated using equation B.1 below:

$$GI = 0.2a + 0.005ac + 0.01bd \tag{B.1}$$

where, a = % passing the 80 μm sieve greater than 35% and not exceeding 75%, expressed as a positive whole number (0-40).
b = % passing the 80 μm sieve greater than 15% and not exceeding 55%, expressed as a positive whole number (0-40).
c = the portion of the numerical Liquid Limit greater than 40 and not exceeding 60%, expressed as a positive whole number (0-20).
d = the portion of the numerical plasticity index greater than 10 and not exceeding 30%, expressed as a whole number (0-20).

Using equation B.1, then the GI of the subgrade soil is,

$$GI_{SG} = 0.2(21) + 0.005(21)(0) + 0.01(40)(7) = 7$$

- From 1978-TRB correlation chart, shown in Fig. B.2, the CBR was found to be ≈ 7.

- The $M_{r(SG)}$ was then calculated using the relation

$$M_r = 1500\ CBR \tag{B.2}$$

$\therefore M_{r(SG)} = 1500 \times 7 = 50000$ psi.

- $M_{r(\text{critical thaw})} = r_{ct} \times M_{r(SG)}$ (B.3)

where r_{ct} is the reduction factor and = 0.44 for MAAT of 60°F (the climatic regime in which Hwy 2:28 is situated)
$\therefore M_{r(\text{critical thaw})} = 0.44 \times 10500 = 4620$ psi

- $M_{r(\text{frozen subgrade})} = 50000$ psi

Table B.6 Continued

- The monthly distribution of the subgrade moduli is as follows:

Jan	Feb	Mar	Apr	May	June
40125	50000	4620	5796	6972	8148

Jul	Aug	Sept	Oct	Nov	Dec
9324	10500	10500	10500	20375	30250

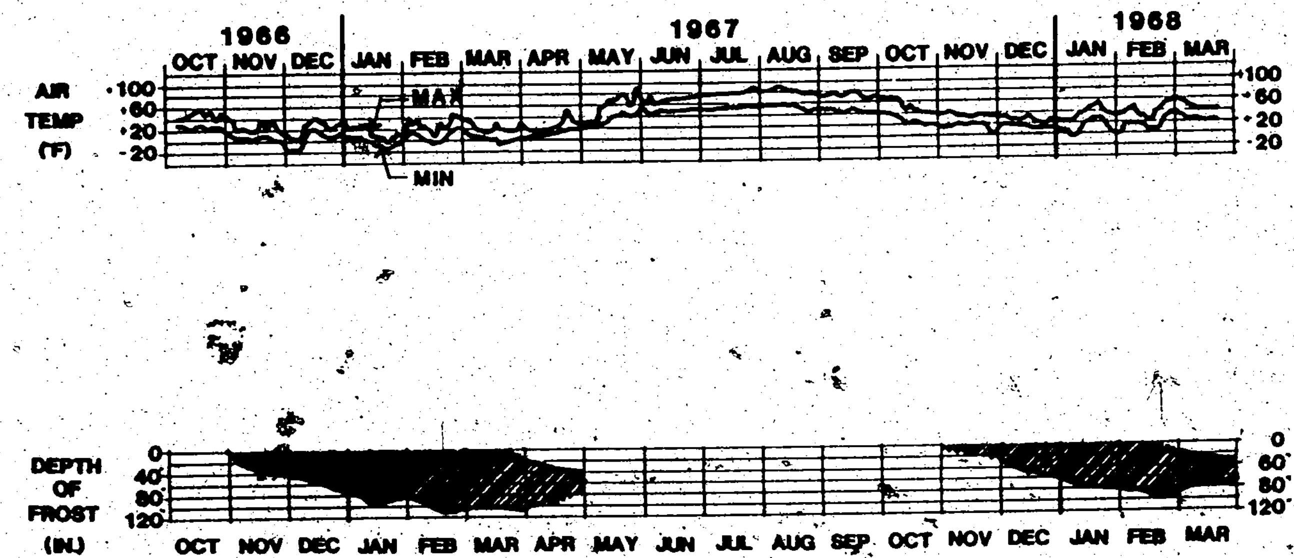
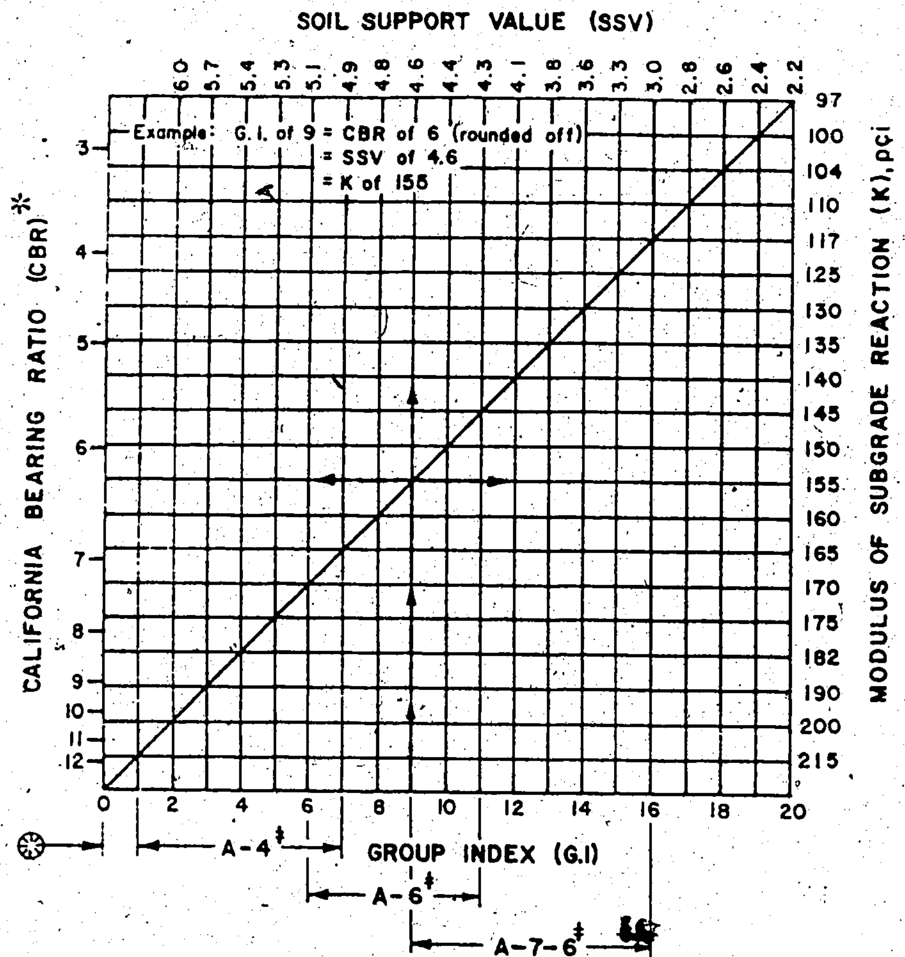


Fig. B.1 Depth of frost measured at the roadway section during the period 1966-1968



⊛ AASHTO Classes A-1, A-2 & A-3 lie below 0. SSV=6-10; K=200+.

† Usual range of AASHTO Classes.

* 5-1/2 Lb. hammer, 12" drop, 4 layers, 45 blows per layer, compacted at optimum moisture as determined by AASHTO T-99.

Fig. B.2 Correlation chart for subgrade strength

APPENDIX C

FACTORIAL ANALYSIS OF FACTORS AFFECTING PAVEMENT DESIGN LIFE

C.1 General

In this appendix, the statistical technique used to identify the significant design parameters that influence the prediction of pavement design life is presented along with a description of the analysis performed and the results evolved from that analysis.

The technique used in isolating and identifying the significant design parameters is named fractional factorial design. Using this technique, the investigator usually selects a fixed number of "levels" for each of a number of variables (or factors) and then runs experiments with all possible combinations.

Because the Asphalt Institute mechanistic-empirical model tested by the author in this investigation already exists, the statistical technique described above is used merely to check the sensitivity of the model output to small changes in the factors used as inputs to the model. This model, which took the form of a computer program (DAMA), utilizes several input parameters to predict the design life of a specific pavement structure. A set of some of these inputs is selected for inclusion in the analysis. The specific factors chosen were selected because of the many available research results that indicate their probable potential influences on pavement design life.

C.2 Formulation of the Factorial Analysis

A set of 8 factors is selected for inclusion in the analysis. These factors are:

1. Tire pressure of the wheel load system (TP).
2. Mean Annual Air Temperature of the geographical region in which the pavement structure is analyzed (MAAT).
3. Percent volume of air voids in the asphalt concrete layer (VV).
4. Percent volume of bitumen in the asphalt concrete layer (VB).
5. Expected frequency of traffic load application in Hz (f).
6. Type of bitumen used expressed in terms of the asphalt absolute viscosity at 21°C ($\eta_{70°F}$, 10^6 poises).
7. Quality of aggregate materials used in the base and subbase layers of the pavement system (K_1).
8. Resilient modulus of the subgrade layer (E_{SG}).

Two levels for each of these factors are chosen for the analysis and are shown in Table C.1. Because of the many runs, 2^8 runs, that will be needed to perform the analysis for the whole factorial, only a 1/16 fraction of the factorial is analyzed.

With the above considerations in hand, the analysis is reduced to a 2^{8-4}_{IV} fractional factorial design. The resolution IV design was selected based on the assumption that all three- and other higher order interactions are not of importance and can be neglected. This assumption was made to:

1. decrease the number of computer runs needed for the factorial analysis;
2. obtain only the main effects which will later be used as indicators of the significant factors to be included in the sensitivity analysis phase of the investigation.

Table C.2 shows the design matrix for the fractional factorial analysis. This matrix was formulated as follows:

1. A full 2^4 design was written for the first four variables 1, 2, 3 and 4 (TP, MAAT, VV and VB, respectively).
2. The four generators of the fractional factorial, namely

   5 = 123

   6 = 234

   7 = 124

   8 = 134

   are used to produce the columns of signs for the other four factors 5, 6, 7 and 8 (f, η, K_1, and E_{SG} respectively).
3. The columns of signs for the interactions 12, 13, 14, 15, 16, 17, and 18 are obtained by multiplying the signs of the individual columns. For example, column 12 is obtained by multiplying column 1 with 2

and so forth.

4. The column designated "Design Life" contains the output obtained from the analysis. This output is expressed in terms of either fatigue (F) or permanent deformation (D) life in years.

Note that in the design matrix shown in Table C.2, a plus sign indicates a high level of the factor, whereas a minus sign indicates a low level.

C.3 Calculation of Main and Other Effects

The main and two-factor interaction effects are calculated using the method of least squares. A multiple linear regression analysis, using MIDAS Statistical Package, was performed on the 16 variables contained in Table C.2. From the results of this analysis shown in Table C.3, the following effects were obtained.

Confounding Pattern	Effect
$L_1 \rightarrow 1$	2(-30.9) = -61.8
$L_2 \rightarrow 2$	2(-11.4) = -22.8
$L_3 \rightarrow 3$	2(-51.7) = -103.4
$L_4 \rightarrow 4$	2(28.5) = 57
$L_5 \rightarrow 5$	2(-3.5) = -7.0
$L_6 \rightarrow 6$	2(4.8) = 9.6
$L_7 \rightarrow 7$	2(16.4) = 32.8
$L_8 \rightarrow 8$	2(53.3) = 106.6
$L_{12} \rightarrow 12+35+47+58$	2(2.4) = 4.8

L_{13}	$\rightarrow$ 13+25+48+67	2(28.4) = 56.8
L_{14}	$\rightarrow$ 14+27+38+56	2(-48.5) = -97.0
L_{15}	$\rightarrow$ 15+23+46+78	2(14.6) = 29.2
L_{16}	$\rightarrow$ 16+28+37+45	2(-13.9) = -27.8
L_{17}	$\rightarrow$ 17+24+36+58	2(-4.9) = -9.8
L_{18}	$\rightarrow$ 18+26+34+57	2(-30.2) = -60.4

These effects are then plotted on a half-normal probability plot, shown in Fig. C.1. This plot is used to separate the significant effects from those resulting from random variation about the mean.

From the half-normal plot, it appears that all factors are significant. However, effects L_5, L_6, L_{12} and L_{17} were excluded since their magnitudes are relatively small compared to the other effects. This was considered necessary to enable estimating the standard error of the effects which will be used to judge the significance of the various variables. Using the four effects L_5, L_5, L_{12} and L_{17}, the standard error of the effect is calculated as follows:

$$\text{SE (effect)} = \sqrt{\frac{\Sigma\,[(\text{negligible effect})^2]}{\text{no. of negligible effects}}} \qquad \text{(C.1)}$$

$$\text{SE (effect)} = \sqrt{\frac{[(7.0)^2 + (9.6)^2 + (4.8)^2 + (-9.8)^2]}{4}}$$

$= 7.7$

The procedure used to check the significant factors using the above calculated standard error is illustrated in Table C.4.

C.4 Discussion of the Results

Among the eight factors analyzed, only six appears to have significant influence on the pavement design life. These factors are:

1. Mean Annual Air Temperature (MAAT)
2. Subgrade resilient modulus (E_{SG})
3. Volume of air voids, % (VV)
4. Volume of bitumen, % (VB)
5. Tire pressure (TP)
6. Quality of aggregate material used in the base and subbase layers of the pavement structure (K_1).

On the other hand, the type of asphalt used and/or frequency of load application has shown very little or no effect on pavement design life.

Due to the constraints mentioned earlier in Chapter 5, only MAAT, TP, E_{SG}, and VV were chosen to be included in the parametric evaluation part of the investigation. The details of this part of the study together with the results evolved are discussed elsewhere.

Table [illegible].1 Levels of design factors

Factor	Level (-)	Level (+)	
1. TP	70	120	(psi)
2. MAAT	7	29	(°C)
3. VV	3	10	(%)
4. VB	9	12	(%)
5. f	1	16	(Hz)
6. $\eta_{70°F, 10^6}$ poises	0.5	5.0	($\times 10^6$ poises)
7. K_1	8000	12000	(-)
8. E_{SG}	4500	22500	(psi)

Table C.2 Design matrix for the 2_{IV}^{8-4} fractional factorial

Run	1	2	3	4	5	6	7	8	12	13	14	15	16	17	18	Design life (years) F	D
1	−	−	−	−	−	−	−	−	+	+	+	+	+	+	+	31.4	1.0*
2	+	−	−	−	+	−	+	+	−	−	−	+	−	+	+	134.4*	1211.1
3	−	+	−	−	+	+	+	−	−	+	+	−	−	−	+	105.8	17.1*
4	+	+	−	−	−	+	−	+	+	−	−	−	+	−	+	53.1*	583.7
5	−	−	+	−	+	+	−	+	+	−	+	−	−	+	−	8.8*	1480.1
6	+	−	+	−	−	+	+	−	−	+	−	−	+	+	−	1.5*	1.7
7	−	+	+	−	−	−	+	+	−	−	+	+	+	−	−	22.9*	674.8
8	+	+	+	−	+	−	−	−	+	+	−	+	−	−	−	0.9*	4.7
9	−	−	−	+	−	+	+	+	+	+	−	+	−	−	−	402.0*	1122.4
10	+	−	−	+	+	+	−	−	−	−	+	+	+	−	−	128.4	7.3*
11	−	+	−	+	+	−	−	+	−	+	−	−	+	+	−	258.0*	543.7
12	+	+	−	+	−	−	+	−	+	−	+	−	−	+	−	94.2	8.2*
13	−	−	+	+	+	−	+	−	+	−	−	−	+	−	+	4.3	1.0*
14	+	−	+	+	−	−	−	+	−	+	+	−	−	−	+	2.9*	336.7
15	−	+	+	+	−	+	−	−	−	−	−	+	−	+	+	6.3	4.4*
16	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	11.8*	808.2

* Indicates the value used in the analysis.

Table C.3 MIDAS output

<REG OPTIONS=MEANZERO VAR=16:1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,>

LEAST SQUARES REGRESSION

ANALYSIS OF VARIANCE OF 16.V16 N= 16 OUT OF 16

SOURCE	DF	SUM SQRS	MEAN SQR	F-STAT	SIGNIF
REGRESSION	15	.19556 +6	13038.	.23846	.9415
ERROR	1	.54674.	54674.		
TOTAL	16	.25024 +6			

OPT: MEANZERO R-SQR=1.00000 SE= 233.83

VARIABLE	PARTIAL	COEFF	STD ERROR	T-STAT	SIGNIF
1.V1	-.46784	-30.944	58.456	-.52935	.6901
2.V2	-.19151	-11.406	58.456	-.19512	.8773
3.V3	-.66236	-51.681	58.456	-.88410	.5391
4.V4	.43816	28.494	58.456	.48744	.7113
5.V5	-.06051	-3.5437	58.456	-.60622 -1	.9615
6.V6	.08173	4.7937	58.456	.82006 -1	.9479
7.V7	.27022	16.406	58.456	.28066	.8258
8.V8	.67364	53.281	58.456	.91147	.5295
9.V9	.04092	2.3937	58.456	.40949 -1	.9739
10.V10	.43754	28.444	58.456	.48658	.7117
11.V11	-.63818	-48.456	58.456	-.82893	.5594
12.V12	.24280	14.631	58.456	.25029	.8439
13.V13	-.23104	-13.881	58.456	-.23746	.8516
14.V14	-.08427	-4.9437	58.456	-.84572 -1	.9463
15.V15	-.45952	-30.244	58.456	-.51737	.6960

Table C.4 Determination of significant factors

- The hypothesis to be tested here is

$$H_o : |\text{effect}| = 0$$
versus $$H_1 : |\text{effect}| > 0. \quad \text{(i.e. one-sided test)}$$

- The table below contains the estimates of the effects together with their standard errors and t-statistic.

Factor	Estimated effect ± SE(effect)	$t_{calc.}$-statistic $\left(= \frac{\lvert\text{effect}\rvert}{SE(\text{effect})}\right)$
L_1	-61.8 ± 7.7	8.03
L_2	-22.8 ± 7.7	2.96
L_3	-103.4 ± 7.7	13.43
L_4	57.0 + 7.7	7.40
L_5	7.0 ± 7.7	0.91*
L_6	9.6 ± 7.7	1.25*
L_7	32.8 ± 7.7	4.26
L_8	106.6 ± 7.7	13.84
L_{12}	4.8 ± 7.7	0.62*
L_{13}	56.8 + 7.7	7.38
L_{14}	-97.0 ± 7.7	12.60
L_{15}	29.2 ± 7.7	3.79
L_{16}	-27.8 ± 7.7	3.61
L_{17}	-9.8 ± 7.7	1.27*
L_{18}	-60.4 ± 7.7	7.84

- From statistical tables,

$$t_{tab.(0.05,4)} = 2.132 \quad \text{(for a one-sided test)}$$

where, 0.05 = α → 95% confidence level
and, 4 = ν → degrees of freedom.

- Now significant factors are those whose $t_{calc.} > t_{tab.}$. Other insignificant factors are marked with asterisks in the table above.

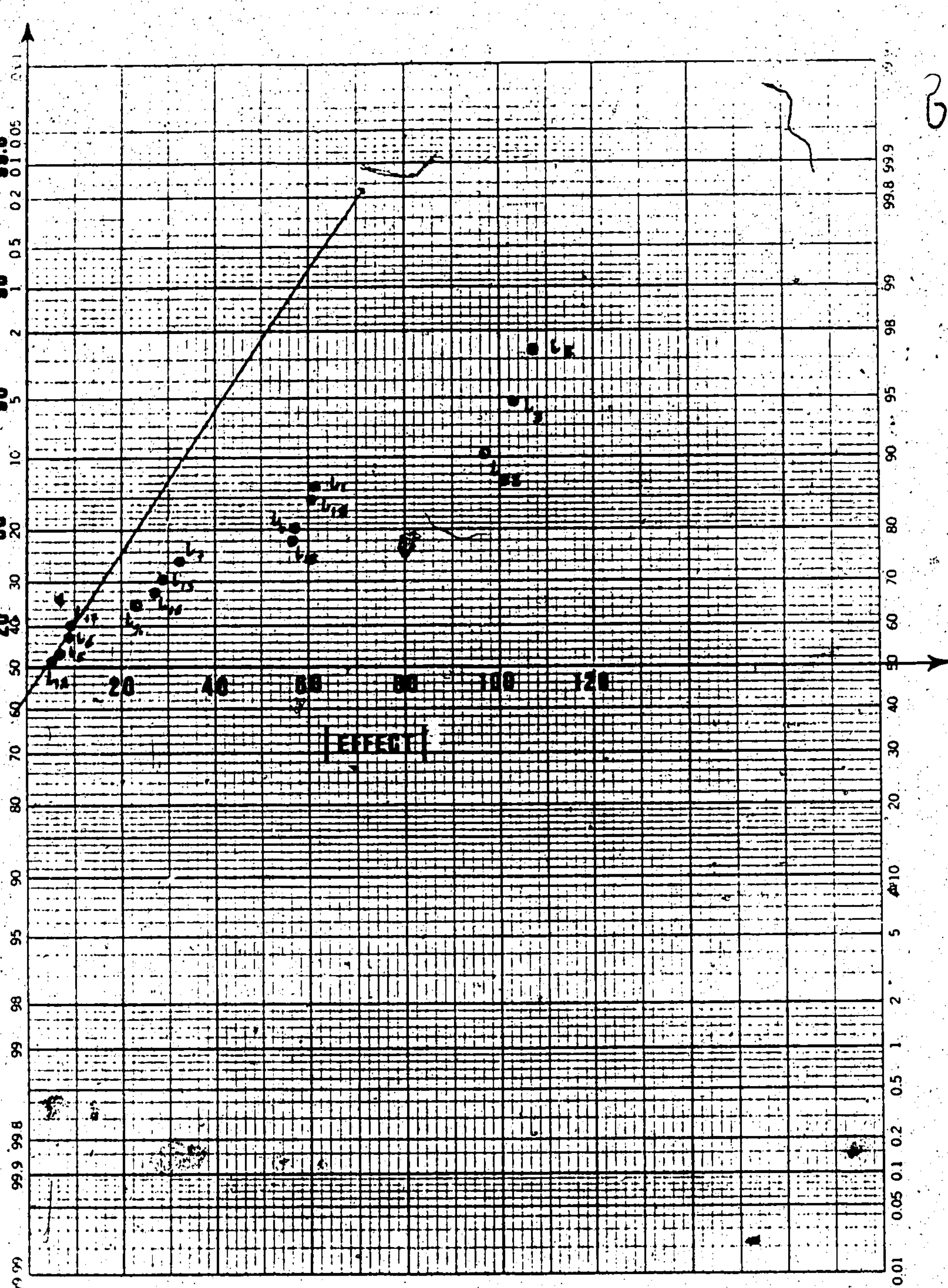


Fig. C.1 Half-normal plot of effects

THE UNIVERSITY OF ALBERTA

A PARAMETRIC EVALUATION OF A MECHANISTIC-EMPIRICAL DESIGN PROCEDURE FOR ASPHALT CONCRETE PAVEMENTS

by

WALAA ELDIN IBRAHIM KHOGALI

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

SPRING 1988

ISBN 0-315-42733-7

ASPHALT INSTITUTE
EXECUTIVE OFFICES AND RESEARCH CENTER
Asphalt Institute Building | College Park, Maryland 20740 | Telephone: 301/ASPHALT (277-4258)

October 27, 1987

Mr. Wala E. I. Khogali
Graduate Student
Civil Engineering Department
University of Alberta
Edmonton, Alberta, CANADA
T6G-2G7

Dear Mr. Khogali:

Permission is granted for you to include some of the figures, tables, etc., in MS-1, RR-82-2 and CP-1.

Please give a courtesy by-line to the Asphalt Institute.

Sincerely,

James A. Burton, P.E.
Principal Engineer

JAB/lf

University of Alberta
Edmonton

Department of Civil Engineering

Canada T6G 2G7

220 Civil/Electrical Engineering Building,
Telephone (403) 432-4235

December 30, 1987

Mr. Walaa Khogali
Graduate Student
Department of Civil Engineering
University of Alberta

Dear Walaa,

With regard to some Figures used in your thesis entitled "A PARAMETRIC EVALUATION OF A MECHANISTIC-EMPIRICAL DESIGN PROCEDURE FOR ASPHALT CONCRETE PAVEMENTS", my permission is given to use the information obtained from References 2 and 3 of Chapter 4 on page 145 of your thesis. Your Figures 4.3 and 4.4 were redrawn from the above noted references. Figure 4.1 has also been redrawn from information given in reference 4, and permission is also given for use in this thesis.

Sincerely,

K.O. Anderson
Professor of Civil Engineering

THE UNIVERSITY OF ALBERTA

RELEASE FORM

NAME OF AUTHOR: WALAA ELDIN IBRAHIM KHOGALI

TITLE OF THESIS: A PARAMETRIC EVALUATION OF A MECHANISTIC-EMPIRICAL DESIGN PROCEDURE FOR ASPHALT CONCRETE PAVEMENTS

DEGREE: MASTER OF SCIENCE

YEAR THIS DEGREE GRANTED: 1988

Building & Road Research Institute
University of Khartoum, P.O. Box 321
Khartoum, SUDAN

Date:

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled A Parametric Evaluation of a Mechanistic-Empirical Design Procedure for Asphalt Concrete Pavements submitted by Walaa Eldin Ibrahim Khogali in partial fulfillment of the requirements for the degree of Master of Science.

....
(Supervisor)

R. G. Cheng ....

....

....

Peter ....

Date: Dec 31/87

DEDICATION

To my dearest parents, wife and son.

ABSTRACT

Mechanistic-empirical thickness design methodologies for flexible pavements have gained wide popularity in recent years. This is come from the facts that they can more rationally represent the pavement structure and they have more flexibility in incorporating new materials and/or analytical techniques in designing or evaluating pavement systems. This investigation aimed at evaluating the applicability of one of these theoretically-based design procedures, and achieving a better understanding of the influence of the various design parameters on the service life of asphalt concrete pavements. The Asphalt Institute design procedure developed between 1977-1981 was the method selected for this investigation.

The study was separated into two distinct phases. In phase I, a pavement section located in central Alberta was used to validate the applicability of the design procedure to Albertan conditions. Phase II was a more detailed investigation dealing with a parametric evaluation of the various design parameters. In this phase, a set of design factors was analyzed to obtain quantitative measures of the influence of each of these factors on the pavement design life.

Results from phase I indicated good potentials for implementing the design method in Alberta. However, since only one pavement section is involved in this phase, it is recommended that further verification is needed before a

general conclusion can be drawn with respect to that regard.

Phase II identified four factors to be mostly significant in affecting the pavement design life. These factors are the subgrade modulus, mean annual air temperature of the climatical region in which the pavement structure is situated, percent volume of air voids in the asphalt concrete mix, and the tire pressure of the wheel load system. Other factors appear to have little or no effect on pavement design life.

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to Professor K.O. Anderson for his invaluable guidance and support throughout the development of this thesis. Without his direction and suggestions the objectives of this research could not have been met.

I would like also to extend my sincere thanks to:

my wife, Safaa, for her support, understanding and scrifices made,

Dr. J.T. Christison, from Alberta Research Council, for his assistance in providing the necessary information regarding traffic estimation in Alberta,

Mr. J.A. Burton for his permission to include some of the Asphalt Institute illustrations in the thesis,

Ms. Nola Shaw for typing the manuscript,

many individuals within the Civil Engineering Department of the University of Alberta for their assistance and cooperation.

Moreover, I would like to acknowledge the financial support provided by CIDA/U of A Scholarship during my study.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

1.1 General

For the past three decades, a considerable amount of research has been directed toward developing more rational procedures for designing and evaluating asphalt pavement structures. The need for developing such procedures has been substantiated largely by the depletion of quality aggregate, the increasing cost of asphalt cement, the introduction of new vehicles with different than usual axle loads and tire pressures, and the effective use of the pavement facility without the need for performing major maintenance activities at short time intervals.

With developments in both analysis techniques and materials testing procedures of the past 25 years, several mathematical models have been postulated that correlate the pavement mechanical responses to loading with the different distress modes deemed responsible for unacceptable pavement performance.

Among the many available mechanistically-based design methodologies, those that are based on the classical elastic theory received most attention. This is primarily due to two facts: (a) the simplicity involved in using such procedures; and (b) the reasonable reliability of such procedures in predicting the actual pavement response to load.

However, since all aspects of pavement design involve inherent variations that cannot be measured adequately or predicted in the deterministic sense, a parametric evaluation of such aspects of any of the aforementioned design procedures constitute an important consideration that warrant further research.

1.2 Purpose and Objectives of the Investigation

The overall purpose of the investigation was to evaluate the potential use of a mechanistic-empirical thickness design procedure for asphalt concrete pavements under extreme climatological conditions. Specifically speaking, the objectives were:

1. To validate the applicability of the mechanistic-empirical design approach to predict pavement response to load under typical Albertan conditions.
2. To identify the significant design parameters that influence the design thickness requirements.
3. To quantify the individual and/or interactive effects of such factors on the pavement design life.
4. To provide guidelines for future implementation of the design procedure in Alberta.

1.3 Scope of the Investigation

The scope of the investigation was limited to conventional asphalt concrete pavements situated under either extreme hot or cold climatic environment.

Of the many available theoretically-based solutions for predicting pavement responses in layered systems, only the computer program DAMA developed by the Asphalt Institute was used.

For all phases of the investigation, only one pavement section within the primary highway 2 located in central Alberta was included. Attempts were made to utilize all the available information pertaining to the pavement section under consideration to provide the inputs needed for the computer program DAMA.

Six significant design parameters were identified which influence design life. These factors are: tire pressure of wheel load system, percent volume of voids in asphalt concrete mix, subgrade resilient modulus, mean annual air temperature of the particular regime where the structure is situated, percent volume of bitumen, and aggregate quality factor K1. Only the first four of these factors were included in the parametric study. This was found necessary to reduce the large number of computer runs needed if all factors were to be considered.

1.4 Organization of the Thesis

This study is divided into 6 chapters and 3 appendices.

The Introduction, together with the purpose, objectives, scope, and organization of the thesis is presented in Chapter 1.

In Chapter 2, a critical review of the literature dealing with mechanistic-empirical design methodologies is described.

The Asphalt Institute thickness design procedure is summarized in Chapter 3. Appendix A contains a general description of program DAMA input and output formats together with some examples illustrating how the program can be used for analyzing a specific pavement cross section.

Chapter 4 describes the procedure used in analyzing an existing pavement section within a primary highway located in central Alberta. The results evolved from this study are also presented and discussed. Materials, environmental, and traffic inputs used in the analysis of the case study are contained in Appendix B.

Chapter 5 is entirely devoted to the presentation and discussion of the results of the parametric study. This chapter contains both phases of the sensitivity analysis. In the first part of this chapter a general description of the procedure used in screening out the significant design parameters affecting the thickness design requirements is given. The results of this analysis are then presented and discussed. The second phase of the sensitivity analysis dealing with the parametric evaluation of the individual significant factors is described in the second part of this chapter. Appendix C contains the detailed information pertaining to the screening analysis part of the parametric study. In this Appendix, the details of the statistical technique used for identifying the significant design parameters affecting design life are given.

Chapter 6 presents the summary and major conclusions of this investigation. This chapter also contains the recommendations evolved from the results of the study.

2.0 LITERATURE REVIEW ON MECHANISTIC-EMPIRICAL DESIGN METHODOLOGIES

2.1 General

This chapter reviews the literature dealing with the development of mechanistic pavement design methodologies and is divided into three major sections. The first section reviews the currently available mechanistic models that use analytically-based methods for the structural design of asphalt pavements. The second and third sections review the input variables and design criteria commonly used in such design procedures.

2.2 Mechanistic Design Models

Pavements are considered extremely complicated physical systems that involve interactions of a large number of complex and interrelated factors. Therefore, a systematic and logical approach must be followed in developing a rational and generally applicable method for pavement design. Such an approach had been the concern of many investigators particularly during the past three decades. The efforts of all those researchers had been directed toward developing a mechanistic basis for designing and evaluating pavement structures. Many theoretically based design methodologies utilizing linear-elastic, non-linear-elastic, and viscoelastic layered system theories were developed and are currently under evaluation by major highway organizations for potential future use (1).

A mechanistic pavement design methodology is an approach whereby the response of a pavement system is postulated to be closely related to the mechanistic responses of stresses, strains, and displacements induced in the pavement under traffic loadings and/or environmental influences. Such an approach follows a framework like that shown in Fig 2.1. Essentially, the thickness design involves the selection of a pavement structure in which an appropriate combination of materials and layer thicknesses is chosen to mitigate the various forms of distress indused in the pavement from traffic and environmental related factors. Such a methodology has been incorporated into different design formats including: (a) design charts like those developed by Shell researchers (2,3,4) and the Asphalt Institute (5,6); (b) computer solutions like the DAMA program (6) or the use of the BISAR program (7); (c) and catalogues of design like the one developed by NITRR of South Africa (8) or the one developed by LCPC of France (9). However, since all these design models involve an empirical part, they are usually referred to as mechanistic-empirical procedures (10).

The merits of adopting a mechanistic-empirical design approach over an empirical one can be viewed as threefold. First, a mechanistically-based approach permits the effective and expeditious utilization of existing materials and the incorporation of new materials into the design process. Second, it relates the pavement prime responses under load to the distress design criteria in a rational and applicable manner. Third, the design of new pavements under different sets of loading and/or environmental conditions can be achieved using mechanistic methodologies whereas difficulties and erroneous designs may arise upon extrapolating empirical design correlations beyond the conditions for which those were originally developed. There are, however, a number of pitfalls involved in using theoretical techniques. Such pitfalls include input data reliability, a tendency to blindly use computer programs as "black boxes" with complete trust in the results, and a lack of appreciation of the sensitivity of the design factors. Thus while mechanistic analysis provides a very powerful, potential tool for making pavement design more fundamental in nature, the disadvantages should clearly be recognized.

2.2.1 Layered System Analysis

Pavements are considered by many designers as multi-layered systems that can be analyzed using the principles of continuum mechanics. Strictly speaking, there are three factors that must be considered in predicting the response of multi-layered pavement systems; these are: (a) the stress-strain behavior of the materials, (b) the initial and boundary conditions of the problem, and (c) the partial differential equations that govern the problem.

Utilization of the principles of continuum mechanics has evolved three basic system approaches for the design of asphalt pavements. Those are: (a) linear and non-linear elastic multi-layered systems, (b) finite-element approaches, and (c) viscoelastic approaches. Detailed discussion on the principles and limitations of each of these models will be pursued in the following sections.

2.2.2 Linear Elastic Models

The response of pavement systems to wheel loadings has been of interest since 1926 when Westergaard (11) used elastic layered theory to predict the response of rigid pavements. Fig. 2.2 illustrates the general concept of a multilayered elastic system. In order to simplify the analytical solution for pavement systems, the following assumptions were made by the several investigators who adopted the elastic approach:

1. Each layer acts as a continuous, isotropic, homogeneous, linearly elastic medium infinite in the horizontal extent.
2. The surface loading can be represented by a uniformly distributed vertical stress acting over a circular area.
3. The interface conditions between the layers can be represented as either perfectly smooth or perfectly rough.
4. Each layer is continuously supported by the layer beneath.

5. Inertial forces are negligible.
6. Deformations throughout the system are small.
7. Temperature effects are negligible.
8. Materials' properties remain constant i.e. time effects are negligible.

These assumptions constitute the basis as well as the limitations of the elastic layered system approach.

In the multilayered elastic approach, pavement structures were considered as composed of one or more layers resting on a semi-infinite homogeneous elastic mass.

Boussinesq (12) was the first to formulate solutions for stresses induced in a semi-infinite, homogeneous, isotropic, elastic solid due to a vertical load applied at the surface. This solution was extended to the case of load uniformly distributed over any finite area by Love (13). Particular solutions have been reported by many other investigators (14,15,16).

Although Burmister's work (17,18) provided analytical expressions for stresses and displacements in two-and three-layer systems, Fox (19) and Acum and Fox (20) produced the first extensive tabular summary of normal and radial stresses in three-layer system at the intersection of the plate axis with the layer interfaces. Jones (21) and Peattie (22) subsequently expanded these solutions to a much wider range of solution parameters.

Work of all those authors together with the work of many others (23,24) have evolved comprehensive tables and charts of influence values for the solutions of 2-, 3-, and 4-multi-layered systems subjected to uniform circular loadings. Unfortunately, working with such design charts is a tedious job even for a 3-layered system, and the development of such charts for 4- or more layered systems is not practical. Rather, computer programs for pavement analysis that are based on the classical elastic theory comprised an attractive alternative, and many have been set up and implemented sucessfully by research departments of many highway organizations.

2.2.3 Finite-Element Approach

The finite element method, as compared to the classical elastic layered approach, offers the capability of more realistically modelling the pavement structure. In contrast to the classical elastic approach, the finite element approach considers the pavement system as composed of a large number of grid elements. Each of these elements can be given independent anisotropic material properties, and the layers need not be infinite in width. In addition, solutions of the finite element formulation for stresses, strains, and displacements are obtained for each element in the grid, whereas for the classical elastic approach, such quantities must be calculated individually at each desired point.

Usually modelling a pavement structure according to the finite element method is done by approximating the pavement system problem as one of axial symmetry, and a cylindrical coordinate system represented by r, θ, and z is used. A layered pavement structure may then be idealized as an assemblage of finite number of discrete triangular or rectangular ring-shaped elements as those shown in Fig.2.3. Each adjacent element in the system comes close together at a common point and is interconnected by frictionless pins called nodes.

The structural stiffness properties of each element can be determined by the application of the energy principles, as stated by Zienkiewiez and Cheung (25) using an appropriate displacement function and the usual elastic relations for stresses, strains, and displacements.

The major disadvantage of the finite-element method as compared to the classical elastic approach is the complexity of the analysis and the increased cost of computer time involved.

2.2.4 Limitations of The Layered Theory

Both the classical and the finite-element theories have certain limitations which must not be overlooked. Some of these limitations can be listed as:

1. Normally ,pavements structures are modelled as axisymmetric solids. This means that both load and pavement geometrics are symmetrical about a common centerline. Unfortunately, the effects of wheel loads applied close to a crack or pavement edge can

not be analyzed by such methods. A solution to the problem of axisymmetry assumption was suggested by Pichumani (26) who used an extended 2-dimensional finite-element program that had the capability of modelling pavement structrues when the load is applied close to the pavement edge. But the trade-off with that program is that, it requires too much computer time to be suitable for general application in a routine design method.

2. Another limitation is the negligence of the inertial forces within the pavement system. The inertial force is the force on a small element caused by a dynamic loading equal to the mass of the element times the acceleration.
3. Neglecting the effects of vibrations is also a limitation of the elastic approach. Although such an assumption is not a bad one for cohesive soils, yet for cohesionless materials compacted at relatively lower densities, neglecting such effects may lead to densification that would cause rutting and changes in material properties.
4. Another major limitation of the classical methodology is the selection of the appropriate moduli values to be assigned to the different materials comprising the pavement stucture. Such moduli values were found to be function of the stress states that exist within each layer.

Due to variations in the stress states, the dynamic moduli values also vary with both depth and lateral position in each layer. Therefore, assigning a single moduli value to represent each material in the pavement system is probably incorrect and can only be done as an approximation.

The limitations listed above can, however, be dealt with more flexibly by using certain advanced finite-element programs. But as mentioned before, the trade-off will be the high computer time costs of running such programs. Another attractive alternative suggested by many investigators like Hicks (27) and Kasianchuk (28) for solving the problem of assigning appropriate moduli values for the different paving materials,considered using a non-linear,iterative elastic layered solution to represent the behavior of these materials. In this approach, the base and subgrade layers were divided into several fictitious layers for better accuracy. Then within each of these layers, a modulus value that is dependent on the average stress state that exists in the vicinity beneath the wheel loadings is assigned to that layer. This modulus is then used for the computation of stresses which are subsequently utilized to obtain a new modulus. The process is repeated until the assumed value of the modulus is within 5 percent of the calculated one.

A third alternative that might overcome the whole problem of the elastic approach limitations is to use a viscoelastic analytical model for designing asphalt concrete pavements. A brief description of such an approach will be outlined in the next section.

2.2.5 Viscoelastic Models

For a number of years, several researchers have recognized that the constituent materials of a pavement system have time dependent stress-strain properties due to consolidation and creep (29,30). Those time effects have been neglected by the elastic layered approach which reasonably assumes that the loads are applied instantaneously to the pavements. However, under conditions of static loading, which may approximate the case for vehicles moving with low speeds, time effects can no longer be neglected and a more realistic representation of the pavement component materials can be obtained by adopting a viscoelastic layered model. Such models were based on the hypothesis that certain materials under load behave like viscous fluids i.e. due to a constant load with respect to time, a time -dependent deformation results.

In order to simplify the mathematical difficulties associated with the solutions of the viscoelastic models, linear viscoelasticity was assumed by many researchers who used that approach. In those solutions, the time is considered as a third variable in the extended Hooke's law. The constants, however, are assumed to be independent of time, load, stress, and strain.

In the linear viscoelastic approach, single discrete models of springs and dashpots in series and/or in parallel were used to characterize the viscoelastic material behavior.

The solutions of linear viscoelastic models were made possible by the formulation of the correspondence principle first developed by Lee (31) in 1955 for isotropic media. This principle was later extended by Biot (32) to include anisotropic media. Using this principle, all time varying quantities were replaced by their Laplace transformed equivalents to obtain an associated elastic problem which can then be solved and transformed back, using the inverse Laplace transformation solution,to obtain the required viscoelastic solution.

Many investigaters have attempted to validate the viscoelastic approach for predicting pavement performance. Worth mentioning among these are the efforts reported by Barksdale and Leonard (33), Monismith and Secor (34), and Kenis (35).

Like the elastic layered approach, several computer programs that use the viscoelastic principle were developed and implemented for use by many highway research organizations. An example of such programs is the VESYS structural subsystem of the Federal Highway Administration (35). Several versions of this program, that can use either elastic or viscoelastic materials input to predict the performance of a pavement in terms of its present serviceability index(PSI), are available.

2.2.6 Model Sélection

Several studies by many workers like Burmister (36), Barksdale and Leonards (33), Brown and Pell (37), and Kenis et al (38) have shown that each of the above described approaches gave reasonably good predictions of deflections, stresses, and strains in multi-layered pavement systems. Therefore, any of these approaches can be used to adequately predict the pavement system reponse to traffic and environmental related factors.

The selection of a model to be used should depend on whether there are any benefits gained in using a more sophisticated approach. Furthermore, the elastic layered approach has proved to be working adequately, from a practical stand point, in estimating actual pavement response to load, and in predicting actual pavement performance. Therefore, the emphasis in this study will be placed on using a multi-layered elastic model and no further discussion of the other models will be mentioned.

2.3 Design Input

The design input variables needed for use in a mechanistic design methodology can be listed in six categories: traffic, environment, materials characterization, construction, structural, and maintenance. The construction, structural, and maintenance inputs are those that are usually controlled by the designer or the pavement management team and that basically affect the load carrying capacity of the pavement as designed, constructed, and modified in service. Traffic and environmental variables are those that basically impose a load on the pavement structure, whereas materials variables are those that characterize the pavement response to load. Another important concept is that of variability. All aspects of pavement design involve inherent variations that can not be measured adequately or predicted in the deterministic sense (39,40). Because of this, exact determination of traffic to be carried by the pavement structure at any time during its service life is not possible, nor is it possible to predict precisely the prevailing environmental conditions. Therefore, pavement design subsystems must consider the stochastic variations of inputs, models, and model coefficients to predict a design in terms of the required levels of reliability.

For the purposes of this study only traffic, environmental, and materials variables will be reviewed. Other inputs i.e. construction, structural, and maintenance will not be discussed any further.

2.3.1 Traffic Variables

Traffic data is essential for investment programming and design purposes. It is also required for certain aspects of construction and maintenance functions.

Traffic variables include all those related to the vehicle loads applied to the pavement. Such variables include total vehicle load, wheel load, tire pressure, wheel or gear configuration, lateral distribution of the wheel loads, number of application of wheel loads, lane and directional wheel load distributions, sequence of load application of the various types, load type, and variability of traffic.

Although evaluation of the actual amount of traffic that may use the design facility at any time is difficult, yet, the problem can still be tackled in any of the following ways as pointed out by Hudson (41) and many others:

(a) Design the pavements for the worst expected load conditions.

(b) Equate loads to equivalent load applications of a base load.

(c) Predict traffic patterns and run a design analysis for the spectrum of load conditions to be encountered.

2.3.1.1 General Approaches Used for Traffic Input Estimation

Historically, two trends were followed in estimating traffic loadings for pavement design purposes. Those generally encompassed: (a) the direct analysis of the actual mixed traffic loads encountered, and (b) the introduction of the equivalent axle load concept which involve converting all mixed traffic into equivalent load applications of a standard single axle load.

The approach of analyzing the actual mixed traffic to be encountered during the pavement service life, was successfully employed by Monismith et al (42) in the course of their study of the asphalt mixtures behavior in repeated flexure. This method, however, had two basic limitations: (a) first, the tedious computational efforts to be encountered in the analysis, and (b) second, the detailed level of information required regarding traffic pattern distributions.

The second and more attractive approach followed by many investigators and highway departments, was based on the concept of converting all mixed traffic loads into an equivalent number of a standard axle load applications, usually taken as 80 kN (18000 lb.) single axle load (ESAL). The basis for load equivalency can either be calculated or measured stress, strain, or deflection at some point in the pavement structure, or equal conditions of distress and loss of serviceability.

The load equivalency concept implies that an axle load is always a multiple or fraction of the equivalent single axle base load for all situations. Strictly speaking, this is not true because equivalency varies with each particular combination of pavement layer thicknesses, layer properties, and loads. For many designs, however, these variations are not excessively large, and for all practical purposes they can reasonably be ignored , especially, in view of the large errors usually encountered in forecasting traffic.

Traditionally, converting mixed traffic into equivalent axle load applications is usually accomplished by the use of the appropriate equivalency factors. The use of such factors to represent the effect of traffic on the deteriorating serviceability of both flexible and rigid pavements was first introduced during the analysis of the AASHO Road Test project in 1960. The definition of these factors, as stated by the AASHO committee on pavement design (43) is "the number of applications of an 18000-lb single-axle load required to produce the same effect on the serviceability of a pavement as one application of a particular axle load".

Scrivener et al (44) later expanded the use of the AASHO load equivalence factors to include mixed traffic based on the assumption that the order of accumulation of traffic is immaterial to the final accumulated damage induced in the pavement. This assumption, although strictly not true, was found to be reasonably valid for "well mixed" traffic that usually could be expected to be encountered during the service life of any pavement structure.

Detailed applications of the concept of the load equivalency in handling traffic inputs were described by many authors in the literature. The Asphalt Institute (45) for example, has developed load equivalence factors for converting measured axle loads (from weight ~~or loadometer~~ surveys) into equivalent number of an 18000-lb single axle load applications.

Another method for converting mixed traffic into ESAL's, as described by Van Til et al (46) in NCHRP report 128, involved classifying the traffic into three basic groups: passenger cars, single-unit vehicles, and multiple-unit vehicles. A weighted equivalence factor for each vehicle type is then used to convert to equivalent axle loads. The equivalent axle loads for each type is then distributed to the design lane. The basic equation describing this is,

$$ESAL = DP\ [(c_1 PC\ .P)+(c_2 SU\ .S)+(c_3 MU\ .U)]/10^6 \quad (2.1)$$

where, DP = design period, years,

c_1 = constant for passenger cars,

c_2 = constant for single units,

c_3 = constant for multiple units,

PC = total passenger cars ADT (two directions) for the design period,

SU = total single-units ADT (two directions) for the design period,

MU = total multiple-units ADT (two directions) for the design period,

P = % of passenger cars ADT in design lane,

S = % of single-units ADT in the design lane, and

U = % of multiple-units ADT in the design lane.

The values of the constants c_1, c_2, c_3 vary, depending on pavement type and highway classification.

A basic limitation of this method is that, if the axle load distribution changes, the predictions could be substantially erroneous.

In addition to the previous method mentioned, Van Til et al (46) also described some other six methods that utilize loadometer data and certain equivalence factors for converting mixed traffic into equivalent axle loads. Derdeyn (47) also suggested a procedure whereby mixed traffic is converted into equivalent axle load applications by using the percent trucks in the ADT and the axle distribution of traffic patterns as supplied by loadometer information.

The Shell method for pavement design (2) uses another approach for estimating traffic input. In this approach, the conversion from other axle loads (L) is made by calculating the equivalent number (n) of the standard 18000-Ib load applications from,

$$n = 2.4 \times 10^{-8} L^4 \tag{2.2}$$

where L is in kN.

Many other highway organizations and reseach individuals have used, more or less, similar techniques for estimating equivalent axle loads from mixed traffic distributions.

The applicability of the equivalence factors approach for estimating traffic input for design was verified by many investigators like Deacon (48). Moreover, this technique was proved to satisfactorily estimate the traffic effects on pavements as depicted by the observed performance of many in-service pavements.

2.3.1.2 Canadian Approaches to Traffic Input Estimation

Most Canadian agencies have well established procedures for estimating traffic volumes and obtaining vehicle classification counts, and for measuring axle loads at either fixed or portable weigh-scale sites. To get better information, some agencies have even conducted "weighing-in-motion " measurements for axle loads including the " Viatec Axle Weight Analyzer " in Ontario, and the Saskatchewan and New Brunswick methods (49).

All Canadian highway departments set legal limits on vehicle weights and dimensions. A major study that aimed at decreasing the degree of non-uniformity in vehicles weights and dimensions among the different provinces, and investigating the issues related to the impact of heavy vehicles on pavement structures, was launched in 1984 (50). This study used the concept of the Load Equivalency Factor mentioned earlier for evaluating the relative damaging effects of the various axle load configurations usually encountered on Canadian highways. The results of this study showed good correlations with those developed for the AASHO Road Test (43).

Another investigation that aimed at establishing correlations between ESAL and other traffic information, was the one conducted by Shields (51), and described in detail in a technical report published in 1974 by the Highway & River Engineering Division of the Alberta Research Council. In this study, an attempt was made to relate the total daily single axle loads to the annual average daily traffic (AADT) and the number of heavy trucks observed. Such relations were set to describe the pattern of traffic loads usually encountered in heavily loaded lanes within Alberta highways.

A further insight look to the Alberta Transportation Highway Department procedure for estimating traffic input in terms of equivalent axle loads, will further be pursued in chapter four.

2.3.2 Materials Characterization

An important part for the development of a mechanistic design procedure for flexible pavements, is the adequate characterization of the constituent materials in the context of the part they play in the pavement structure.

This characterization usually takes the form of determining the elastic constants, and failure criteria that need be used as design inputs to compute critical stresses and strains. The acceptability of these mechanical responses may then be assessed in terms of the anticipated life of the pavement structure.

For all paving materials, namely, asphalt bound materials, cement-bound materials, untreated granular materials, and subgrade cohesive soils, the characterization is usually obtained by estimating the elastic modulus, E, and the Poisson's ratio, ν, for each of the constituent materials in question. According to the theory of elasticity, determination of these constants is sufficient to fully describe the materials' response to load.

2.3.2.1 **Dynamic Properties of Pavement Materials**

The evaluation of the aforementioned constants i.e. E and ν is not an easy task as it first appears. This is primarily due to the fact that those factors are significantly dependent on the stress state that exists within each layer of the pavement system. Therefore, the problem of selecting an appropriate individual value to represent each of these constants for any layer is extemely difficult. However, several investigators have suggested many ways through which such determinations can be achieved to represent the response of each pavement layer to traffic loads with reasonable accuracy.

The approaches currently available for obtaining representative moduli values for the different materials within the pavement system, can be listed as follows:

1. Laboratory testing: This is usually done through repeated triaxial or other dynamic-type testings. In these tests, the resilient modulus (for untreated granular and cohesive materials) or asphalt dynamic stiffness (for asphalt-bound

materials) is obtained from the ratio of the maximum axial repetitive stress to the recoverable axial strain. This can be described mathematically as,

$$M = \sigma_d/\varepsilon_r \tag{2.3}$$

where, M = resilient modulus,

σ_d = max. repetitive axial stress, and

ε_r = recoverable axial strain.

Using a dynamic rather than a static test like the one just described, is essential to realistically simulate the actual stress conditions that expected to be found in the field. This method is the most preferred for determining the moduli values for pavement materials since it closely simulates actual pavement conditions. A disadvantage of this method, as compared to other procedures, is the high cost of the apparatus to be used in such tests.

2. <u>In-situ modulus determination</u>: This is usually accomplished by using either short wave propagation techniques or surface deflection measurement methods. The former technique basically consists of generating steady-state stress waves into a pavement structure, and then a relationship between the modulus and the wave velocity is used for obtaining the modulus. On the other hand, the surface deflection method involves measuring the deflection under a prescribed set of loading conditions which will later be used in an iterative procedure for determining the

modulus.

Both techniques provide estimates of the in-situ moduli values, however, moduli determined by the short wave propagation techniques must be adjusted before they can be used as input in mechanistic design procedures. This is primarily due to the low stress levels involved in the test method which yield unrealistical high moduli values (52).

3. Empirical correlations: Certain empirical relations have been developed by several investigators in an effort to relate the resilient modulus of untreated granular and cohesive materials to more easily measured parameters like the CBR and R-value of such materials. Other correlations, like the one described by Heukelom and Klomp (53), attempted to relate the asphalt dynamic stiffness to the basic mix parameters.

On the other hand, a fair amount of knowledge of the ranges within which Poisson's ratios of the different paving materials fall, is also available.

A brief discussion of the characterization of the individual paving material will now follow.

2.3.2.2 Asphalt Bound Materials

The resilient characteristics of asphalt bound materials were found to be highly dependent on the mix temperature and rate of loading.

Extensive investigations considering both influences of temperature and loading times on asphalt stiffnesses, have been conducted by many reseachers, and procedures to permit measurement or estimation of elastic behaviour for particular times and rate of loadings were established.

Van der Poel (54) was perhaps the first to provide a simple basis for the determination of such characteristics. From extensive tests on a series of materials, he showed that the stiffness of asphaltic concrete is a function of the volume of concentration of the aggregate, as well as the stiffness of the bitumen itself. He proposed a nomographic procedure whereby the asphalt concrete stiffness can be determined from a knowledge of the penetration and softening point of the asphalt as it exists in the mixture, and the volume concentration of the aggregate. Later, Heukelom and Klomp (53) revised the Van der Poel procedure in details and suggested that the asphalt mix stiffness can better be determined from the following expression,

$$(S_{mix}/S_{bit}) = [1 + (2.5/n)\ x(C_v/1-C_v)]^n \qquad (2.4)$$

where, S_{mix} = mixture stiffness,

S_{bit} = bitumen stiffness,

C_v = volume of agg./(volume of agg.+ volume of bit.), and

n = $0.83 \log (4x10^5)/S_{bit}$.

A correction in the volume concentration of aggregate to be used for mixes with void contents greater than 3 percent, takes the form,

$$C_{mv} = (C_v/1+\Delta H) \qquad (2.5)$$

where, C_{mv} = modified volume concentration of aggregate,

C_v = original volume concentration of aggregate,

and

ΔH = difference between air void content and 3 percent, expressed as decimal.

Other investigators have suggested more direct procedures for measuring the influence of rate of loading on stiffness of specific materials through tension, compression, and flexural testing. Monismith et al (55,56), for example, perfomed creep and relaxation tests and have demonstrated that asphaltic concrete can be considered as linear viscoelastic material whose stiffness can be determined from creep compliance, or relaxation modulus as a function of time and temperature.

Papazian (57) and Pagen (58) have used a complex modulus to define the time of loading and temperature dependency of asphaltic concrete.

Pell and Cooper (59) have also described a stiffness machine which can be ~~used~~ for measuring the flexural stiffness of asphalt materials.

The Asphalt Institute (6) has developed an equation wherein the effects of time of loading, temperature, and mix characteristics were incorporated to determine the dynamic modulus of asphalt concrete materials.

Other studies showed that the asphalt stiffness is also stress-dependent, but such effect is still relatively small compared to the effects of temperature, time of loading, and mix characteristics.

As has been desribed, a vast amount of knowledge is currently available for estimating asphalt concrete stiffnesses from either direct measurements, or empirical correlations with other factors.

2.3.2.3 **Cement-treated Materials**

The elastic modulus of cementitious base layers can readily be measured in the laboratory using, for example, dynamic bending tests on bars cut from the pavement. However, there is some doubt whether this modulus will be truly representative of the modulus of the material in situ, which will most likely exhibit shrinkage cracks. Moreover, results from repeated triaxial tests such as those reported by Mitchell and Chen (60) and Wang (61), have shown that moduli values for soil-cement decrease with increasing confining pressure. Also another study (62) indicated that the moduli values obtained by triaxial testing are 10 times greater than what might be obtained from flexural testing. Therefore, and due to all the above considerations, careful interpretation of the laboratory results concerning cementitious materials' moduli values, is essential before such values can be incorporated in a design procedure.

The Shell method for pavement design (2) used typical moduli values($5x10^6-10^7$ kN/m^2), to represent sand-cement soils.

Poisson's ratio, on the other hand, did not receive as much attention as the modulus value. However, values between 0.1 and 0.2 were found to be adequate for such materials.

2.3.2.4 Untreated Granular Materials

The behaviour of untreated granular materials usually used as bases and sub-bases within the pavement structures, was found to be significantly affected by the degree of confinement i.e. function of the confining pressure in the soil (62,63,64).

The tests usually used for characterizing these materials, differ in the method of applying the confining pressure. Basically, such tests can be divided into the following categories (64):

1. Triaxial-compression repeated-load tests with either constant or varying cell pressures.
2. Repeated-load tests performed in the Hveem Resiliometer.
3. Tests wherein the specimens are compacted and tested in hollow rigid cylinders in the absence of any confining pressure.

Several expressions have been proposed by many investigators to describe the non-linear behaviour of granular materials. These, in general, can be categorized as follows:

1. Resilient modulus as a function of the sum of the principal stresses (64),

$$M_r = k\,\theta^n \tag{2.6}$$

where, M_r = resilient modulus of elasticity,
θ = sum of principal stresses (= $\sigma_1 + 2\sigma_3$ in conventional triaxial cell), and
k & n = experimentally determined constants.

2. Resilient modulus as a function of the confining pressure (64),

$$M_r = k_1(\sigma_3)^{n_1} \tag{2.7}$$

where, σ_3 = confining pressure, and
k_1 & n_1 = experimentally determined constants.

3. Resilient modulus as a function of the mean normal stress, p, and deviator stress, σ_d, (65),

$$M_r = f(p,\ \sigma_d) \tag{2.8}$$

where, $p = (\sigma_1 + 2\sigma_3)/3$, and
$\sigma_d = (\sigma_1 - \sigma_3)$.

A description of the factors influencing the granular materials' resilient moduli, as reported by Seed et al. (64), can be listed as follows:

1. Duration of stress application and rate of deformation: The results of triaxial repeated loading tests by Seed and Chen (66) have shown that this factor is of relatively small influence on the magnitude of the resilient modulus.
2. Frequency of load application: Coffman's results (65) indicated that the higher the frequency, the higher will be the modulus. Increases in the moduli values ranged from 50 to 100 percent resulted, depending on sample water content and dry density.
3. Type of aggregate and percentage of material passing the 80 μm sieve: Studies by Haynes and Yoder (68) have indicated that the influence of such factors is of minor importance (approximately changing the modulus value by about 20 percent).
4. Degree of saturation: Same investigations by Haynes and Yoder (68) also indicated that, increasing the gravel degree of saturation from 70 percent to 100 percent , will decrease the resilient modulus by one-half its original value.
5. Void ratio: The effect of different initial void ratios of the granular materials, has also been investigated. However, such effect seems to diminish completely when the soil sample had first

conditioned by applying several hundred repetitions of the test load before actually starting the test.

6. Confining pressure: All tests in which the effect of confining pressure was investigated, showed the large influence that this factor had on the resilient modulus. Studies by the Texas Transport Institute (69) indicated that the modulus value could increase by as much as 500 percent upon increasing the confining pressure from 3 to 30 psi.

7. Stress level: Trollope et al (70) concluded that the resilient modulus is independent of the axial stress level so long as that stress does not cause excessive plastic deformation.

Despite the large spectrum of the above factors that tend to influence the modulus value of granular materials, a reasonable estimate of an effective value can readily be obtained if careful simulation of the actual field conditions, and the high degree of dependency of the modulus on confining pressure, were observed.

Following the non-linear dependency of the resilient modulus of granual materials on the stress state, Kasianchuk (28), extended the Chevron 5-layer computer program to include non-linear, iterative elastic layered solutions. In his approach, the granular base and sub-base layers has further been subdivided into several fictitious layers, and a modulus value that is dependent on the average stress state that exist in the vicinity beneath the wheel loadings, is assigned to each of these sub-layers. An iterative solution is then performed by computer program until the difference between the assigned and actual moduli values is within 5 percent, under the prescribed loading conditions.

An alternative approach followed by other design procedures (2,6) that caters for the granular materials dependency on the stress state, considers the resilient modulus of such materials as being a function of some of the parameters of the other pavement materials. An example of such an approach can be found in the Shell method of design (2) wherein, the granular material modulus is determined by an expression of the form:

$$E_r = k\ E_{sub} \tag{2.9}$$

where, E_r = resilient modulus of granular layer,

E_{sub} = resilient modulus of subgrade layer, and

k = $0.2(h_2)^{.45}$ with h_2 as the granular layer thickness in mm, and k is applicable in the range $2 < k < 4$.

Another design procedure that uses a similar approach in describing the granular layer modulus, is the one that the Asphalt Institute suggest (6). This approach will be described in detail in chapter three.

As far as the Poisson's ratio of the granular materials is concerned, investigations have shown that such ratio is apparently related to the principal stress ratio (σ_1/σ_3), (27).

2.3.2.5 Cohesive Subgrade Soils

All studies of the non-linear response of clays have indicated that the resilient modulus of such materials in the direction of an applied stress, decreases as that stress is increased, and is little affected by the transverse stresses (64,71).

Several investigators have shown that the resilient modulus of cohesive materials can be described as a function of the deviator stress, in the form:

$$M_r = A\ (\sigma_d)^B \tag{2.10}$$

where, σ_d = deviator stress $(=\sigma_1 - \sigma_3)$.

M_r = resilient modulus of cohesive soil, and

A & B = experimentally determined constants.

Further, extensive studies reported by Seed et al (64), on a wide range of soil types and conditions, revealed the following basic factors that influence the resilient modulus of cohesive materials:

1. <u>Number of stress applications</u>: Resilient moduli determined under relatively small number of stress applications seem to give a misleading picture of the actual resilience characteristics of the clayey soils due to the stress conditioning effect. Such an effect can be eliminated by estimating the

modulus value after a large number of stress applications.

2. Aging: As the time between compaction and modulus testing increases, resilient modulus was found to increase, specially at lower number of stress applications. After a large number of stress applications, such an effect seems to diminish rapidly and essentially same results were obtained for specimens tested immediately after compaction as for specimens tested after a delay.

3. Stress intensity: Changing the stress level, was found to significantly affect the modulus value. Increasing stress intensity, was found to considerably decrease the modulus value.

4. Method of compaction: compaction methods that produce dispersed structures in soils, tend to produce lower moduli values. Also increasing degree of saturation at time of compaction, tend to decrease the resilient modulus.

5. Changes in water content and density after compaction: On one hand, increasing the water content of cohesive soils after compaction, seems to have adverse effects on the modulus value, whereas increased density tend to produce beneficial influences on the other hand.

Most of the adverse influences of the above mentioned factors on the resilient moduli of cohesive materials, can be catered for by careful preparation and conditioning of the specimens to be used in the tests. Moreover, simulating the actual conditions to be encountered in the field is essential for obtaining realistic representative moduli values for these materials.

In an attempt to avoid the cost and technical complexities usually involved in dynamic-type testing, some investigators developed procedures whereby the resilient modulus of subgrade soils could be obtained from empirical correlations with more easily measured parameters like the CBR ,and R-value of these soils. An example of such relations can readily be found in the Shell method of design (2). In this method, the resilient modulus of the subgrade is related to the CBR value by the expression:

$$M_r \text{ (psi)} = 1500 \text{ CBR} \tag{2.11}$$

It is important, however, to note that such coefficient can vary from 0.5 to 2.0 times that amount.

Another example relating the resilient modulus of cohesive subgrade materials to the R-value were originally derived from data collected during the San Diego County Experimental Base Project (72). This relationship took the form:

$$M_r \text{ (psi)} = 1155 + 555 \text{ R} \tag{2.12}$$

In applying any of the above described empirical correlations, extreme caution should be exercised since the scatter in the data from which these were origanally developed, is great.

As far as the part dealing with the estimation of the Poisson's ratio for such materials is concerned, representative values in the range 0.35 to 0.5 were reported by many investigators.

To conclude this section, extensive studies by several investigators on the materials' responses to traffic loadings, make it possible to incorporate the appropriate material inputs required for pavement design purposes.

2.3.3 Environmental Factors

Environmental variables can include a wide variety of inputs such as the geological soil conditions, oxygen and air that surround the pavement structure, and moisture and temperature conditions that exist within and around the pavement. Among those, moisture and temperature conditions represent the most significant factors that influence pavement response to loads.

The effects of both moisture and temperature variations on pavements, can be viewed through their influences on the properties of the constituent materials. Such environmental variations tend to change the strength of the pavement materials so that its resistance to traffic induced stresses is altered. Moreover, there is much evidence that pavements are deteriorated by environmental factors in the absence of traffic variables. This can be conceived through the fact that temperature and moisture variations, among other environmental variables, can produce internal stresses in the pavement materials which may become excessive and ultimately lead to structural failure.

Therefore, consideration of the effects of such factors in influencing the load-carrying capacity of asphalt pavements, is an important aspect in any mechanistic design procedure.

2.3.3.1 Moisture Variation Effects

The presence of moisture and its cyclic variations within the pavement structures can produce one or more of the following effects:

1. Combination of water and sunlight can lead to the oxidation of the asphalt concrete layer, and therefore change the layer resistance to traffic and/or other environmental factors.
2. Under sub-zero surface temperature conditions, freezing and consequent thawing occur. Such conditions will tend to reduce the subgrade soil modulus value during the spring time, and therefore enhance the damage induced by traffic during this period.
3. Instability may occur under excessive pore-water pressures that develop in the soil due to increased

moisture, and thus lead to base and slope stability failures.

4. Undesirable volume changes may occur in certain soils, e.g. expansive clays, under unstable moisture conditions.

5. Moisture condition variations may also induce internal stresses within the pavement structure which may become excessive and eventually produce structural failure.

For the purposes of incorporating moisture conditions as input variables in any mechanistic design procedure, both effects of freezing and thawing, and stresses induced by soil-suction potentials present in the material, had to be investigated. Studies by Croney et al (73) and Marek and Dempsey (74) emphasized the importance of considering such effects in pavement design. The combined effect of both moisture and sub-zero temperature conditions on pavement materials, will be to increase their ~~ability~~ to withstand distress accumulation while in the winter time, and to reduce such ability while in the spring time. To take such effects into consideration, both Shell (2) and the Asphalt Institute (6) used different subgrade moduli values to represent a typical year period in their design methods.

Another alternative in incorporating moisture variation effects in pavement design, is to evaluate the soil-suction potential under the pavement. A great deal of work has been done with that concept, but most of it is related to the prediction of swell or shrinkage and not directly to the prediction of induced stresses. Measuring devices have been developed to accurately measure the soil-suction potential of a soil, but no attempt appears to be made to relate that to pavement distress.

Because moisture cycling is seasonal and not a daily change, moisture variations do not impose the large number of repeated stresses that temperature variations do. Therefore, the major concern with moisture variations is to cater for its combined influence with sub-zero temperature conditions in changing the materials properties through freezing and thawing cycles.

2.3.4.2 Temperature Variation Effects

Of all environmental factors, temperature is the easiest to estimate and use in a mechanistic design procedure. Therefore, many design procedures have incorporated the environmental influences on pavement thickness design requirements in terms of adjustment factors based on temperature distribution within the pavement structure.

Effects of temperature conditions on pavement structures are twofold. The first, manifest itself in terms of the temperature induced stresses due to seasonally as well as daily temperature variations. The second, is the implicit temperature influence in changing materials properties, and thus causing variations in the pavement materials responses to traffic loadings. These effects coupled by the interaction effect of the sub-zero surface temperature conditions and moisture present within the pavement structure, lead to further reduction in the pavement system ability to withstand traffic loadings while in the spring time due to reduced subgrade strength under thawing conditions.

The stresses usually induced within the pavement structure as a result of varying temperature conditions, are either tensile (or compressive) stresses due to seasonal variations, or bending stresses due to diurnal temperature variations. The effects of these stresses in producing temperature associated cracks were studied by many investigators. Worth mentioning among these, is the work reported by Monismith and Secor (56), and Haas et al (75) on the phenomenon of low temperature cracking.

Due to all mentioned temperature influences on pavement thickness design requirements, several approaches have been taken to the problem of predicting appropriate temperature inputs to be included in pavement design methodologies. Hudson (41) summarized the approaches adopted by the different investigators as follows:

1. The application of regional or environmental factors to adjust the design thickness.
2. Developing design procedures that will relate to a general climatic area where an annual cycle of temperature, freezing, and so on will be about the

same from year to year(this approach is like the one developed by Haas et al (75),in describing low temperature cracking phenomenon).

3. Developing temperature prediction models whereby the temperature profile and history of a particular pavement are predicted based on available weather data.

Shahin and McCullough (76), have developed a model for predicting daily temperature cycling during an average year. In this model, heat transfer concepts were used to develop a set of equations whereby the temperature profile within the pavement system can be obtained from the daily mean air temperature, and solar radiation inputs.

Monismith and Secor (56), developed a technique for predicting pavement temperature distrbutions based on the heat-conduction equation. Other models that used meterological data and are based on the heat-transfer theory are also developed and described by Christison (77), and Robinson (78). These models utilized a finite difference technique, available meterological inputs, and the thermal properties of the pavement materials to predict the temperature distributions within the pavement system.

To evaluate the applicability of any of the above mentioned, or otherwise models, or to develop empirical temperature models, requires a knowledge of the real temperature gradient present in a pavement. From extensive temperature measurements reported by Kallas in 1966 (79), and other similar work, like the one conducted during the AASHO Road Test (48), a considerable amount of information is currently available for checking the validity of any of the proposed temperature prediction models.

2.4 Design Criteria

As mentioned earlier, the second stage in material characterization will be to set up the appropriate designing criteria that define the stages of life of the pavement sructure at which major maintenance and/or rehabilitation activities are required for the pavement system to continue to perform adequately under the prevailing service conditions.

Many design criteria have been proposed by the different design methodologies. Among these, is the criterion that limits low-temperature cracking in areas that are subjected to sub-zero temperature conditions. Other criteria like fatigue cracking, permanent deformation, and pavement surface disintegration can be found in other design procedures.

For the objectives of this study, only the fatigue cracking, and permanent deformation criteria used in the Asphalt Institute thickness design method (6), will be considered.

2.4.1 Fatique Subsystem

Many studies have revealed the unlikelihood of properly designed pavement to fail under a single axle load application. Rather, fatigue cracking of stabilized materials frequently occurs as a result of repeated bending due to a large number of load repetitions applied to the pavement.

2.4.1.1 Definition of Terms and Types of Fatigue Testing

Fatigue has been defined by Monismith and Deacon (80), as" the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material". However, in practice and in laboratory testing, fatigue failure is loosely defined as the point at which the material or specimen ceases to perform in a satisfactory manner under the prescribed loading conditions. The failure or end point in a fatigue test have been defined by investigators in many different ways. It may be the point corresponding to complete fracture of the test specimen, the point at which a crack is first observed, or the point at which the stiffness or other property of the specimen has been reduced by a specific amount from its original value. The choice is often arbitrary depending on the method of testing used.

Fatigue life, N_s or N_f, is defined as the accumulated number of load applications necessary to cause failure, and it is generally a function of the manner in which failure is defined.

The fatigue behaviour of an asphalt concrete specimen subjected to repeated loading, primarily depends on the load, environmental, and specimen variables (80). Loading condition refers to the particular set of values that load and environmental variables assume for a particular load application. Simple loading occurs when loading conditions remain unchanged throughout the fatigue test, whereas compound loading refers to the cases under which loading conditions vary throughout the fatigue test.

Fatigue tests are usually conducted under either controlled stress mode (loading in the form of an applied alternating stress of constant amplitude), or controlled strain mode (loading in the form of an applied alternating strain or deflection of constant amplitude). The mode of loading was found to significantly affect the laboratory determined fatigue life. To compensate for such an effect, Monismith and Deacon (80), have introduced the concept of the mode factor defined as;

$$MF = (|A| - |B|)/(|A| + |B|) \tag{2.13}$$

where, $|A|$ and $|B|$ are the percentage changes in stress and strain, respectively, for an arbitrary but fixed percentage reduction in stiffness.

Following the concept of the mode factor, fatigue tests conducted under controlled stress mode i.e. $MF = -1$, were found to closely simulate the conditions for thick asphalt layers (>6 in.), whereas tests under controlled strain mode i.e. MF = 1, were found to be suitable for thin asphalt layers (<2 in.).

For intermediate thicknesses, some form of testing between these two extremes would strictly be appropriate. However, and from an engineering view point, controlled stress tests would seem more sensible for these thicknesses, since they gave a more conservative estimate of the design life.

A variety of fatigue testing methods that have been performed by several investigators, were critically reviewed by Pell (81). To date, most of the results have been obtained from bending or flexure tests on rectangular specimens tested as simply supported beams, trapezoidal-shaped specimens tested as cantilevers, or specimens having a circular cross-section with varying diameter tested as rotating cantilevers. Cylindrical and rectangular specimens are increasingly being used under direct uniaxial(tension-compression) loading, and under triaxial states of stress. Plate and torsional specimens have also been used to obtain biaxial stress conditions, and the indirect tensile test has also been adapted for repetitive loading.

2.4.1.2 Presentation of Laboratory Fatigue Test Results

It was found that, there is always a considerable scatter in the results of any laboratory fatigue testing of identical specimens. This is usually explained as due to the inherent inhomogeneity of the material tested as well as the unavoidable variations that resulted from specimen preparations. This means that fatigue life must be considered in a statistical manner, and strictly as only a distribution of individual values. It is thus usually assumed that fatigue behaviour of laboratory tested asphaltic concrete specimens, can be expressed as a logarithmic normal ditribution of fatigue lives at a particular loading condition. An investigation conducted by Pell and Taylor (82) justified the use of such relation, and an equation of the following form is proposed for controlled stress tests:

$$N_f = k \left(\frac{1}{\sigma}\right)^a \tag{2.14}$$

N_f = mean service life obtained at particular loading conditions,

σ = amplitude of applied tensile stress, and

k and a = coefficients that could be determined by linear regression analysis.

A similar relation but one in terms of applied tensile strain is also postulated for controlled strain tests.

A slightly different form for laboratory fatigue life estimation, has been suggested by other investigators. An example of such form, is the one proposed by Finn et al (83,84) wherein the fatigue life is related to both the amplitude of the repeated tensile strain, and the asphalt concrete stiffness, by an equation of the form:

$$N_f = (a_1 \cdot \varepsilon^{-b}) . (E^{-c}) \tag{2.15}$$

where, ε = repeated applied tensile strain to failure,

E = asphalt mixture stiffness, and

a,b,c = experimentally determind coefficients.

There is no agreement between the many investigators as to the use of a universal equation to completely describe the fatigue behaviour of asphalt concrete mixes. Rather, many relations, of either of the forms mentioned above, were postulated to describe the performance of these mixes. A possible reason that prevented the development of such a general equation, lies mainly in the difficulties encountered in interpolating fatigue results that were obtained by different testing procedures on different mixes.

2.4.1.3 Factors Affecting Fatigue Behaviour

Due to the considerable experimental efforts involved in developing a fatigue equation of the form described above, attempts were made by several investigators to relate the fatigue behaviour of asphalt concrete to easily measured mix parameters like, for example, the volume of the asphalt binder, volume of air voids in the mix, etc. The major factor influencing the fatigue behaviour of asphalt concrete mixes, is the asphalt dynamic modulus usually referred to as the asphalt stiffness. Consequently, all mix factors that influence the stiffness will in turn influence the fatigue life estimation.

From an extensive experimental study conducted by Pell and Cooper (59) and another work by Kirk (85), the following findings concerning the effects of both mix parameters and test variables on fatigue performance of asphalt concrete, were obtained:

1. Binder content was found to be a primary factor influencing fatigue performance on the basis of applied dynamic strain, longer lives being obtained for a given strain when the relative volume of binder is increased. However, this relation was found to be valid up to a certain optimum binder content volume after which additional increase in binder content have no influence in further increasing the fatigue life.
2. The effect of the air void volume content was found to be dependent to some extent on the volume of binder content. Therefore, combining both effects in terms of one parameter was found to be more adequately representative of both the individual and interaction effects of these factors. This parameter, in most cases, was chosen to be the ratio of the binder volume content to the sum of the binder and air voids volume contents.
3. Binder type was also concluded to be of significant influence on fatigue performance. Longer lives were obtained when a binder having a higher ring

and ball softening point temperature is used.

4. Aggregate type and grading was found to only influence the fatigue performance on the basis of applied dynamic strain insofar as they influence the relative binder content of the mix.

5. The effect of adding filler to a mix was concluded to increase the dynamic stiffness of the mix, but no appreciable improvement in fatigue behaviour was obtained if the relative binder content remained constant.

6. The results obtained from triaxial fatigue testing by Pell and Cooper (59) indicated that, the fatigue performance on the basis of applied dynamic strain of hot rolled asphalt mixes, is independent of the effects of both confining stress and temperature. However, this conclusion was found to apply only over a limited temperature range for certain types of testing.

In view of the above described mix and test variables effects on fatigue behaviour of asphalt concrete mixtures, several attempts were made to develop empirical correlations relating such factors to the prediction of laboratory fatigue performance. An example of such correlations, is the one developed by Pell and Cooper (59), that take the form:

$$\log N_{f(\varepsilon=10^{-4})} = 4.13 \log V_B + 6.95 \log T_{R\&B} - 11.13 \qquad (2.16)$$

where, N_f = the fatigue life expressed as the number of tensile strain applications of amplitude 10^{-4} required to produce failure,

V_B = volume of binder content percent, and

$T_{R\&B}$ = ring and ball softening point temperature, °C.

2.4.1.4 Fatigue Life Estimation of Pavement Structures

Estimating the damage accumulated within the actual pavement structure due to fatiguing of asphalt stabilized materials, has been the prime goal of many investigators. Deacon (48), Terrel (86), and Finn et al (83), have utilized the well known Minor's hypothesis of linear summation of cycles ratio to estimate the damage induced in the pavement by the fatigue distress subsystem. This hypothesis assumes that the damage induced in the pavement due to fatigue under different sets of loading and environmental conditions, can be summed up cumulatively until a damage of 100 percent (or 1.0) is obtained at which the pavement is considered to have failed.

The procedure followed by those investigators, using Minor's rule, in estimating the fatigue design life, can best be illustrated as follows: let d_{ij} be the damage induced in the pavement by one application of the ith load condition while the pavement is in the jth physical (or environmental) state, then let N_{ij} denotes the number of repetitions of that load under that physical state which will cause failure, then

$$d_{ij} = (1/N_{ij}) \tag{2.17}$$

for an efficient pavement i.e. one in which the total cumulative damage, D, = 1.0 at the end of the design life. Now in order to obtain the total cumulative damage, this unit damage is then multiplied by the actual number of applications, n_{ij}, of the ith load condition at the jth physical state i.e.

$$D = \Sigma\ \Sigma\ d_{ij}\ n_{ij} = \Sigma\ \Sigma\ (n_{ij}/N_{ij}) \tag{2.18}$$

Estimating the values of both n_{ij} and N_{ij} for any pavement structure is thus an essential step toward predicting the fatigue design life of that facility.

Determining n_{ij} is not a difficult problem, and can usually be accomplished by analyzing the existing and projected traffic data to yield estimates of the traffic anticipated to use the facility during its design life.

On the other hand, estimating N_{ij} that is solely based on laboratory fatigue test results does not appear to give a realistic representation of the actual pavement performance under the prescribed distress mode. The reason behind this lies in the fact that, the initial occurrence of cracking may not be so crucial to the overall performance as some acceptable level generally indicative of a pending period of accumulated deterioration. Based on this concept, a technique was suggested by some investigators whereby the failure criteria governing the fatigue performance of bituminous pavements, can be obtained through the application of appropriate shift factors to the laboratory developed fatigue relations.

Following the shift factor approach, Finn and associates (83,84) developed the following relations, from laboratory fatigue data provided by Monismith et al (42), to represent the fatigue subsystem:

$$\log N_f(<10\%\ \text{cracking}) = 15.947 - 3.291 \log(\varepsilon_t/10^{-6}) - 0.854 \log(E/10^3), \tag{2.19}$$

and

$$\log N_f(>45\%\ \text{cracking}) = 16.086 - 3.291 \log(\varepsilon_t/10^{-6}) - 0.854 \log(E/10^3), \tag{2.20}$$

where the percentage cracking refers to the amount of cracking in the wheel path area. Those relations were developed from information related to the AASHO Road Test. Some 19 sections, that exhibited fatigue cracking, were involved in obtaining the shift factor which was then used to modify the laboratory fatigue equations to reflect the actual in-situ conditions.

2.4.2 Permanent Deformation Subsystem

Permanent deformation in the wheel path is considered a major source that contribute to the deteriorating behaviour of asphaltic concrete pavements. The accumulated water in such depressions provide the opportunity for both hydroplaning and icing effects to considerably reduce the road safety. Therefore, inclusion of such a criterion is essential for the adequacy of any design procedure that aims at improved road serviceability and safety.

Among the many structural, economical, and safety considerations in developing predictive models for permanent deformation, are the following:

1. Introduction of new heavy vehicles with different than usual axle loads and tire pressures.
2. Usage of softer asphalt grades to prevent premature and low temperature cracking, and at the same time provide resistance against permanent deformation.
3. Elimination of hazardous hydroplaning and icing effects when water accumulates in such depressions.
4. Effective use of the pavement facility without the need of performing major maintenance activities at short time intervals.

The factors mentioned necessitate the development of appropriate permanent deformation designing criterion that will help reducing, if not eliminating completely, such diverse effects on pavement performance.

2.4.2.1 Definition and Mechanisms of Permanent Deformation

Defining the permanent deformation precisely is the first step toward formulating the predictive models that govern the phenomenon.

Morris (87) defined the permanent deformation as the longitudinal depressions or "ruts" that develop in the wheel path due to each application of a wheel load. Barksdale (88) similarly defined the phenomenon as one of progressive accumulation of the plastic strain in each layer of the pavement structure that occurs under each load repetition. Haas et al (89), also defined the permanent deformation as "the principal mode of distress that manifest itself as permanent longitudinal deformations along the wheel path as a result of wheel load applications".

The magnitude and rate at which permanent deformation occurs depend on the type of materials used in the various layers, degree of compaction, loading, and environmental conditions.

Two major mechanisms contribute to permanent deformation. Those are: first, densification that result from inadequate compaction of pavement materials; and second, plastic shear deformation that reflect the pavement response under different traffic and environmental conditions. These two mechanisms contribute in varying degrees to rutting in all the pavement layers and the subgrade.

2.4.2.2 **Factors Affecting Permanent Deformation**

Factors responsible of, and influencing, permanent deformation can be grouped into traffic-related and material-related. Many investigators like Monismith (90) and Haas et al (89), have summarized such factors as follows:

1. Traffic-related variables include the number and magnitude of load repetitions, the traffic composition, the lateral placement of axles, the vehicle speed, and the tire inflation pressures.
2. Material-related variables affecting permanent deformation include, the temperature related rheological properties of bitumenous layers, the nature of aggregates, the mix design, and the deformation characteristics of both unbound granular layer and the subgrade which may vary with the moisture content.

Traditionally, traffic-related factors were controlled through the use of load enforcement strategies which limit the axle loads and/or gross vehicle weights, of the traffic that use the facility, during the period where the subgrade resistance to traffic is reduced. On the other hand, material-related factors were controlled, in early design methods, by specifying a minimum thickness and/or minimum CBR values for the pavement layers. Also specifications of viscosity and Marshall index properties were used as means for controlling the asphalt layer rutting.

2.4.2.3 Permanent Deformation Models

The attempts made by the several investigators that studied the permanent deformation, followed two basic approaches in modelling the phenomenon. The first, considered limiting the amount of rutting while the second, tried predicting its magnitude as a function of the factors affecting its manifestation.

Early systematic observations of rutting were obtained at the AASHO Road Test (43). The analysis of these results indicated that most of the rutting observed was primarily due to plastic shear deformation which was in turn found to be season dependent.

Shell researchers in 1962, ~~were the~~ first to present a mechanistically-based design procedure wherein considerations for eliminating rutting were incorporated. Following that approach, Dormon and Metcalf (91) developed a rut limiting criterion in the form of relationships between the vertical compressive strain at the top of the subgrade layer and the number of traffic load applications. However, such an approach does not include provision for rutting formation as a result of densification and/or shear deformation that take place in the other layers above the subgrade.

During the late 1960's and the early 1970's, renewed interest in the problem of rutting arose, possibly as a result of the trend toward thicker asphalt designs and increased binder contents. Furthermore, rapidly increasing traffic loads called for the need for predictive, rather than limiting, models for rutting. Thus most of the rutting models that were developed during this period followed the so-called "layer-strain " approach. The basic idea behind this approach was first introduced by Romain (92). The method involves three major steps. First, the pavement layers are divided into sublayers with lesser thicknesses, and the stress state within each layer is determined assuming linear or non-linear elastic behaviour. Second, relationships of plastic strain in each sublayer as a function of the stress state, number of load applications, temperature, density, etc. are established. Finally, the contribution of the various sublayers to plastic strain are summed to yield the total vertical plastic deformation. In mathematical form, this can be expressed as:

$$d = \Sigma \; \varepsilon_i \cdot h_i \qquad (2.21)$$

where, d = total vertical plastic deformation,

ε_i = average plastic strain within layer i,

and h_i = thickness of layer i.

This concept gained popularity between the many investigators that tackled the rutting problem, and later several models were postulated based on that approach. Barksdale (93) used the approach to compare the behaviour of various base materials with respect to rutting. He utilized the plastic strain relationship proposed by Duncan et al

(94), to model the plastic behaviour of granular bases. This relation took the form:

$$\varepsilon_p = \frac{[\sigma_1 - \sigma_3)/(k\sigma_3{}^n)]}{[1-(\sigma_1-\sigma_3)(1-\sin\phi)R_f/(2c\ \cos\phi+2\sigma_3\sin\phi)]} \qquad (2.22)$$

where, ε_p = plastic strain,

σ_1 & σ_3 = major and minor principal stress,

k & n = constants interpreted as the intercept and slope of the logarithmic relationship between the modulus of elasticity and the confining pressure,

R_f = asymptotic value of deviatoric stress, and

c & ϕ = effective shear strength parameters.

Morris et al (95), proposed a "layer-strain" model wherein the plastic deformation characteristics of the asphaltic layer were related to the stress state, temperature, and number of load repetitions, using repeated triaxial testing and statistical analysis techniques. The adequacy of this model was tested by comparing predicted and measured rut depths of full-depth asphalt sections from the Brampton Road Test project.

Meyer et al (96) expanded the previously described rut prediction model developed by Morris et al, to include the contribution of the unbound granular base layers to permanent deformation.

Glaessen et al (2) proposed an improved Shell design method wherein the final thickness design satifying the limiting criteria for vertical compressive strain at the top of the subgrade, should be checked for permanent deformation in the bituminous layers.

Finn et al (83) also described a technique whereby a statistical procedure utilizing observed rutting performance of in-service pavement was used to develop a model that minimizes pavement rutting.

A major disadvantage of the above described models which are based on the elasticity theory, is that such models can not reproduce the condition of the ridge that usually forms adjacent to the rut due to the shear displacement of the material under the wheel path. This difficulty was accounted for by the efforts of many other investigators that developed predictive rutting models that were based on viscoelastic rather than elastic behaviour. Haas et al (89) conducted a comprehensive literature review wherein the efforts of many of those researchers were described.

Recent developments in formulating permanent deformation models in the early 1980's, with few exceptions, followed the two basic approaches described earlier; i.e. either indirectly limiting or directly predicting rutting.

The basic notion of indirectly limiting rutting by limiting the subgrade compressive strain, was further extended to include surface deflection criterion. Roberts et al. (97) used limiting both the subgrade compressive strain criterion and the surface deflection criterion as the basis for a deformability system.

Other simplified models were proposed wherein the plastic strain is related to the elastic strain and the number of load applications (98).

In 1982, renewed interest in developing rutting models that were based on the performance of in-service pavements, were further pursued. Lytton et al (99) have developed a rut depth prediction model that was based on field observations from some 400 Texas sites.

To conclude this part, a considerable body of information is now available for explaining, and minimizing the permanent deformation phenomenon. However, the suitability of such information to effectively improve the actual pavement performance, still remains the subject of further research activities.

2.5 Summary

Mechanistically-based design procedures constitute the most rational way for designing flexible pavements. However, the pitfalls involved in these procedures must clearly be recognized and minimized. Availability and reliability of input data, and a lack of knowledge concerning the sensitivity of the design factors are among the drawbacks of such design procedures.

Currently available mechanistic-empirical methodologies consist of two parts: an analytical part and an empirical one. The analytical part utilizes elastic or viscoelastic materials inputs to determine the pavement mechanical responses under the prevailing load and environmental conditions. The empirical part then correlates this information with the amount and extent of the structural distress induced to achieve a better understanding of the overall pavement performance.

Different models for describing the pavement system response to load are available. Among these, the approach based on the classical elastic theory constitutes the most cost-effective from the computational stand point. In addition, elastic models have been verified to give a reasonably good indication of the overall structural response of the pavement system to loading.

Cracking of the asphalt layer due to fatiguing and/or sub-zero temperature effects, and rutting induced in the different pavement layers, are recognized as appropriate criteria governing the design. Limiting the amount of stresses and/or strains induced in the pavement structure under the prevailing loading conditions, constitutes the basis for these criteria.

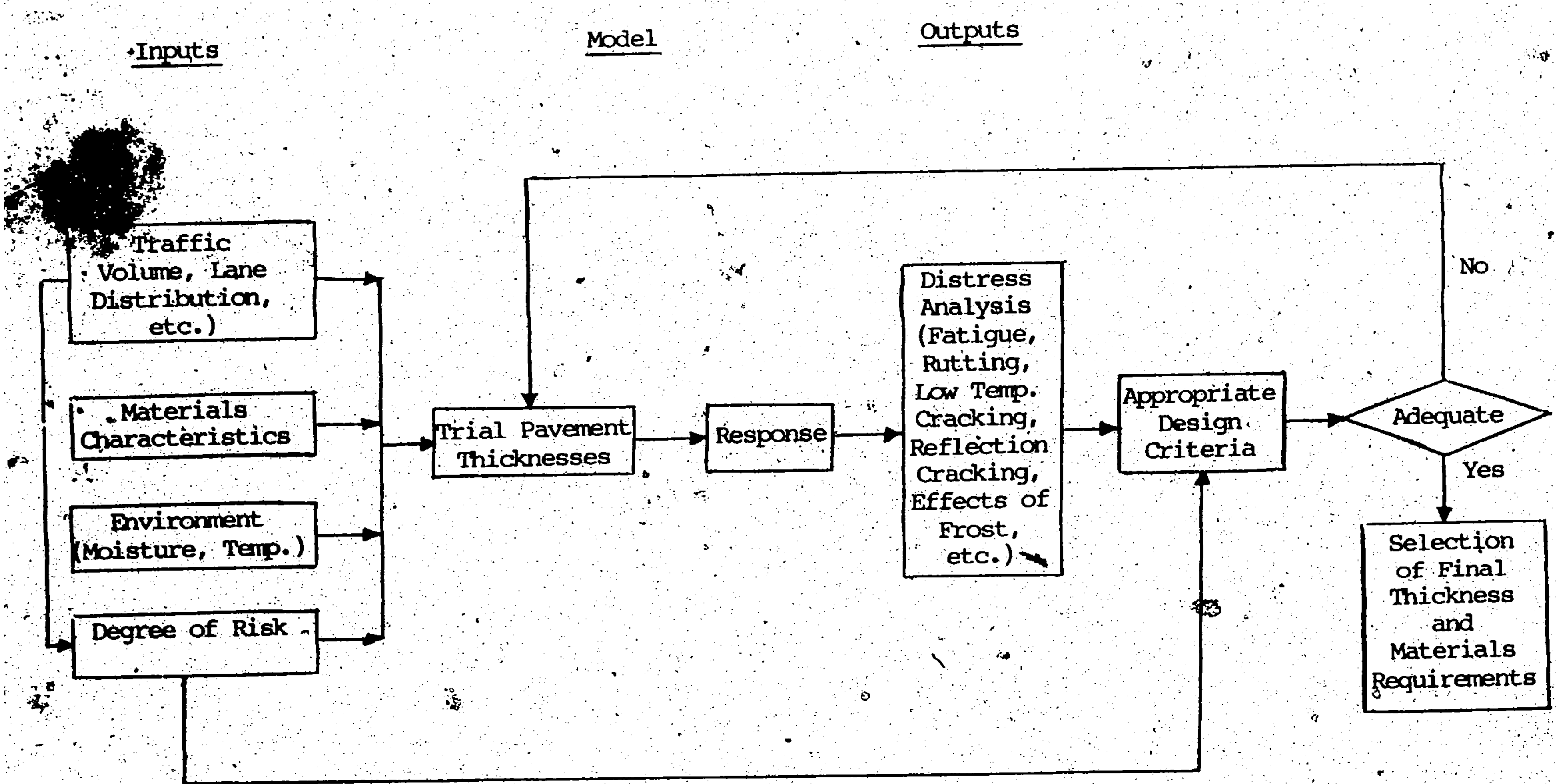


Fig. 2.1 Framework for pavement design

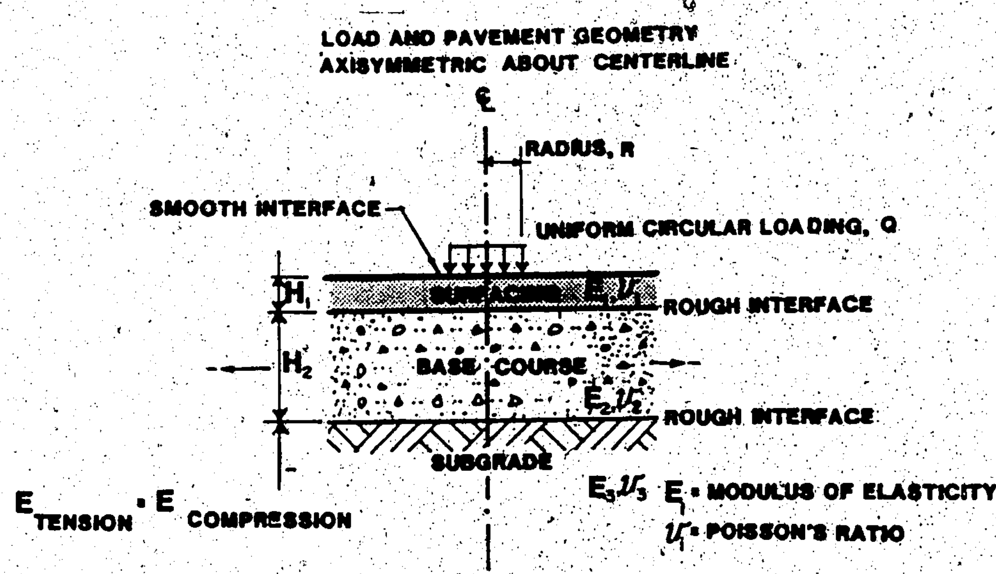


Fig. 2.2 Generalized multilayered elastic system

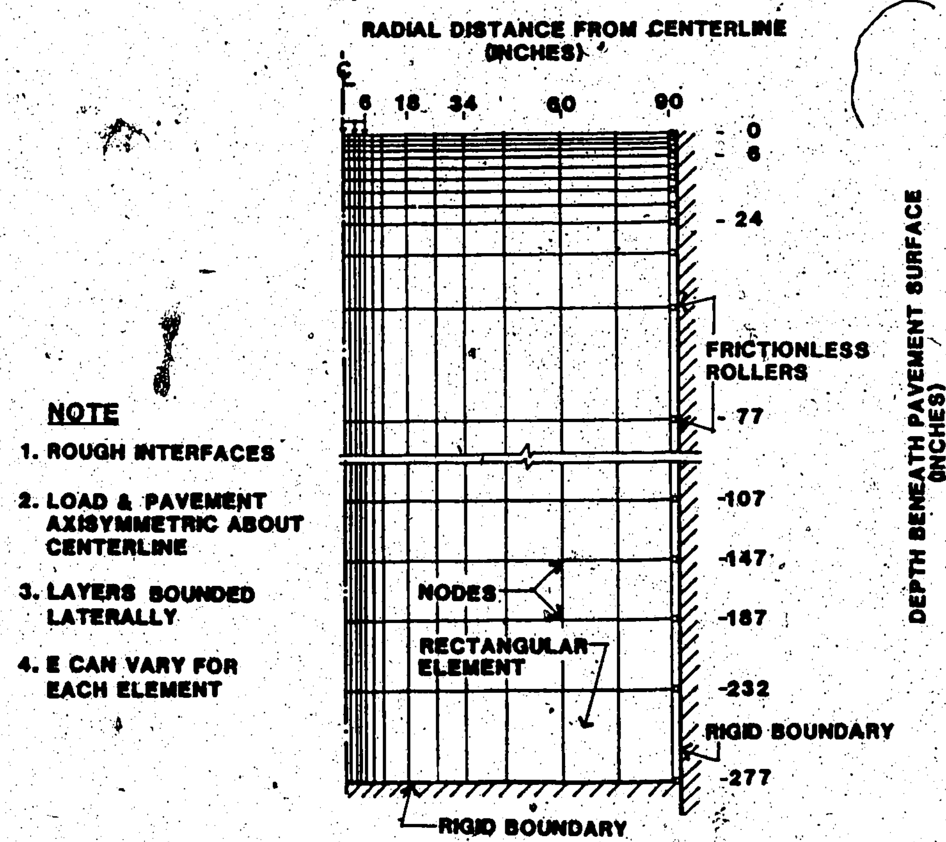


Fig. 2.3 Finite-element idealization of pavement system

References

1. AASHTO, AASHTO Guide for the Design of Pavement Structures, American Association of State Highway and Transportation Officials, 1986.

2. A.I.M. Claessen, J.M. Edwards, P. Sommer, and P. Uge, "Asphalt Pavement Design: the Shell Method", Proceedings, Vol. 1, Fourth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1977.

3. Shell, Shell Pavement Design Manual, Shell International Petroleum Company, Limited, London, 1978.

4. Shell, Addendum to the Shell Pavement Design Manual, Shell International Petroleum Company, Limited, London, 1985.

5. The Asphalt Institute, "Thickness Design - Asphalt Pavement Structures for Highways and Streets", The Asphalt Institute, College Park, Maryland, Asphalt Institute Manual Series No. 1, 9th Edition, 1981.

6. The Asphalt Institute, "Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1), Ninth Edition", The Asphalt Institute, College Park, Maryland, Asphalt Institute Research Report No. 82-2, Aug. 1982.

7. D.L. DeJong, M.G.F. Peutz, and A.R. Kors-Wagon, Computer Program BISAR, Layered Systems under Normal and Tangential Surface Loads, Koninklijke/Shell-Laboratorium, Amsterdam, External Report AMSR - 0006.73, 1973.

8. R.N. Walker, W.D.O. Patterson, C.R. Freeme, and C.P. Marais, "The South African Mechanistic Pavement Design Procedure", Proceedings, Vol. 1, Fourth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1977, pp. 363-415.

9. P. Autret, A.B. De Boissoudy, and J.P. Marchand, "ALIZE III Practice", Proceedings, Vol. 1, Fifth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, and the Delft University of Technology, The Netherlands, Aug. 1982, pp. 174-191.

10. J.B. Rauhut and D.S. Gendell, "Proposed Development of Pavement Performance Prediction Models from SHRP/LTPP Data", Proceedings, Second North American Conference on Managing Pavements, Volume 2, Nov. 1987, pp. 2.21-2.37.

11. H.M. Westergaard, "Stresses in Concrete Pavements Computed by Theoretical Analysis", Public Roads, Vol. 6, No. 2, 1926, pp. 25-35.

12. J. Boussinesq, "Application des Potentiels a l'Etude de l'Equilibre et du Mouvement des Solids Elastiques", Gauthier-Villars, Paris (1885).

13. A.E.H. Love, "The Stresses Produced in a Semi-Infinite Body by Pressure on Part of the Boundary", Phil. Trans. Roy. Soc., Series A, Vol. 223, 1928, pp. 377-420.

14. C.R. Foster and R.G. Ahlvin, "Stresses and Deflections Induced by a Uniform Circular Load", HRB, Proceedings of the Thirty-third Annual Meeting, 1954, pp. 467-470.

15. R.G. Ahlvin and H.H. Ulery, "Tabulated Values for Determining the Complete Pattern of Stresses, Strains, and Deflections Beneath a Uniform Circular Load on a Homogeneous Half Space", HRB Bulletin 342, 1962, pp. 1-13.

16. J.L. Sanborn and E.J. Yoder, "Stress and Displacement in an Elastic Mass under Semi-Ellipsoidal Loads", Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1967, pp. 309-320.

17. D.M. Burmister, "The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways", HRB Proceedings, Vol. 23, 1943, pp. 126-144.

18. D.M. Burmister, "The General Theory of Stresses and Displacements in Layered Systems", Joun. Appl. Phys., Vol. 16, No. 2, pp. 89-96; No. 3, pp. 126-127; No. 5, pp. 296-302, 1945.

19. L. Fox, "Computation of Traffic Stresses in a Simple Road Structure", Department of Scientific and Industrial Research, Road Research Technical Paper 9, 1948.

20. W.E.A. Acum, and L. Fox, "Computation of Load Stresses in a Three-Layer Elastic System", Geotechnique, Vol. 2, 1951, pp. 293-300.

21. A. Jones, "Tables of Stresses in Three-Layer Elastic Systems", HRB Bulletin 342, 1962, pp. 176-214.

22. K.R. Peattie, "Stress and Strain Factors for Three-Layer Elastic Systems", HRB Buleltin 342, 1962, pp. 215-253.

23. J. Verstraeten, "Stresses and Displacements in Elastic Layered Systems", Proceedings, Vol. 1, Second International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1967, pp. 277-290.

24. M.K. Charyuln and J.B. Sheeler, "Theoretical Stress Distribution in an Elastic Multi-Layered System", HRB, Highway Research Record 228, 1968, pp. 11-17.

25. O.C. Zienkiewicz and Y.K. Cheung, "The Finite Element Method in Structural and Continuum Mechanics", McGraw-Hill, London, 1967.

26. R. Pichumani, "Theoretical Analysis of Airfield Pavement Structures", Air Force Weapons Lab., Kirtland Air Force Base, Tech. Rept. AFWL-TR-71-26, July 1971.

27. R.G. Hicks, "Factors Influencing the Resilient Properties of Granular Materials", Inst. of Transp. and Traffic Eng., University of California, Berkeley, Ph.D. Dissertation, May 1970.

28. D.A. Kasianchuk, "Fatigue Considerations in the Design of Asphalt Concrete Pavements", University of California, Berkeley, Ph.D. Dissertation, 1968.

29. K. Terzaghi, "The Static Rigidity of Plastic Clays", Journal of Rheology, Vol. 2, No. 3, July 1931.

30. C. Mack, "Rheology of Bituminous Mixtures Relative to the Properties of Asphalts", Proceedings of the AAPT, Vol. 13, 1942, pp. 194-255.

31. E.H. Lee, "Stress Analysis in Viscoelastic Bodies", Quarterly of Applied Mathematics, July 1955, pp. 183-190.

32. A.N. Biot, "Theory of Stress-Strain Relations in Anisotropic Viscoelasticity and Relaxation Phenomena", Journal of Applied Physics, Vol. 25, No. 11, Nov. 1954, pp. 1385-1391.

33. R.D. Barksdale and G.A. Leonards, "Predicting Performance of Bituminous Surface Pavements", Proceedings, Vol. 1, Second International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1967, pp. 321-340.

34. C.L. Monismith and K.E. Secor, "Viscoelastic Behaviour of Asphalt Concrete Pavements", Proceedings, Vol. 1, First International Conference on the Structural Design

of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1962, pp. 476-498.

35. W.J. Kenis, "Predictive Design Procedures - A Design Method for Flexible Pavements Using the VESYS Structural Subsystem", Proceedings, Vol. 1, Fourth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1977, pp. 101-130.

36. D.M. Burmister, "Evaluation of Pavement Systems of the WASHO Road Test by Layered System Methods", HRB Bulletin, No. 177, 1958, pp. 26-54.

37. S.F. Brown and P.S. Pell, "An Experimental Investigation of the Stresses, Strains, and Deflections in a Layered Pavement Structure Subjected to Dynanic Loads", Proceedings. Vol. 1, Second International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1967, pp. 487-504.

38. W.J. Kenis, J.A. Sherwood, and T.F. McMahon, "Verification and Application of the VESYS Structural Subsystem", Proceedings, Vol. 1, Fifth International Conference on the Structural Design of Asphalt Pavements, Delft University of Technology, The Netherlands Aug. 1982, pp. 333-346.

39. M.I. Darter, B.F. McCullough, and J.L. Brown, "Reliability Concepts Applied to the Texas Flexible Pavement System", Highway Research Record 407, 1972, pp. 146-161.

40. P. Irick, W.R. Hudson and B.F. McCullough, "Application of Reliability Concepts to Pavement Design", Proceedings, Vol. 1, Sixth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, July 1987, pp. 163-179.

41. W.R. Hudson, "Other Input Variables: Traffic and Environmental", TRB Special Report No. 140, 1973, pp. 65-77.

42. C.L. Monismith, J.A. Epps, D.A. Kasianchuk, and D.B. McLean, "Asphalt Mixture Behaviour in Repeated Flexure", Institute of Transportation and Traffic Engineering, University of California, Berkeley, Report TE 70-5, 1972.

43. The AASHO Road Test: Report 5 - Pavement Research, HRB Special Report 61E, 1962.

44. F.H. Scrivner, "A Theory for Transforming AASHO Road Test Pavement Performance Equation to Equations Involving Mixed Traffic", HRB Special Report 66, 1961, pp. 39-46.

45. The Asphalt Institute, "Thickness Design - Asphalt Pavement Structures for Highways and Streets", The Asphalt Institute, College Park, Maryland, Asphalt Institute Manual Series No. 1, 6th Edition, 1963, pp. 50.

46. C.J. Van Til, B.F. McCullough, B.A. Vallerga, and R.G. Hicks, "Evaluation of AASHO Interim Guides for Design of Pavement Structures", NCHRP Report 128, 1972, pp. 63-67.

47. C.J. Derdeyn, "A New Method of Traffic Evaluation for Pavement Design", Highway Research Record 46, 1964, pp. 1-10.

48. J.A. Deacon, "Fatigue Life Prediction", TRB Special Report 140, 1973, pp. 78-92.

49. Roads and Transportation Association of Canada, Pavement Management Guide, 1977, pp. 4.40-4.42.

50. Roads and Transportation Association of Canada, Vehicle Weights and Dimensions Study - Technical Steering Committee Report, Dec. 1986, pp. 1-34.

51. B.P. Shields, "Truck Factors 1970-1972", Alberta Research Council, Highway and River Engineering Division, Report HTE/74/72, March 1974.

52. J.T. Christison and B.P. Shields, "Pavement Elastic Moduli by the Surface Wave Method Correlated with In-Situ Measurements of Elastic Response Under Moving Vehicle Loads", Proceedings, Vol. 19, Canadian Technical Asphalt Association, Regina, Saskatchewan, 1974, pp. 229-252.

53. W. Heukelom and A.J.G. Klomp, "Road Design and Dynamic Loading", Proceedings, AAPT, Vol. 33, 1964, pp. 92-125.

54. G. Van der Poel, "A General System Describing the Viscoelastic Properties of Bitumens and Its Relation to Routine Test Data", Jour. Appl. Chem., May 1954, pp. 221-236.

55. K.E. Secor and C.L. Monismith, "Viscoelastic Response of Asphalt Paving Slabs Under Creep Loading", Highway Research Record No. 67, 1965, pp. 84-97.

56. G.A. Secor, K.E. Secor, and C.L. Monismith,

"Temperature Induced Stresses and Deformations in Asphalt Concrete", Proceedings, AAPT, Vol. 34, 1965, pp. 248-285.

57. H.S. Papazian, "The Response of Linear Viscoelastic Materials in the Frequency Domain with Emphasis on Asphalt Concrete", Proceedings, Vol. 1, First International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, 1962, pp. 454-463.

58. C.A. Pagen, "Rheological Response of Bituminous Concrete", Highway Research Record No. 67, 1965, pp. 1-26.

59. P.S. Pell and K.E. Cooper, "The Effect of Testing and Mix Variables on the Fatigue Performance of Bituminous Materials", Proceedings, AAPT, Vol. 44, Phoenix, Arizona, Feb. 1975, pp. 1-37.

60. J.K. Mitchell and C.K. Chen, "Soil-Cement Properties Determined by Repeated Leading in Relation to Bases for Flexible Pavements", Proceedings, Vol. 1, 2nd International Conference on Structural Design of Asphalt Pavements, 1967, University of Michigan, Ann Arbor, Michigan, pp. 427-451.

61. M.C. Wang, "Stress and Deflections in Cement Stabilized Soil Pavements", University of California, Berkeley, Ph.D. Dissertation, 1968.

62. P.S. Pell and S.F. Brown, "The Characteristics of Materials for the Design of Flexible Pavement Structures", Proceedings, Vol. 1, 3rd International Conference on the Design of Asphalt Pavements, London, England, Sept. 1972, pp. 326-342.

63. H.B. Seed, F.G. Mitry, C.L. Monismith and C.K. Chan, "Factors Influencing the Resilient Deformations of Untreated Aggregate Base in Two-Layer Pavements Subjected to Repeated Loadings", Highway Research Record No. 140, 1967, pp. 19-57.

64. H.B. Seed, F.G. Mitry, C.L. Monismith and C.K. Chan, "Prediction of Flexible Pavement Deflections from Laboratory Related Load Tests", NCHRP Report No. 35, Highway Research Board Publication, 1967, pp. 6-28.

65. S.F. Brown, "Repeated Load Testing of a Granular Material", Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. G77, July 1974, pp. 825-841.

66. H.B. Seed and C.K. Chan, "Effect of Duration of Stress

Application on Soil Deformation Under Repeated Loading", Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1964, pp. 341-345.

67. B.S. Coffman, D.C. Kraft and J. Tamayo, "A Comparison of Calculated and Measured Deflections for the AASHO Road Test", Proceedings, AAPT, Vol. 33, 1964, pp. 54-91.

68. J.H. Haynes and E.J. Yoder, "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test", Highway Research Record No. 39, 1963, pp. 649-666.

69. W.A. Dunlap, "A Report on a Mathematical Model Describing the Deformation Characteristics of Granular Materials", Technical Report No. 1, Texas Transport Institute, Texas A & M University, 1963.

70. D.H. Trollope, I.K. Lee and J. Morris, "Stresses and Deformation in Two-Layer Pavement Structures Under Slow Repeated Loading", Proceedings, Australian Research Board, Vol. 1, Part 2, 1962, pp. 693-721.

71. H.B. Seed, C.K. Chan, and C.E. Lee, "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements", Proceedings, Vol. 1, 1st International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, 1962, pp. 611-636.

72. B.F. Kallas and J.F. Shock, "San Diego County Experimental Base Project", Final Report, The Asphalt Institute, Research Report 77-1 (RR-77-1), Nov. 1979.

73. D. Croney and J.N. Bulman, "The Influence of Climatic Factors on the Structural Design of Flexible Pavements", Proceedings, Vol. 1, 3rd International Conference on the Structural Design of Asphalt Pavements, London, England, 1972, pp. 67-71.

74. C.R. Marek and B.J. Dempsey, "A Model Utilizing Climatic Factors for Determining Stresses and Deflections in Flexible Pavement Systems", Proceedings, Vol. 1, 3rd International Conference on the Structural Design of Asphalt Pavements, London, England, 1972, pp. 101-114.

75. R.C.G. Haas and T.H. Topper, "Thermal Fracture Phenomena in Bituminous Surfaces", HRB Special Report No. 101, 1969, pp. 136-153.

76. M.Y. Shahin and B.F. McCullough, "Prediction of Low-

Temperature and Thermal-Fatigue Cracking", Texas Highway Department, Texas Transp. Inst., Texas A & M University, and Centre for Highway Research of Texas at Austin, Research Report 123-14, 1973.

77. J.T[illegible]stison, "The Response of Asphaltic Concrete Pave[illegible]s to Low Temperatures", University of Alberta, Edmonton, Ph.D. Dissertation, 1972, 323 pp.

78. R.G. Robinson, "A Model for Calculating Pavement Temperatures from Meterological Data", TRRL, Digest of Research Report RR44, 1986.

79. B.F. Kallas, "Asphalt Pavement Temperatures", Highway Research Record 150, 1966, pp. 1-11.

80. C.L. Monismith and J.A. Deacon, "Fatigue of Asphalt Paving Mixtures", Jour. Transp. Eng. Div., ASCE, No. TE2, 1969, pp. 317-346.

81. P.S. Pell, "Characterization of Fatigue Behaviour", TRB Special Report 140, 1973, pp. 49-64.

82. P.S. Pell and I.F. Taylor, "Asphaltic Road Materials in Fatigue", Proceedings, AAPT, Vol. 38, 1969, pp. 371-422.

83. F. Finn, C. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abdullah, "The Use of Distress Prediction Subsystems for the Design of Pavement Structures", Proceedings, Vol. 1, 4th International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, Aug. 1977, pp. 3-38.

84. F. Finn, C. Saraf, R. Kulkarni, N. Nair, W. Smith, and A. Abdullah, "Development of Pavement Structural Subsystems", NCHRP Report 291, Dec. 1986.

85. J.M. Kirk, "Relations Between Mix Design and Fatigue Properties of Asphaltic Concrete", Proceedings, 3rd International Conference on the Structural Design of Asphalt Pavements, London, England, 1972, pp. 241-247.

86. R.L. Terrel, "Examples of Approach and Field Evaluation: Research Applications", TRB Special Report 140, 1973, pp. 98-111.

87. J. Morris, "The Prediction of Permanent Deformation in Asphalt Concrete Pavements", University of Waterloo, Ph.D. Dissertation, Aug. 1973.

88. R.D. Barksdale, "Performance of Asphalt Concrete Pavements", Transportation Engineering Journal, Vol. 103, Jan. 1977.

89. R.C.G. Haas and A.T. Papagianakis, "Understanding Pavement Rutting", RTAC, Special Workshop on Rutting in Asphalt Pavements, Toronto, Ontario, Sept. 1986.

90. C.L. Monismith, "Rutting Prediction in Asphalt Concrete Pavements", TRB Record 616, 1976, pp. 2-8.

91. G.M. Dormon and C.T. Metcalf, "Design Curves for Flexible Pavements Based on Layered Elastic Theory", HRB Record No. 71, 1965, pp. 69-84.

92. J.E. Romain, "Rut Depth Prediction in Asphalt Pavements", Proceedings, Vol. 1, 3rd International Conference on the Structural Design of Asphalt Pavements, London, England, 1972, pp. 205-210.

93. R.D. Barksdale, "Laboratory Evaluation of Rutting in Base Course Materials", Proceedings, Vol. 1, 3rd International Conference on the Structural Design of Asphalt Pavements, London, England, 1972, pp. 161-174.

94. J.M. Duncan and C.Y. Chang, "Nonlinear Analysis of Stress and Strain in Soils", ASCE, Journal of Soil Mechanics and Foundation Engineering, Vol. 96, 1970, pp. 1629-1653.

95. J. Morris, R.C.G. Haas, P. Reily and H. Hignell, "Permanent Deformation in Asphalt Pavements can be Predicted", Proceedings, AAPT, Vol. 43, 1974, pp. 41-76.

96. F.R.P. Meyer and R.C.G. Haas, "A Working Design Subsystem for Permanent Deformation in Asphalt Pavements", Proceedings, Vol. 1, 4th International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, 1977, pp. 519-528.

97. F.L. Roberts, H. von Quintus, and W.R. Hudson, "Design Procedure for Premium Flexible Pavements", Proceedings, Vol. 1, 5th International Conference on the Structural Design of Asphalt Pavements, The Delft University of Technology, The Netherlands, 1982, pp. 92-115.

98. J. Verstraeten, V. Veverka, and L. Franchen, "Rational and Practical Design of Asphalt Pavements to Avoid Cracking and Rutting", Proceedings, Vol. 1, 5th International Conference on the Structural Design of Asphalt Pavements, The Delft University of Technology, The Netherlands, 1982, pp. 45-58.

99. R.L. Lytton, C.H. Michalak, and T. Scullion, "The Texas Flexible Pavement Design System", Proceedings,

Vol. 1, 5th International Conference on the Structural Design of Asphalt Pavements, The Delft University of Technology, The Netherlands, 1982, pp. 131-141.

3.0 ASPHALT INSTITUTE METHOD OF DESIGN

3.1 General

This chapter is concerned with the revision of the development of the current (1981) Asphalt Institute thickness design procedure for asphalt concrete pavements. Firstly, an historical review of the various editions of the thickness design manual and the basic relationships upon which these were originally based will be presented. Secondly, the development of the new mechanistic procedure and the basic input parameters needed for the thickness design will be discussed in detail.

3.2 Historical Background

Starting in 1956, The Asphalt Institute published a set of thickness design relationships in which the total pavement thickness was related to the traffic composition, design wheel load, subgrade support value (California Bearing Ratio, CBR, Stabilometer R-value or Plate Load Test value), and the amount of asphalt concrete base used above certain minimums. The last edition of the Thickness Design Manual in which those relationships were used was the sixth, published in 1962 (1).

The 1956-62 design relationships included in the first through the sixth editions of the Thickness Design Manual were based, in part, on the concept that the total thickness of cover over any layer should be sufficient to prevent shear failure in that layer, and that the cover thickness was not affected by the characteristics of the overlying material. This concept was further modified by provision for different traffic classifications, minimum thickness requirements, and the permission of substitutions of lesser thicknesses of asphalt concrete base for portions of granular base.

The basic design curve using the above described concept has the approximate formula (L):

$$T = \frac{\sqrt{L/2}}{CBR^{0.6}} \tag{3.1}$$

where

T = total thickness of the cover over the subgrade, in., and

L = single-axle load, lb.

The effect of traffic was also incorporated in the design chart by classifying the traffic into four categories: very heavy, heavy, medium and light traffic.

In 1963, an extensive revision of the sixth edition of the Thickness Design Manual was completed and a seventh edition was issued. The revisions included new thickness design relationships, and additional or revised recommendations on subgrade strength evaluation, subgrade compaction, stage construction, methods for making economic comparisons, drainage, frost and many other design details.

The new design relationships included in the seventh edition (2), were developed largely on the basis of observed performance from the AASHO Road Test, together with information from the WASHO Road Test, British test roads, and comparisons with then - existing design procedures of the U.S. Corps of Engineers and state agencies (3,4,5).

The steps used in deriving the mathematical relationships included in the seventh edition were summarized by Shook (5) as follows: First, data from the AASHO Road Test were analyzed to provide the basic relationship between pavement design and the number and weight of axle loads applied to the pavement. Second, means were developed to extrapolate these equations to subgrade soils other than that encountered at the test road site. Third, mathematical rearrangement of the equations, to permit determination of equivalent axle load factors, was made to enable application of the design method to traffic consisting of mixtures of axle loads of different weights. Finally, the equations were put into a convenient form and plotted for use as thickness design charts.

In 1969, an eighth edition of the Thickness Design Manual was issued which incorporated revisions with regard to procedures for estimating equivalent axle loads with new thickness design charts. This eighth edition reflected some changes from the seventh edition; however, the basic thickness design relationship, remained unchanged.

3.3 Development of the Asphalt Institute Mechanistic Procedure

In 1977, the Asphalt Institute initiated an effort to explore the possibility of using mechanistic design procedures to revise the eighth edition of the Thickness Design Manual. In doing so, both mechanistic and empirical state-of-the-art information related to pavement design were utilized. A mechanistic procedure was intended for the revision for the sake of bypassing the limitations usually imposed by empirical procedures. Furthermore, it was considered desirable to eliminate the use of layer equivalencies in the new design procedure.

A major improvement in the ninth edition of the Thickness Design Manual MS-1 (6), is its inclusion of structural performance characteristics in addition to the functional characteristics used in the seventh and eighth editions. In this regard, fatigue cracking and excessive permanent deformation in the wheel path area were selected as the distress criteria indicative of structural adequacy of the designed pavement.

One other useful feature of the ninth edition of MS-1, is the incorporation of improved and practical procedures for estimating ESAL's, or equivalent 80 kN (18,000 lb) single-axle loads. In this regard, a knowledge of the truck-type distribution and volume is considered essential for the estimation of the equivalent single axle load applications of the traffic anticipated to use the facility during its design life.

During the development stages of the new design procedure included in the ninth edition of MS-1, a computer program utilizing an elastic pavement model was developed by Witczak (6). This computer program, which will later be referred to as the DAMA program, comprised the analytical tool used for producing the design charts included in the Manual.

It is also worth mentioning that the criteria used for both fatigue cracking and permanent deformation are based on empirical data. The fatigue cracking prediction is based on data taken from the AASHO Road Test observations (7) and rutting is based on field observations of pavements in California. Thus, the distress criteria is in fact a combination of both mechanistic and empirical input.

3.4 Design Principles

In the Asphalt Institute 1981 thickness design procedure (6), the pavement is considered as a multi-layered elastic model. The materials in each of the layers are characterized by a modulus of elasticity and a Poisson's ratio. Traffic is expressed in terms of repetitions of an equivalent 80 kN (18000 lb) single-axle load applied to the pavement. For purposes of analysis, each set of dual tires is approximated by two circular plates with radius of 115 mm (4.52 in.) spaced 345 mm (13.5 in.) apart. A load of 20 kN (4500 lb) on each plate is used, corresponding to an 80 kN total single axle load and a 483 kPa (70 psi) contact pressure.

The design procedure can be used to design both full-depth and conventional asphalt pavements. For full-depth asphalt pavements the pavement is regarded as a three-layer system, whereas a conventional pavement with untreated aggregate base layer is considered a four-layer system as depicted in Fig. 3.1.

Following the assumption of an elastic multi-layered pavement system, all layers apart from the subgrade are assumed infinite in extent in the horizontal direction. On the other hand, the subgrade layer is assumed to be infinite in both the vertically downward and horizontal directions. Furthermore, full continuity i.e. full friction is assumed at all interfaces between each of the layers.

Loads at the pavement surface are considered responsible for producing two strains which are believed critical for design purposes. These are: (1) the horizontal tensile strain, ε_t, on the underside of each asphalt-bound layer, either asphalt concrete or emulsified asphalt treated, and (2) the vertical compressive strain, ε_v, at the top of the subgrade layer.

Exceeding the maximum horizontal tensile strain, ε_t, which the asphalt-treated layer can accommodate will result in excessive fatigue cracking. On the other hand, if the vertical compressive strain, ε_v, is excessive, permanent deformation will result at the pavement surface due to overstressing the subgrade. Excessive deformation in the treated layers is expected to be controlled by limits on materials properties and adequate construction procedures.

To cater for the above described distress subsystems, two design criteria, one for fatigue cracking and the other for permanent deformation, were selected by the Asphalt Institute to be incorporated in the design procedure.

The fatigue criterion utilized was the one developed by Finn, et al. (7), for NCHRP project 1-10B. The relationship, expressed in logarithmic form, is:

$$\log N_f = 16.086 - 3.291 \log (\varepsilon_t/10^{-6})$$
$$- 0.854 \log (|E^*|/10^3) \qquad (3.2a)$$

or

$$N_f = 18.4 \left(4.32 \times 10^{-3}\ \varepsilon_t^{-3.291} |E^*|^{-0.854}\right) \qquad (3.2b)$$

where

N_f = number of load applications to cracking

ε_t = tensile strain in asphalt layer, mm/mm, (in./in.)

$|E^*|$ = asphalt mixture dynamic stiffness, MPa (psi)

This relationship was [illegible]ed from laboratory fatigue data presented by Monismith [illegible] al. (8), and adjusted to provide an indication of approximately 20 percent or greater fatigue cracking (based on total pavement-area) in selected sections of the AASHO Road Test.

To reflect the effect of both air voids content and asphalt content on fatigue performance, the correction factor C has been developed and incorporated in equation 3.2 above. This correction factor, C, is expressed mathematically as:

$$C = 10^{M} \tag{3.3}$$

where

$$M = 4.84 \left(\frac{V_b}{V_v + V_b} - 0.69\right) \tag{3.4}$$

V_b = volume of asphalt, percent, and

V_v = volume of air voids, percent.

The term M was obtained from laboratory fatigue data developed by Pell and Cooper (9) and Epps (10).

The second design criterion selected by the Asphalt Institute to limit the vertical compressive strain at the top of the subgrade layer, takes the form:

$$N = 1.365 \times 10^{-9}\ \varepsilon_c^{(-4.477)} \tag{3.5}$$

where

N = number of load applications to produce excessive rutting

ε_c = vertical compressive strain at the top of subgrade layer, mm/mm

This criterion was obtained from analysis of pavements designed by the California procedure and analysed using the Chevron method (11). With this criterion, so long as good compaction of the pavement components is obtained and the asphalt mix is well designed, rutting should not exceed approximately 13 mm (0.5 in.) for the design traffic. However, if such conditions were not met, an overestimation of the design life of the pavement system will result.

3.5 Materials Characterization

In the development of the design procedure, all pavement materials were characterized by a modulus of elasticity (usually called dynamic modulus for asphalt mixes, or resilient modulus for untreated granular and subgrade soils), E, and a Poisson's ratio, ν. Specific values were selected based on extensive testing and experience.

3.5.1 Asphalt Concrete Moduli

As pointed out earlier, the asphalt dynamic stiffness, $|E^*|$, is strongly dependent on the mix temperature. To simulate the effects of temperature as it changes through the year, a typical distribution of mean monthly air temperatures (MMAT) characterizing the specific region for which the pavement facility was to be designed, was used as one of the input parameters for the computer program DAMA.

In the original development of the design procedure, three sets of environmental conditions were selected to represent the range of conditions to which the manual should apply (12):

Mean Annual Air Temperature (MAAT)	Frost Effects	Asphalt Grades
< 7°C (45°F)	Yes	AC-5, AC-10
15.5°C (60°F)	Possible	AC-10, AC-20
> 24°C (75°F)	No	AC-20, AC-40

Mean Annual Air Temperature (MAAT) was used to characterize the environmental conditions applicable to each region, and the characteristics of the materials were selected accordingly. However, only design charts for the 15.5°C condition were enclosed in the manual.

The asphalt viscosities associated with each of the three temperature regimes used in developing the design charts were selected as representative of practices in such specific regions. In the colder regions (<7°C) softer asphalt grades have been recommended to be used (AC-5 or AC-10) in order to minimize thermal stresses which may lead to transverse cracking. On the other hand, harder asphalt grades have been used in warmer regions (> 24 °C) in order to limit the permanent deformation and improve resistance to fatigue cracking.

Although it is not necessary to know the asphalt mixture stiffness to use the manual design charts directly, it is crucial to have temperature-modulus data to use the computer program DAMA, or to have other input data that can be used to generate temperature-modulus relationships. In the latter case, the Asphalt Institute proposed the following relationship:

$$\begin{aligned}
\text{Log } |E^*| &= 5.553833 + 0.028829 \left(\frac{P_{200}}{f^{0.17033}}\right) \\
&- 0.03476(V_v) + 0.070377 \left(\eta_{70^0F,10^6}\right) \\
&+ 0.000005 \left[t_p^{(1.3+0.49825 \log f)} P_{ac}^{0.5}\right] \\
&- 0.00189 \left[t_p^{(1.3+0.49825 \log f)} \frac{P_{ac}^{0.5}}{f^{1.1}}\right] \\
&+ 0.931757 \left(\frac{1}{f^{0.02774}}\right) \qquad (3.6)
\end{aligned}$$

where,

$\|E^*\|$	= dynamic modulus (stiffness) of asphalt concrete, psi (kPa/6.8948),
P_{200}	= percent aggregate passing #200 sieve,
f	= frequency of loading, Hz,
V_v	= % air voids
$\eta_{70°F,10^6}$	= absolute viscosity at 70°F, poises x 10^6
P_{ac}	= asphalt content, % by weight of mix, and
t_p	= asphalt concrete temperature, °F (1.8°C + 32).

If sufficient viscosity data are not available in order to estimate $\eta_{70°F,10^6}$, then the following relationship may be used:

$$\eta_{70^0F,10^6} = 29508.2\ pen_{77^0F}^{-2.1939} \qquad (3.7)$$

where,

$pen_{77°F}$ = asphalt penetration at 77°F, dmm.

The value of P_{ac} may be estimated as follows:

$$P_{ac} = 0.483V_{bc} \qquad (3.8)$$

or

$$P_{ac} = 0.434V_b \qquad (3.9)$$

where,

V_{bc} = effective volume of asphalt, %, defined as the total asphalt content minus the quantity lost by absorption into the aggregate particles

V_b = % total volume of asphalt.

Equations (3.6) to (3.9) were derived by regression analysis of actual test data. Comparisons of measured and predicted values for $|E^*|$ from 41 different mixes tested at three different temperatures of 4.4°C, 21.1°C, and 37.8°C, and under three different load frequencies of 1, 4 and 16 Hz showed good correlation indicative of the suitability of such equations to reasonably estimate the modulus value

(12).

In addition to being used to generate temperature modulus data for preparing design charts, eq. (3.6) can also be used (with the program DAMA) to study the effects of the mixture propeties on the thickness design requirements.

With regard to the Poisson's ratio, the Asphalt Institute suggests that a value of 0.35 is to be used for asphalt mixes.

3.5.2 Asphalt Emulsion Mixes

The asphalt emulsion mix stiffness relationships were based on data developed by Chevron Research Company (11) for 32 different mixes tested at 23°C (73°F) and 38°C (100°F). The analysis of these data lead to the identification of three different mix categories termed in the design manual as Types I, II, and III.

Since the asphalt emulsion mixes develop greater modulus value after some time is past after the construction, data for those materials in the uncured (initial) state together with some measure of the rate of development of the modulus to the cured value are required. Following the method developed by the Chevron Research Company (11), a temperature-modulus relationship of the form:

$$E_{T,t} = E_{T,f} - (E_{T,f} - E_{T,i})(RF_t) \tag{3.10}$$

where,

$E_{T,t}$ = modulus at temperature T and any time t

$E_{T,f}$ = modulus at fully cured state at temperature T

$E_{T,i}$ = modulus in uncured (initial) state at temperature T

RF_t = reduction factor representing the amount of cure at time (t) and defined as shown in Fig. 3.2 for a 6-month cure period

is adopted by the Asphalt Institute to obtain the modulus value for such materials.

Furthermore, all emulsion mixes were assumed to have a maximum stiffness of 5 x 10^6 psi at -29°C (-20°F) in the fully cured state and a minimum of 50,000 psi in the uncured state.

Also, an analysis conducted by the Asphalt Institute disclosed that longer cure periods (> 6 months) have no significant influence on thickness requirements; thus a 6-month curing period was selected and used in developing the manual design charts.

The fatigue response for each of the asphalt emulsion mixes were assumed to be the same as those containing asphalt concrete.

A Poisson's ratio value of 0.35 is also adopted for asphalt emulsions.

3.5.3 Untreated Granular Materials

From among the many available relations that exist, the

Asphalt Institute selected the following one to be used for obtaining the untreated granular materials modulus, namely

$$M_r = k_1 \theta^{k_2} \qquad (3.11)$$

where,

M_r = resilient modulus of untreated aggregate layer

θ = first stress invarient which equals the sum of the principle stresses, and

k_1, k_2 = experimentally determined constants for each aggregate material type.

From an analysis of the available data, (in the U.S. Customary Units), k_2 was set equal to 0.5 and k_1 was selected to be in the range 8000 to 12000 when θ is expressed in psi.

The direct use of eq. 3.11 above in a layered elastic analysis requires an iterative type of solution. To avoid this complexity in the program DAMA, an indirect predictive equation (13) has been utilized:

$$E_{ua} = M_r = g_o(h_1^{-g_1})(h_3^{-g_2})(E_1^{-g_3})(E_{sg}^{g_4})(K_1^{g_5}) \qquad (3.12)$$

where,

$g_o, \ldots, g_5$ = regression constants with the following values:

g_o = 10.447 $\qquad$ g_3 = 0.139

g_1 = 0.471 $\qquad$ g_4 = 0.287

$g_2 =$ [illegible]1 $\qquad g_5 = 0.868$

and,

E_{sg} = subgrade modulus, psi.

h_3 = thickness of untreated aggregate layer, in.

h_1 = asphalt layer (total) thickness, in.

E_1 = modulus of the asphalt layer, psi.

When two different asphalt stabilized layers are present (with h_1, E_1 and h_2, E_2), the equivalent modulus of the combined asphalt layers with thickness of $(h_1 + h_2)$ is given by:

$$E_1{}'_{eq.} = \left[\frac{h_1 E_1^{1/3} + h_2 E_2^{1/3}}{h_1 + h_2}\right]^3 \tag{3.13}$$

The use of the indirect predictive equation (eq. 3.12) in the program DAMA is reported to produce essentially the same subgrade compressive strain as if the iterative procedure were used.

Material quality for the untreated aggregate layer is represented by the input factor K_1 ($K_2 = 0.5$ is always assumed) which will assume higher values for better quality aggregates.

As far as Poisson's ratio goes, a value of 0.40 is usually assigned to these materials.

3.5.4 Subgrade Soils

For the case of subgrade materials, the manual does not use a particular modulus relation for estimating the subgrade modulus; rather the subgrade moduli values determined by either dynamic testing or any other means are used directly as input in the program DAMA.

To get a realistic value representative of the subgrade modulus to be used in the design process, careful simulation of the in-situ field conditions is required. This is strictly true due to the fact that the subgrade modulus is a function of both type and degree of compaction as well as the stress state that exists within the subgrade layer. As an example, flocculated and dispersed soil structures behave differently under the same conditions of dry density and water content. Also increasing the applied stress tends to significantly decrease the subgrade modulus value.

To obtain the subgrade modulus from laboratory dynamic testing, the Asphalt Institute recommended that six to eight tests be performed for each soil type with an absolute minimum of five recommended. It must be emphasized that the number of tests (6-8 tests) refer to a specific condition of the sample which will usually be the prevailing in-situ conditions of dry density, water content and stress-state. The procedure then used to select the design resilient subgrade modulus from the test results, can be outlined as follows (6):

(1) Arrange all test values in numerical order.

(2) For each change in test value, beginning with the lowest value, compute the percentage of the total number of values that is equal to, or greater than.

(3) Plot the percent equal to or greater than vs. M_r.

(4) Read from the curve plotted in step (3) above, the subgrade strength value at the 60 percent (for ESAL of 10^4 or less), 75 percent (ESAL between 10^4 and 10^6), or 87.5 percent (ESAL > 10^6) as appropriate. This is the design subgrade strength value.

An example illustrating these steps is depicted in Fig. 3.3.

Relationships between the subgrade modulus and conventional pavement design soil tests like the CBR and the R-value have also been included in the manual as a substitute for determining the subgrade modulus. These relationships are based on correlations with either field or laboratory stiffness measurements. Accordingly, caution must be exercised when using such correlations for estimating the subgrade modulus since the values thus obtained are approximate and are subjected to variations within the limits established by the correlations.

The relationship betwen resilient modulus and CBR is that established by Shell (14) and used in their design procedure. This relationship takes the form

$$M_r = 10.3 \text{ CBR} \quad \text{(MPa)} \tag{3.14a}$$

or

$$M_r = 1500 \text{ CBR} \quad \text{(psi)} \tag{3.14b}$$

It should be noted that M_r values determined from such relation can vary from 0.5 to 2.0 times that amount. Moreover, available data have indicated that the above equations provide better results at values of CBR less than about 20.

The relationships between the resilient modulus and R-value were developed originally from data collected on the San Diego County Experimental Base Project (15). Two relationships were developed among which the following is the one adopted by the Asphalt Institute in its design procedure:

$$M_r \text{ (MPa)} = 7.96 + 3.83 \text{ (R)} \tag{3.15a}$$

or

$$M_r \text{ (psi)} = 1155 + 555 \text{ R} \tag{3.15b}$$

As with the M_r-CBR relationship, the M_r-R-value relationship also gives approximate subgrade moduli values which must be used with caution in the design process.

3.6 Environmental Considerations

The effects of both temperature and moisture variations on pavement materials properties, were incorporated in the MS-1 design procedures.

The effects of temperature variations on the dynamic modulus of asphalt concrete and emulsified asphalt mixtures were accounted for by the utilization of appropriate modulus-temperature relationships described earlier in sections 3.5.2 and 3.5.3.

Studies summarized by Witczak (16) indicate that pavement temperatures can be estimated from air temperatures, using the following relationship:

$$MMPT = MMAT \left[1 + \frac{1}{(Z + 4)}\right] - \frac{34}{(Z + 4)} + 6 \qquad (3.16)$$

where,

MMPT = mean monthly pavement temperature, °F

MMAT = mean monthly air temperature, °F, and

Z = depth below pavement surface, in.

The mean monthly pavement temperatures obtained using eq. 3.16 were then used as input in the appropriate temperature modulus relationship to obtain the values of the monthly dynamic moduli to be used for either the asphalt concrete or emulsified asphalt layers.

The combined effect of both moisture and sub-zero air temperatures on the subgrade and untreated granular base and subbase layers were also compensated for by using an increased modulus value to represent the freezing period and a reduced modulus value to represent the thaw period.

To enable the assignment of appropriate subgrade resilient moduli to be used under temperature regimes which include freezing conditions, a schematic diagram like the one shown in Fig. 3.4 was used to reflect the subgrade modulus variations throughout the year. Using this approach, a maximum subgrade modulus value of 50000 psi was usually used to represent the frozen subgrade condition whereas a reduced (or minimum) subgrade modulus representing the thaw condition was given by the expression:

$$E_{ts} = r_t E_{ns} \tag{3.17}$$

where,

E_{ts} = thaw (reduced) subgrade modulus,

r_t = thaw reduction factor, and

E_{ns} = normal subgrade modulus.

Tables 3.1 and 3.2 contain a summary of the various parameters shown schematically in Fig. 3.3 for the MAAT = 7°C (45°F) and 15.5°C (60°F) conditions while Tables 3.3 and 3.4 show examples of the actual subgrade stiffness values to be used in the computations (12).

A similar procedure was used to adjust the modular values for the untreated granular sections. In this instance, the value of K_1 (eq. 3.11) was increased by a factor of 300 percent for the frozen condition and reduced during spring thaw to a level of 25 percent of the unfrozen value for conditions that have frost. Table 3.5 (12) contains a summary of the monthly K_1 values used for MAAT of 7°C (45°F) and 15.5°C (60°F). The value of K_2 was maintained constant at 0.5.

Seasonal reductions in the granular layer are not unusual. Finn and Shook (3) found it necessary to make such adjustment in the granular layer in order to properly match measurements of deflection on the AASHO Road Test.

Additional justification for reduced granular material moduli come from the relatively poor drainage characteristics during the spring thaw period.

3.7 Traffic Analysis

This section describes the methods for determining appropriate traffic input parameters to be used for the structural design of the desired pavement facility.

Of primary concern are the number and weights of the axle loads expected to be applied to the pavement during a given period of time.

The traffic analysis procedure used in the MS-1 design manual incorporates the conversion of all mixed traffic into equivalent numbers of an 80 kN (18000 lb) single-axle load applications (ESAL). Conversion of the mixed traffic into ESAL's is obtained by the use of appropriate load equivalency factors. These factors were taken from AASHO Interim Guide (17) and were based on data from the AASHO Road Test.

3.7.1 Traffic Volume Estimates

The traffic analysis procedures used in the design manual require that estimates be obtained of the number of vehicles of different types, such as ~~passenger ca~~rs, buses, single-unit trucks, and multiple-unit trucks of different types expected to use the proposed facility. Adjustments are then made to that volume to reflect both the relative proportion of the trucks to be expected in the design lane and the anticipated future traffic growth.

3.7.2 Estimating ESAL's

The terms listed below are identified by the design manual as the ones to be used for estimating the ESAL's expected to be applied to the pavement structure during its design life (6):

- Truck Factor - The number of equivalent 80 kN (18000 lb) single-axle load applications contributed by one passage of a vehicle.
- Load Equivalency Factor - The number of equivalent 80 kN (18000 lb) single-axle load applications contributed by one passage of an axle.
- Number of Vehicles - Total number of vehicles involved.

ESAL is calculated by multiplying the number of vehicles in each weight class by the appropriate Truck Factor (TF) and obtaining the sum of the products:

$$\text{ESAL} = \Sigma\,(\text{No. of vehicles in each weight class} \times \text{TF}) \qquad (3.18)$$

The Truck Factors are determined from axle-weight data using Load Equivalency Factors. This is usually done by multiplying the number of axles in each weight class by the appropriate Load Equivalency Factor and dividing the sum of the products by the total number of vehicles involved.

$$\text{TF} = \frac{(\text{No. of axles} \times \text{Load Equivalency Factor})}{\text{number of vehicles}} \qquad (3.19)$$

Load equivalency factors used in eq. 3.19 above were based originally on data collected during the AASHO Road Test; the procedure used for calculating Truck Factors is best illustrated by the example taken from the design manual and shown in Fig. 3.5.

3.7.3 Parkways and Shoulders

Residential streets which carry relatively high, almost 100 percent, light traffic must be designed to withstand not only such traffic but also occasional heavy traffic, such as snowplows, maintenance trucks, ready-mixed concrete trucks, buses, and garbage trucks. The MS-1 design manual recommends that a realistic estimate of the future traffic, including the occasional heavy traffic like the special types of trucks used during construction, be made, or else minimum thickness recommendations should be maintained.

For parkways that are restricted to buses and light automobiles, experience showed that design ESALs calculated only for high volumes of light axles often results in pavements that are too thin. Therefore, when heavy truck traffic represents not more than 2 percent, a reasonable estimate of the design ESAL can be obtained by multiplying total traffic, including automobiles, trucks, and buses by a truck factor of 0.06.

The manual also recommends that for minimum protection against damaging effects of occasional heavy vehicles, the design ESAL for shoulders should be equal to or greater than

2 percent of the design ESAL for the design lane.

3.8 Computer Program DAMA

The computer program DAMA was developed at the University of Maryland by Witczak (18) to provide the computational basis for developing the structural design charts included in the Asphalt Institute's MS-1 manual. Essentially, the program is used to analyze multi-layered elastic pavement structures by cumulative damage techniques for a single or dual wheel load system. Because program DAMA analyzes a given pavement cross-section structure, it is not a design program per se. However, structural designs can easily be obtained by analyzing a factorial set of pavement structures (i.e. multiple DAMA runs).

In the developmental stage of program DAMA, two separate versions were formulated. One version utilizes the Shell BISAR elastic-layered stress-strain-displacement program (called DAMA-BISAR) while the second version uses the Chevron N-layer program as the theoretical layered system (called DAMA-CHEV). Because of the differences between the BISAR and the Chevron N-layer programs, the flexibility of each version of the DAMA program to handle various wheel load configurations and layer interface degree of friction depends upon the theoretical stress program used as a subroutine. In Table 3.6 some of the differences between the two versions of the DAMA program are summarized (12).

In view of the cost per run for each version of the program DAMA coupled with practical considerations, the DAMA-CHEV program was selected as the major analytical source used for the development of the design curves included in the MS-1 manual.

The DAMA program routinely prints selected deflection and critical strain values at specified computational points along each critical interface. A critical interface is defined as one where an asphalt stabilized material lies above it or as one under which an untreated aggregate or subgrade layer lies. These critical interfaces are automatically determined within DAMA and it is not necessary for the user to define them.

3.8.1 Program Restrictions and Flexibility

In the development of the DAMA program, some restrictions were introduced to account for cost considerations. The major restrictions of DAMA (CHEV) are that it is limited to analyzing up to a five-layer pavement system (four pavement layers plus subgrade). In addition, a maximum of one granular layer may be used but its location must be directly above the subgrade. Finally, an asphalt stabilized layer must be the top pavement layer (i.e. the program cannot analyze a granular surfaced pavement structure).

Additional restrictions that apply to DAMA-CHEV is that only a single or dual wheel load system can be analyzed.

While DAMA-CHEV primarily uses a dual wheel load as input, a single wheel load analysis can easily be obtained by using a large dual spacing to simulate damage due to a single wheel.

On the other hand, the most important advantage of program DAMA lies in its flexibility to analyze pavement systems subjected to monthly variations in material properties relative to user-defined deformation and fatigue cracking distress criteria. Both fatigue cracking of each asphalt stabilized layer and the subgrade deformation are examined on the basis of monthly cumulative damage concepts. Damage computed on a monthly basis for a given monthly traffic repetition input value is then accumulated up to a damage of 100 percent for each distress mode. The design life and number of load repetitions to failure are summarized for cracking and deformation and the governing layer for the design situation is noted.

3.8.2 Material and Distress Input for Program DAMA

As mentioned previously, the major advantages of program DAMA lie in its capability to incorporate user-defined materials and distress relationships within a monthly cumulative damage scheme. For asphalt concrete layers, the user has the option of directly input laboratory moduli responses for various design frequencies and temperatures of a specific mix or utilize the regression equation (eq. 3.6) developed by the Asphalt Institute.

For asphalt emulsions exhibiting a curing effect with time upon modulus, an exponential cure law is used. Required input by the user involves noting the cure time, the time construction was completed, and the time the pavement was opened to traffic. This allows for the monthly damage computations to be conducted over a period of 12 months plus the time (in months) required by the emulsion layer to cure while open to traffic.

For obtaining granular layer modulus which is assumed to be stress dependent, the program DAMA uses an indirect equation developed by Witczak and Smith (13) to avoid the need for the highly repetitive trial and error iterative solutions. The equation take the form described earlier by eq. (3.12). Thus in using program DAMA, one need only define the monthly estimate of K_1 (to model variations in moisture or possible frost effects) as additional input for the granular layer evaluation.

As with the case with materials properties, program DAMA also provides for the maximum in user flexibility for incorporating different coefficients for both fatigue and subgrade deformation relationships.

Finally, a general description of the program DAMA input and output formats together with an illustrative example are shown in Appendix A.

3.9 Verification and Implementation of the Design Procedure

Attempts were made by the Asphalt Institute research staff to verify the use of the design procedure. To achieve that, a study was completed that compared the thicknesses predicted by use of the manual MS-1 to thicknesses obtained from actual in-service pavement sections. Over 400 individual pavement sections were included in the study.

From the results of the the study, the design concept is said to agree extremely well with observed pavement peformance.

However, although the above is a positive factor for the design approach, the design methodology appears to be more conservative for some situations than for others. For example, the present MS-1 design curves in their present form, will always favor the use of thick asphalt layers and minimal granular base or subbase layers, particularly for high-traffic situations (12).

Thus, although there appears to be a good potential for applying the design procedure on a routine design basis, yet a parametric evaluation of the various design input parameters is required before attempting to generalize the applicability of the design procedure to all geographic areas. Furthermore, until now there is no practical implementation of the design procedure reported anywhere in the literature that the author is aware of.

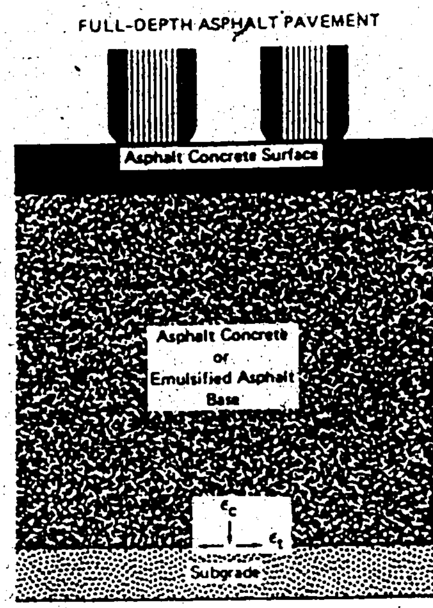
3.10 Summary

The review of the Asphalt Institute mechanistic method of pavement design has disclosed that:

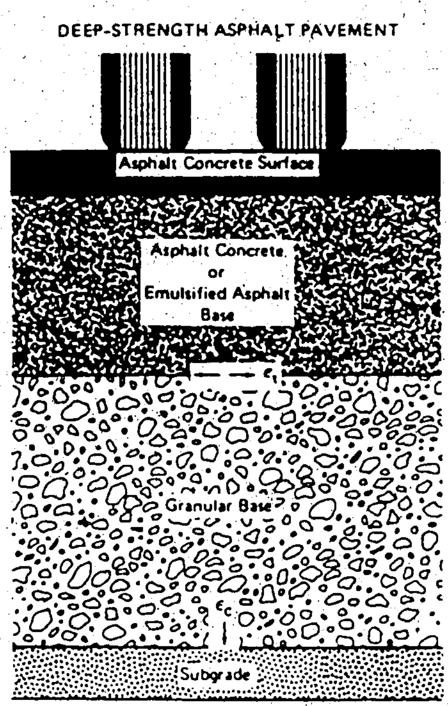
1. The inclusion of structural performance characteristics

in terms of controlling fatigue cracking and excessive permanent deformation facilitates the prevention of excessive physical distress and the retention of adequate riding quality for the design life of the pavement structure.

2. The utilization of available research results dealing with materials characterization enables the effective use of the different paving materials.
3. The use of mechanistic procedures along with field experience (empirical) has made it possible to evaluate more accurately the seasonal influence of temperature on the properties of asphalt concrete, and the loss of strength of unstabilized materials and subgrades during the spring thaw period.
4. The flexibility of the program DAMA to incorporate different user-defined material and distress relationships within a monthly cumulative damage scheme, gives an indication of the potential adapatability of the design methodology to other geographical regions.



(a) Full-Depth asphalt concrete and emulsified asphalt base pavements



(b) Pavements with granular base

Fig. 3.1 Pavement Model (Courtesy of the Asphalt Institute)

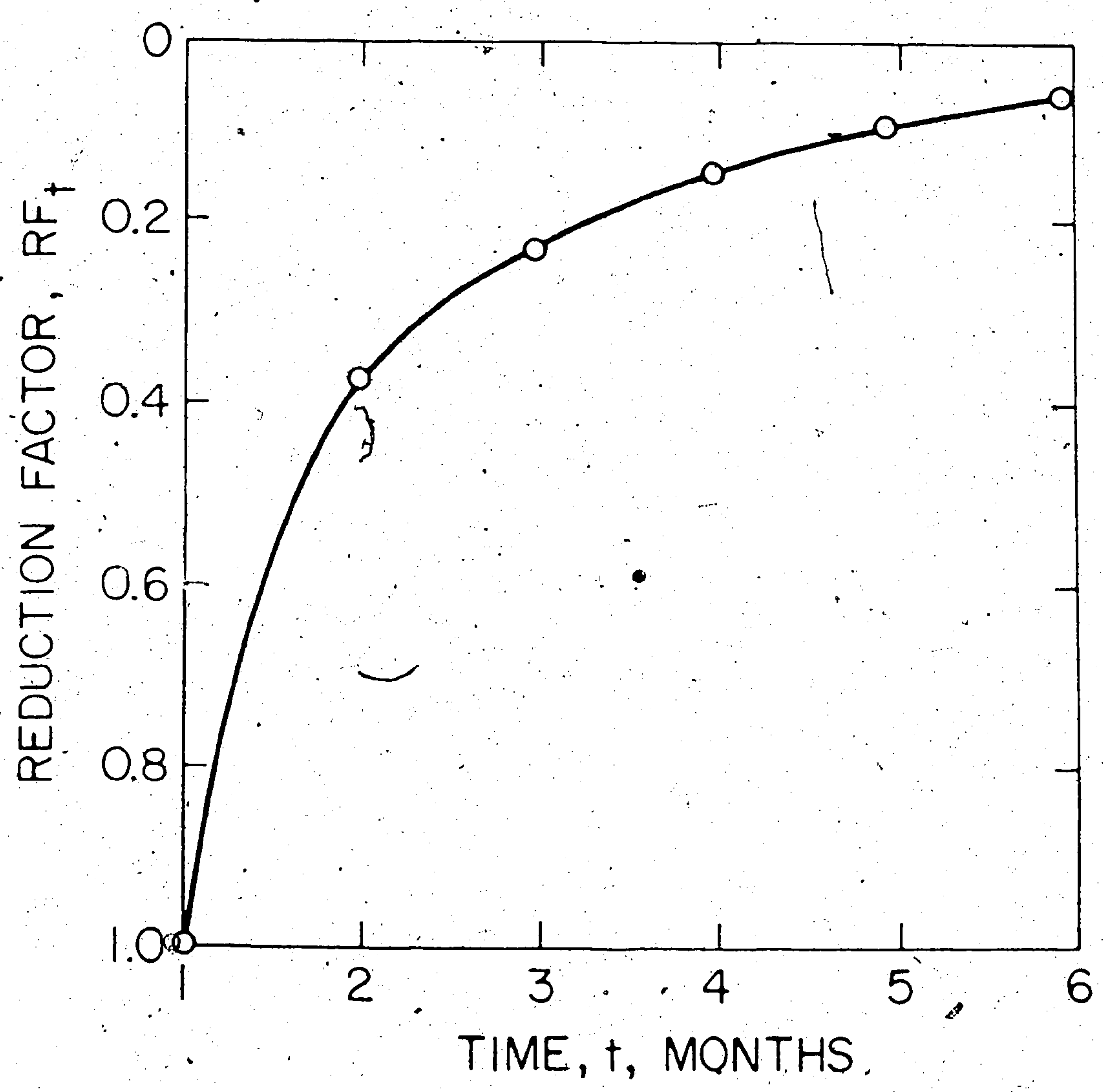


Fig. 3.2 Reduction factor vs time for 6 month cure period
(Courtesy of the Asphalt Institute)

Example: Determine Design Subgrade Resilient Modulus for the Conditions Given Below

Conditions:

1. Design Traffic ESAL = 10^4, 10^5 and 10^6

2. Results of 8 tests produced the following subgrade resilient moduli values:
40.5, 45.3, 59.0, 78.2, 80.0, 85.0, 90.3, 109.2 MPa

Procedure:

1. Arrange test values in descending order starting with the biggest value, i.e.
109.2, 90.3, 85.0, 80.0, 78.2, 59.0, 45.3, 40.5

2. Calculate percentile values equal to or greater than as follows:

Test Values*	# Equal To or Greater Than	% Equal To or Greater Than
109.2	1	(1/8)100=12.5
90.3	2	(2/8)100=25.0
85.0	3	(3/8)100=37.5
80.0	4	(4/8)100=50.0
78.2	5	(5/8)100=62.5
59.0	6	(6/8)100=75.0
45.3	7	(7/8)100=87.5
40.5	8	(8/8)100=100

3. Plot test values vs percent equal to or greater than.

4. From the plot read the subgrade resilient moduli values at 60%, 75% and 87.5% to represent the design subgrade strength values for the 10^4, 10^5 and 10^6 ESAL's respectively.

* hypothetical test values

Fig. 3.3 Determination of Design Subgrade Resilient Modulus from Test Results

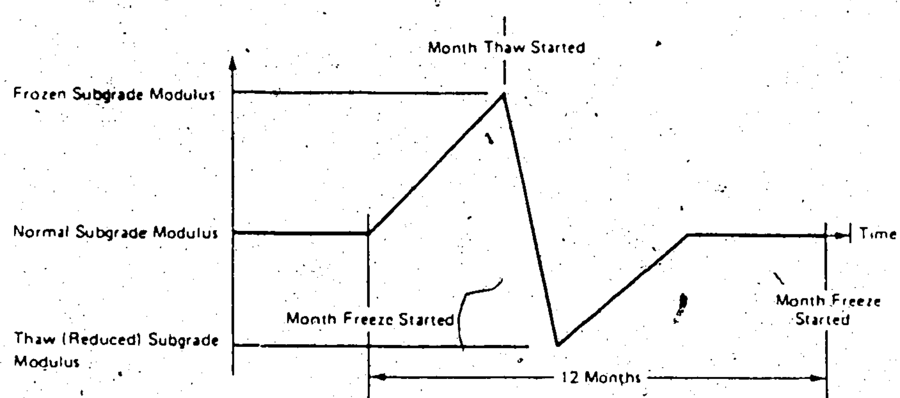


Fig. 3.4 Subgrade modulus variations throughout a typical year, for the conditions where freeze-thaw occurs (Courtesy of the Asphalt Institute)

Axle Load Group kN	(1000 lb)	No. of Axles Per 1000 Vehicles*	Load Equivalency Factor (Table IV-4)	EAL (Col. (2) X Col. (3))
(1)		(2)	(3)	(4)
		Single Axles		
<13.35	(<3)	604	0.0002	0.1
13.35- 31.15	(3- 7)	557	0.005	2.8
31.15- 35.6	(7- 8)	140	0.027	3.8
35.6 - 53.4	(8-12)	493	0.087	42.9
53.4 - 71.2	(12-16)	154	0.360	55.4
71.2 - 80.0	(16-18)	75	0.796	59.7
80.0 - 89.0	(18-20)	33	1.24	40.9
89.0 - 97.8	(20-22)	5	1.83	9.2
97.8 -106.8	(22-24)	2	2.58	5.2
106.8 -115.6	(24-26)	1	3.53	3.5
115.6 -133.5	(26-30)	0	5.39	5.4
		Tandem Axles		
<26.7	(<6)	22	0.010	0.2
26.7 - 53.4	(6-12)	227	0.010	2.3
53.4 - 80.0	(12-18)	162	0.036	5.8
80.0 -106.8	(18-24)	108	0.148	16.0
106.8 -133.5	(24-30)	140	0.426	59.6
133.5 -142.3	(30-32)	58	0.753	43.7
142.3 -151.2	(32-34)	25	0.971	24.3
151.2 -160.0	(34-36)	6	1.23	7.4
160.0 -169.0	(36-38)	3	1.53	4.6
169.0 -178.0	(38-40)	1	1.88	1.9
178.0 -187.0	(40-42)	1	2.29	2.3
187.0 -195.7	(42-44)	1	2.75	2.8
195.7 -204.5	(44-46)	1	3.27	3.3
204.5 -222.4	(46-50)	1	4.17	4.2
>222.4 (Use 226.8 (51))	(>50)	1	5.2	5.2
				Σ EAL = 412.5

Truck Factor = EAL/Vehicle = 412.5/1000 = 0.41

*Most U.S. truck weight and truck count data are reported in terms of 1000 vehicle-units. This convention is retained here for convenience in using state Truck Weight Study reports for traffic analysis.

Fig. 3.5 Truck factor computations for a typical U.S. interstate rural highway (Courtesy of the Asphalt Institute)

Table 3.1 Conditions used to represent frost effects on subgrade (S.I. units) (Courtesy of the Asphalt Institute)

MAAT °C	E_{ns} MPa	E_{fs} MPa	r_t %	E_{ts} MPa	T_{fo}	ΔT_f mo.	ΔT_{ct} mo.	ΔT_{rc} mo.	ΔT_n mo.
7	31.0	344.7	20	6.2	Dec	4	1	5	2
	82.7	344.7	50	41.4	Dec	4	1	5	2
	155.1	344.7	70	108.9	Dec	4	1	5	2
15.5	31.0	344.7	30	9.3	Jan	2m	1	4	5
	82.7	344.7	60	49.6	Jan	2	1	4	5
	155.1	344.7	80	124.1	Jan	2	1	4	5

Table 3.2 Conditions used to represent frost effects on subgrade (U.S. Customary Units) (Courtesy of the Asphalt Institute)

MAAT °F	E_{ns} psi	E_{fs} psi	r_t %	E_{ts} psi	T_{fo}	ΔT_f mo.	ΔT_{ct} mo.	ΔT_{rc} mo.	ΔT_n mo.
45	4,500	50,000	20	900	Dec	4	1	5	2
	12,000	50,000	50	6,000	Dec	4	1	5	2
	22,500	50,000	70	15,800	Dec	4	1	5	2
60	4,500	50,000	30	1,350	Jan	2	1	4	5
	12,000	50,000	60	7,200	Jan	2	1	4	5
	22,500	50,000	80	18,000	Jan	2	1	4	5

Table 3.3 Subgrade moduli used in DAMA program for MAAT of 7°C and 15.5 °C (S.I. units) (Courtesy of the Asphalt Institute)

MAAT	E_{ns}	Subgrade Modulus (by month) - MPa											
°C	MPa	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov
7	31.0	31	110	188	267	345	6	11	161	21	26	31	31
	82.7	83	148	214	279	345	41	50	58	66	74	83	83
	155.1	155	203	250	297	345	109	118	128	137	146	155	155
15.5	31.0	31	31	188	345	9	15	20	26	31	31	31	31
	82.7	83	83	214	345	50	58	66	74	83	83	83	83
	155.1	155	155	264	345	124	132	140	148	155	155	155	155

Table 3.4 Subgrade moduli used in DAMA program for MAAT of 45°F and 60°F (U.S. customary units) (Courtesy of the Asphalt Institute)

MAAT °F	E_{ns} psi	Subgrade Modulus (by month) - 10^3 psi Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov
45	4,500	4.5	15.9	27.3	38.7	50.0	0.9	1.62	2.34	3.06	3.78	4.5	4.5
	12,000	12.0	21.5	31.0	40.5	50.0	6.0	7.20	8.40	9.6	10.8	12.0	12.0
	22,500	22.5	29.4	36.3	43.1	50.0	15.8	17.1	18.5	19.8	21.2	22.5	22.5
60	4,500	4.5	4.5	27.3	50.0	1.35	2.14	2.93	3.71	4.5	4.5	4.5	4.5
	12,000	12.0	12.0	31.0	50.0	7.2	8.4	9.6	10.8	12.0	12.0	12.0	12.0
	22,500	22.5	22.5	38.3	50.0	18.0	19.1	20.3	21.4	22.5	22.5	22.5	22.5

Table 3.5 Monthly values for k_1 for granular base sections for MAAT of 45°F and 60°F (U.S. customary units) (Courtesy of the Asphalt Institute)

MAAT °F	k_1 (normal)	Monthly Value for k_1 (10^3) Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov
45	8,000	8.0	12.0	16.0	20.0	24.0	2.0	3.2	4.4	5.6	6.8	8.0	8.0
	12,000	12.0	18.0	24.0	30.0	36.0	3.0	4.8	6.6	8.4	10.2	12.0	12.0
60	8,000	8.0	8.0	16.0	24.0	2.0	3.5	5.0	6.5	8.0	8.0	8.0	8.0
	12,000	12.0	12.0	24.0	36.0	3.0	5.25	7.50	9.75	12.0	12.0	12.0	12.0

Table 3.6 Difference between DAMA (BISAR) and DAMA (CHEV) (Courtesy of the Asphalt Institute)

Variable	DAMA (BISAR)	DAMA (CHEV)
Number of Loads	Up to 10	Single or Dual Wheel
Types of Loads	Vertical and Horizontal	Vertical Load Only
Position of Damage Calculation	Up to Five Positions (User Selected)	Three Positions (Prefixed)
Interface Friction	Frictionless Slip to Full Friction	Full Friction Only
Executive Time*		
Full Depth AC	130 Sec.	40 Sec.
Granular Base	160 Sec.	45 Sec.
Full Depth AE	220 Sec.	55 Sec.

* Approximate CPU time by UNIVAC 1108 system at the University of Maryland.

References

1. The Asphalt Institute, "Thickness Design - Asphalt Pavement Structures for Highways and Streets", The Asphalt Institute, College Park, Maryland, Asphalt Institute Manual Series No. 1, 6th Edition, 1962.

2. The Asphalt Institute, "Thickness Design - Asphalt Pavement Structures for Highways and Streets", The Asphalt Institute, College Park, Maryland, Asphalt Institute Manual Series No. 1, 7th Edition, 1963.

3. F.N. Finn and J.F. Shook, "Thickness Design Relationship for Asphalt Pavements", Proceedings, First International Conference on the Structural Design of Asphalt Pavements, The University of Michigan, Ann Arbor, 1962, Vol. 1, pp. 52-83.

4. L.J. Painter, "Analysis of AASHO Road Test Data by the Asphalt Institute", Proceedings, First International Conference on the Structural Design of Asphalt Pavements, The University of Michigan, Ann Arbor, 1962, Vol. 1, pp. 84-97.

5. J.F. Shook, "Development of Asphalt Institute Thickness Design Relationships", Proceedings, Association of Asphalt Paving Technologists, Dallas, Texas, Feb. 17-19, 1964, Vol. 33, pp. 187-220.

6. The Asphalt Institute, "Thickness Design - Asphalt Pavements for Highways and Streets", The Asphalt Institute, College Park, Maryland, Asphalt Institute Manual Series No. 1, 9th Edition, 1981.

7. F.N. Finn, C.L. Saraf, R. Kulkarni, K. Nair, W. Smith and A. Abdullah, "The Use of Distress Prediction Subsystems for the Design of Pavement Structures", Proceedings, Fourth International Conference on the Structural Design of Asphalt Pavements, The University of Michigan, Ann Arbor, Aug. 22-26, 1977, Vol. 1, pp. 3-38.

8. C.L. Monismith, J.A. Epps, D.A. Kasianchuk, and D.B. Mclean, "Asphalt Mixture Behaviour in Repeated Flexure", Report No. TE 70-5, University of California, Berkeley, Jan. 1972.

9. P.S. Pell and K.E. Cooper, "The Effect of Testing and Mix Variables on the Fatigue Performance of Bituminous Materials", Proceedings, Association of Asphalt Paving Technologists, Phoenix, Arizona, Feb. 10-12, 1975, Vol. 44, pp. 1-37.

10. J. Epps, "Influence of Mixture Variables on the Flexural Fatigue and Tensile Properties of Asphalt Concrete", University of California, Berkeley, Ph.D. Dissertation, 1968.

11. L.E. Santucci, "Thickness Design Procedure for Asphalt and Emulsified Asphalt Mixes", Proceedings, Fourth International Conference on the Structural Design of Asphalt Pavements, The University of Michigan, Ann Arbor, Aug. 22-26, 1977, Vol. 1, pp. 424-456.

12. The Asphalt Institute, "Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition", The Asphalt Institute, College Park, Maryland, Asphalt Institute Research Report No. 82-2, Aug. 1982.

13. M.W. Witczak and B.E. Smith, "Prediction of Equivalent Granular Base Moduli Incorporating Stress Dependent Behaviour in Flexible Pavement", Transportation Journal, American Society of Civil Engineers, 1981.

14. A. Glaessen, J. Edwards, P. Sommer and P. Ugé, "Asphalt Pavement Design - The Shell Method", Proceedings, Fourth International Conference on the Structural Design of Asphalt Pavements, The University of Michigan, Ann Arbor, Aug. 22-26, 1977, Vol. 1, pp. 39-74.

15. B.F. Kallas and J.F. Shook, "San Diego County Experimental Base Project", Final Report, The Asphalt Institute, College Park, Maryland, Research Report 77-1 (RR 77-1), Nov. 1977.

16. M.W. Witczak, "Design of Full Depth Asphalt Airfield Pavements", Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, London, England, Aug. 7-11, 1972, Vol. 1, pp. 550-567.

17. Association of State Highway and Transportation Officials, "AASHO Interim Guide for Design of Pavement Structures - 1972", AASHO Manual Series, 1974.

18. The Asphalt Institute, "Computer Program DAMA - User's Manual", The Asphalt Institute, College Park, Maryland, Asphalt Institute Computer Program Series CP-1, Oct. 1983.

4.0 ALBERTA HIGHWAY 2:28 - A CASE STUDY

4.1 General

In this chapter a selected pavement section from a primary highway in central Alberta will be analysed using the computer program DAMA developed by the Asphalt Institute.

This part of the investigation is conducted primarily to evaluate the potential applicability of the Asphalt Institute design methodology to predict structural performance characteristics of conventional asphaltic pavements under Albertan conditions.

The specific pavement section selected for the study was chosen because of the availability of sufficient performance data. Besides the structural, geometric, and performance data usually available from the Alberta Transportation Pavement Information and Needs System (PINS) regarding any pavement section with the primary highway network in Alberta (1), detailed performance information pertaining to the specific pavement section selected is also available in the literature (2-5). This information will later provide for the opportunity of comparing predicted and measured structural performance characteristics.

4.2 The Existing Pavement

The control section 2:28 within the Primary Highway 2 is a four-lane divided pavement facility located in central

Alberta between highway 53 west of Ponoka and the junction with highway 13 at Brightview, a distance of 31.67 km. The cross section generally consisted of a 7.32 m wide asphalt concrete pavement (ACP) with 2.95 m of outside and 1.63 m inside ACP shoulder, shown in Figure 4.1. A 20 m wide depressed median separates the northbound and southbound directions. This section of the highway was constructed in the period of 1964-1966 and has the following existing pavement structure (4,5):

100 mm of 16 000 μm topsize ACP (1965-1966)

50 mm of 20 000 μm topsize asphalt stabilized base course (1964-1965)

50 mm of 20 000 μm topsize granular base course (1964-1965)

250 mm of 40 000 μm topsize granular base course (1964-1965)

During the construction of the 100 mm ACP surface layer, three 200-300 penetration grade asphalt cements representing major suppliers of asphalts in the province were used. These three asphalt sources represented high, low, and intermediate viscosity materials as measured at 60°C (140°F). A test location with uniform topographic relief was selected where all three sources would adjoin. At this location (km 9.01 north of the junction with highway 53), the low viscosity asphalt material occupied the northbound roadway while the other two sources adjoined at this point on the southbound roadway as shown in Fig. 4.2.

This highway carries heavy commercial and recreational traffic throughout the year. In 1981, section 2:28 of this highway was estimated to be carrying an Average Annual Daily Traffic (AADT) of 11 000 with 20 percent trucks.

Field condition surveys showed that severe rutting, in the order of 20-25 mm, existed within the boundaries of the roadway section where the low viscosity asphalt material was used. This necessitated the initiation of cold milling and recycling of a distance of 14.97 km of the outer northbound lane during the summer and fall of 1982 (4,5).

In the next sections, materials, environmental and traffic inputs pertaining to the roadway section will be reviewed and a subsequent structural analysis using the DAMA program of the Asphalt Institute will be conducted.

4.3 Materials and Environmental Inputs

In this section, the properties of the various pavement materials and the environmental considerations pertaining to the roadway section selected for the case study will be reviewed.

4.3.1 Materials Input

(a) Asphalt Concrete

Detailed information regarding the design and construction of the highway section 2:28 has been reported as a part of an earlier investigation directed towards evaluating the variability of the asphalts used and their subsequent behaviour in service (2). In this study an extensive testing program was conducted to determine the asphalt rheological properties. Table 4.1 contains a summary of the aggregate gradation together with some of the mix characteristics pertaining to the low viscosity material.

Changes in the asphalt properties and mix characteristics with time for all asphalt sources used in the construction of the highway section under consideration were reported by Shields et al. (2), and Anderson and Shields (3). The most important change reported that might significantly influence the pavement performance, is the considerable densification which occurred within the section where the low viscosity material had been used. This densification reduced the air voids content from an initial value of 8.3 percent to 1.7 percent, shown in Figure 4.3.

From the information contained in Table 4.1 and Fig. 4.3, the resilient modulus of the asphalt concrete layer was estimated. The procedure used for obtaining such modulus along with other information concerning the estimation of moduli of other materials, environmental, and traffic inputs used in the analysis are contained in Appendix B.

(b) Granular Base and Subgrade Materials

Prior to the commencement of paving, a detailed subsurface and base course testing was carried out to determine the properties of both the subgrade and base course layers. Table 4.2 summarizes the major properties pertaining to these two materials for the northbound roadway (2).

From the information contained in Table 4.2 and other information found in the literature (2), the subgrade can be classified as a CL soil consisting primarily of alluvium and medium plasticity tills, with the former predominating. Such subgrade was found to possess a medium to high frost susceptibility and a derived CBR value of 7. The estimation of the subgrade resilient modulus from correlations between such modulus and the CBR value is described in Appendix B.4.

Also from Table 4.2, the base course is mostly gravel which makes such material possess good drainage potentials and can thus be classified as a good quality aggregate material. A K_1 value for such material is also selected and enclosed in Appendix B.4.

4.3.2 Environmental Considerations

Since the roadway section selected for the analysis lies in an area where sub-zero air temperature conditions prevail for a long period throughout a typical year, considerations must be given to freezing and subsequent thawing of the subgrade soil and the influence of that on subgrade modulus.

Fig. 4.4 shows selected temperatures within and around the pavement structure for the period 1966-1968 as reported by Shields et al. (2). Using the information contained in this plot, the mean monthly air temperatures characterizing the region were determined and used as input for program DAMA to reflect the influence of pavement temperature on the dynamic modulus of asphalt concrete. Also, the month at which freezing of the subgrade started and the durations of the various freezing and thawing periods were determined and used in the analysis by assigning the appropriate subgrade modulus value pertaining to each month.

4.4 Traffic Analysis

In this section, a review of the procedures employed by Alberta Transportation for estimating the traffic anticipated to use the pavement facility during its design life will be presented.

In addition, the specific traffic input characteristics pertaining to the highway control section 2:28 will be summarized.

4.4.1 Alberta Transportation Procedure for Estimating ESAL's

The traffic analysis procedure used in the province involves the conversion of all mixed traffic into equivalent number of an 80 kN single-axle load applications (ESAL's). This procedure can be outlined as follows (6,7,8):

(1) Traffic volumes (expressed as AADT) are first generated with information available from the 98 permanent weigh-scale sites distributed throughout the province. This is further supplemented by traffic data obtained

annually from approximately 444 short term portable machine counts and about 599 annual coverage counts.

(2) Adjustments are then made to the AADT volume to reflect both the relative proportion of the trucks to be expected in the design lane and the anticipated future traffic growth.

(3) The percentage truck traffic thus obtained is then broken up into single unit trucks (SUT) and tractor trailer combinations (TTC).

(4) ESAL is then calculated by multiplying the number of vehicles in each truck class by the appropriate truck factor (TF) and obtaining the sum of the products,

$$\text{ESAL} = \Sigma\ (\text{No. of vehicles in each truck class} \times \text{TF}) \qquad (4.1)$$

Truck factors are determined from axle-weight data using load equivalency factors derived from the AASHO equation. Calculating a truck factor is usually done by multiplying the number of axles in each truck class by the appropriate load equivalency factor and dividing the sum of the products by the total number of vehicles involved. Truck factors of 0.56 and 1.37 are currently in use for respectively converting single unit trucks, and tractor trailer combinations into ESAL's.

A major limitation of the above described procedure, involves the uncertainties usually encountered in estimating the percentage truck traffic expected to use the pavement facility. However, much of the risk can be eliminated by applying appropriate safety factors to the calculated ESAL's.

4.4.2 Roadway Section Traffic Input

Following the traffic analysis procedure described above, the Traffic Information Services Unit of Alberta Transportation publishes an annual statistics report summarizing the ESAL's distribution within the primary and other highways in the province. Using the information contained in such reports (7,8,9), the daily ESAL's pertaining to the highway control section 2:28 were converted to average ESAL's per month per design lane. Because the ESAL's distribution varied throughout the in-service years of the pavement section, different average ESAL's per month per design lane were used in the analysis. The following values are the ones used to approximate the actual ESAL's distribution measured throughout the 16 years the pavement spent in service before the major rehabilitation took place in 1982.

Year	Average ESAL/Month/Design Lane
1966 (Year Hwy opened to traffic)	5400
1974 (8 years after construction)	6800
1982	12500

(Year Hwy recycled)

Other traffic characteristics include an average tire pressure of 80 psi for the first 8 years after construction, and 95 psi for the next 8 years and until 1982. These values were selected based on information obtained from Alberta Research Council (6).

4.5 Analysis of the Pavement Section Using Computer Program DAMA

A summary of the design input data related to the pavement section under consideration together with the results obtained from the analysis will be presented here.

4.5.1 Pavement Section Design Inputs

In the course of the analysis, the asphalt bound base layer was assumed to be acting as an additional 50 mm of granular base layer. This layer was stabilized using an MC-250 or MC-800 cutback and had an average residual asphalt content of 4.4% based on tests prior to construction in 1966 (2). Since this layer could not be considered as an asphalt concrete layer subjected to fatigue damage, it was included in the thickness of the granular base layer. Thus the pavement section analyzed was viewed as being composed of a 100 mm-thick asphalt concrete layer underlain by a 350 mm-thick granular base layer.

Also and as pointed out earlier, all materials, environmental, and traffic inputs used for the analysis are based on information contained in Tables 4.1 and 4.2 and Figs. 4.3 and 4.4, and are shown in Appendix B.

Furthermore, it was considered essential to incorporate different air voids volumes in the analysis to take into account the rapid densification effects that were observed to occur during the early life of the pavement section as shown in Fig. 4.3.

4.5.2 Results of the Analysis

Seven runs were made using the computer program DAMA to analyze the pavement section. In these runs, only three input parameters were varied, namely, the average number of ESAL's per month, the percent air voids content, and the tire pressure exerted by the dual wheel load system on the pavement structure. Variations in these parameters were considered to account for the rapid densification that was observed to occur during the early life of the pavement facility, and the postulated annual increase in tire inflation pressures.

In all runs, different subgrade moduli values were assigned to each month to account for the variations in subgrade strength brought about by freezing and thawing effects. A similar approach was also adopted for the aggregate base material to reflect the influence of sub-zero air temperature on the base layer drainage characteristics.

Tables 4.3(a) to 4.3(g) contain a summary of the analyses results in terms of the percent cumulative damage induced in the pavement structure during its design life. Tables 4.3(a) through 4.3(d) represent the damage accumulated within each 6 month time for a period of two years after the road had been opened to traffic while Table 4.3(e) represents the damage accumulated during the third year. The damages summarized in Tables 4.3(f) and 4.3(g) represent the typical yearly damage values used for obtaining the cumulative damage for years 4 to 8 and years 9 to 16 respectively.

Using the procedure discussed above, the analysis yielded 19 years of service life before the pavement section was considered to have failed due to excessive rutting. The computation of the service life is depicted in Fig. 4.5.

4.5.3 Discussion of the Results

Analysis of the results contained in Tables 4.3(a) through 4.3(g) indicates that the pavement is calculated to fail in about 19 years of service due to the manifestation of excessive permanent deformation resulting from overstressing the subgrade layer.

It should also be noted that most of the damage induced occurs during the spring time when the subgrade strength is drastically reduced due to thawing effects. On the other hand, negligible damage accumulates during the winter which is indicative of the beneficial effects brought about by the freezing of the subgrade.

Another conclusion which can be drawn from the analysis is that the fatigue cracking mode of failure appears to represent no problem since more than 11 x 10^6 ESAL's are required to accumulate before serious cracking starts to develop.

4.6 Comparison of Observed and Predicted Performance

During the spring of 1982, the Civil Engineering Department of the Alberta Research Council conducted a condition survey for the northbound direction of highway 2:28 from km 2.4 to km 14.4 and is presented in Table 4.4. This survey as well as others performed by Alberta Transportation (5) recorded excessive permanent deformation averaging 20-25 mm and up to 40 mm to occur within the outer and inner wheel paths of the outer lane. Fig. 4.6 shows this condition prior to the rehabilitation made in 1982 by cold milling and recycling the outer lane followed by overlaying the entire cross-section. Fig. 4.7 shows the cross-section after it has been overlaid.

Comparing the observed and predicted performance of the roadway section, program DAMA appears to reasonably predict the pavement service life. From the results of the analysis, the predicted service life of 19 years is only about 3 years longer than the actual observed life of that pavement section. This is not unusual, particularly in view of the fact that precise estimates of the number of ESAL's per month and the subgrade resilient modulus are not available. This might have further been substantiated by the fact that program DAMA predicts only the amount of permanent deformation resulting from overstressing the subgrade without regard to the additional amount of rutting resulting from the densification of the asphalt surface layer. This densification could amount to an excess of 6 mm for the 100 mm asphalt concrete layer resulting from the reduction in air voids content from 8.3 percent to below 2.2 percent as shown in Fig. 4.3.

4.7 Summary of Case Study

The findings from this study can be summarized as follows:

1. The Asphalt Institute method appears to have potential in reasonably predicting the performance of both in-service and newly constructed asphalt concrete pavements under Albertan conditions provided that appropriate design inputs are available.

2. The calculated design life is found to be 19 years as compared to an actual observed life of 16 years. This discrepancy between the mechanistic evaluation and actual behaviour was believed to be brought about by:
   (a) difficulties encountered in precisely estimating the average ESAL's per month for the design period;
   (b) the additional rutting, which is not accounted for in

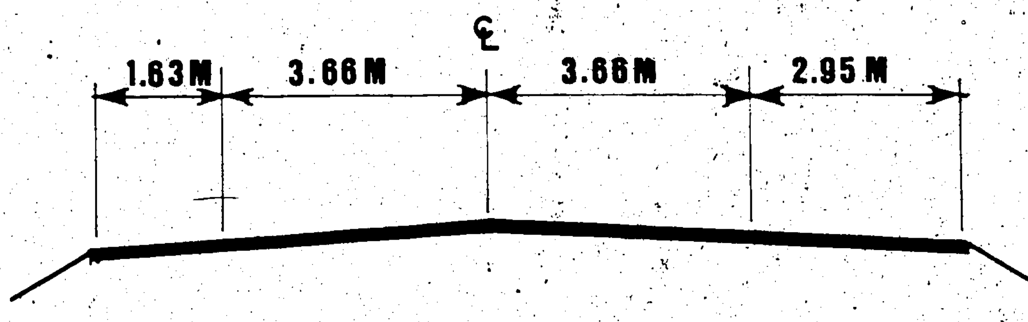
the analysis, that might have occurred due to the consolidation and shear of the upper asphalt concrete layer;

(c) imprecise estimation of some of the materials' properties, especially the resilience characteristics of the subgrade layer; and

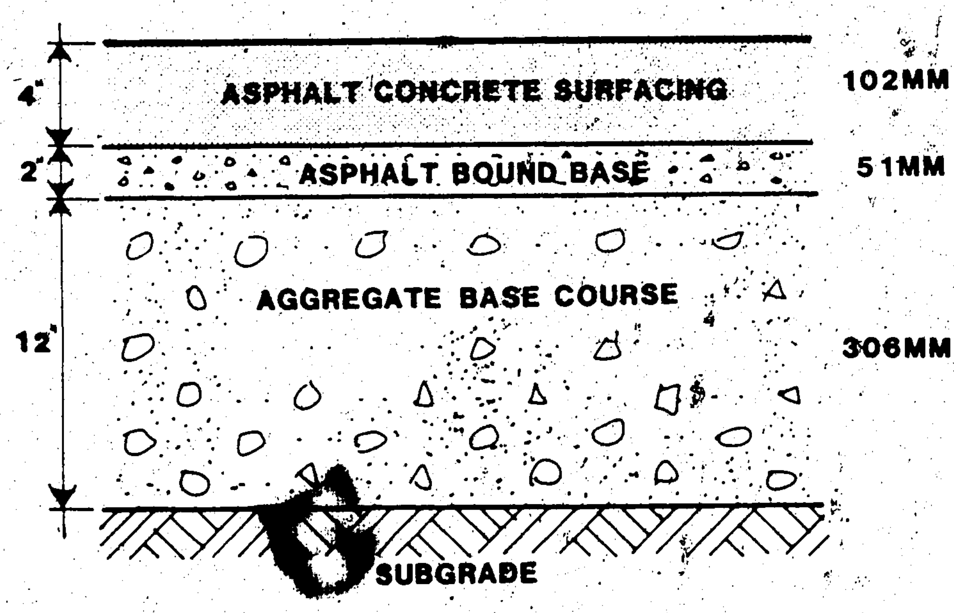
(d) changes in materials' properties that might have occurred due to aging of the asphalt concrete layer and discontinuity brought about by excessive low temperature cracking.

3. Excessive permanent deformation in the subgrade layer is found to be the distress mode controlling the pavement design life. This agrees reasonably well with the observed performance of the roadway section.

4. The rapid densification which occurred within the asphaltic concrete layer during the early life of the pavement reduced the initial air voids content and provided for a fatigue life of more than 75 years.



(a) Geometry of the roadway section



(b) Pavement cross-section

Fig. 4.1 Geometric and cross-sectional layouts of the roadway section

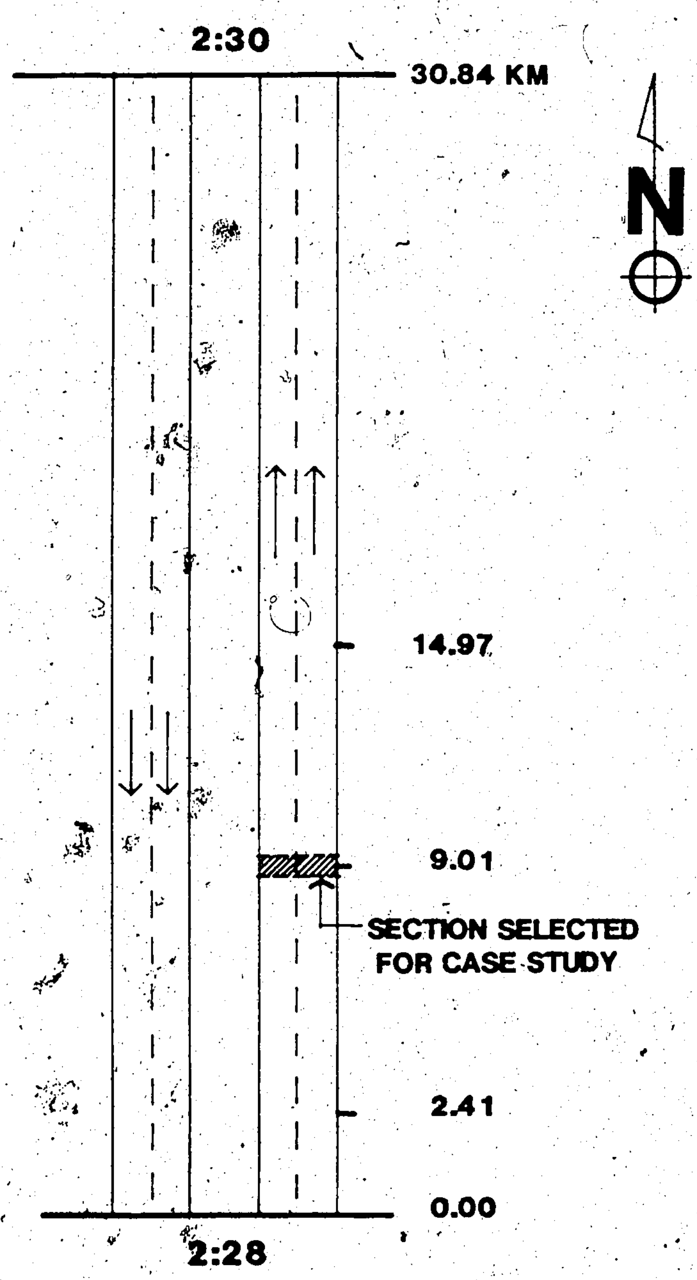


Fig. 4.2 Plan of Highway 2:28 from km 0.00 to km 30.84 north of Ponoka

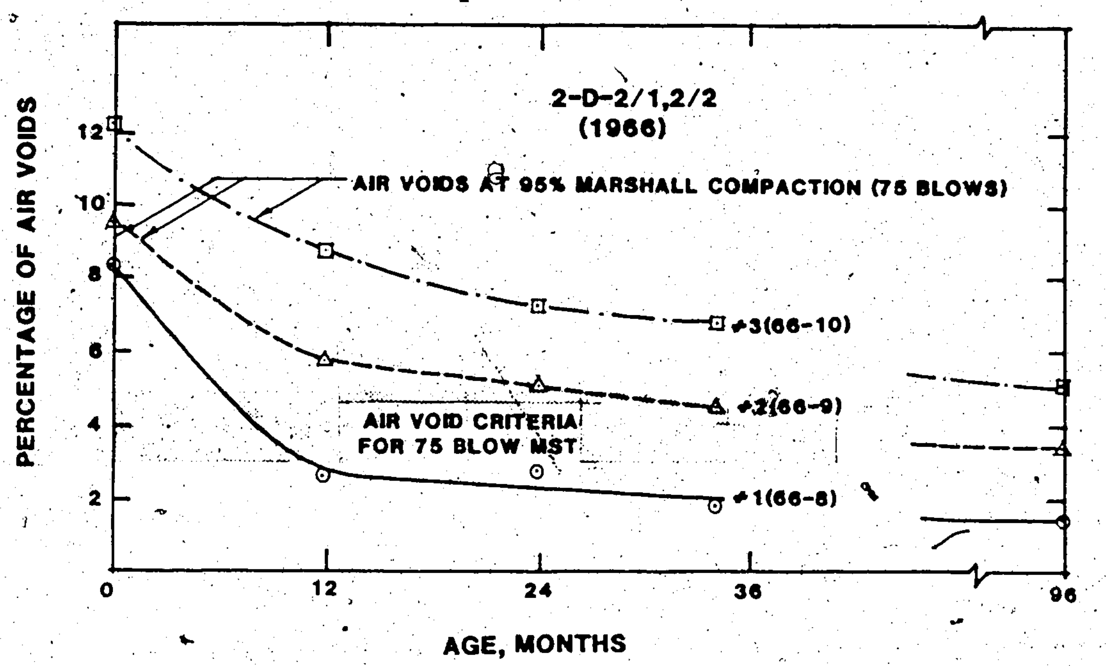


Fig. 4.3 Air void changes with time for 200-300 penetration grade asphalt cement

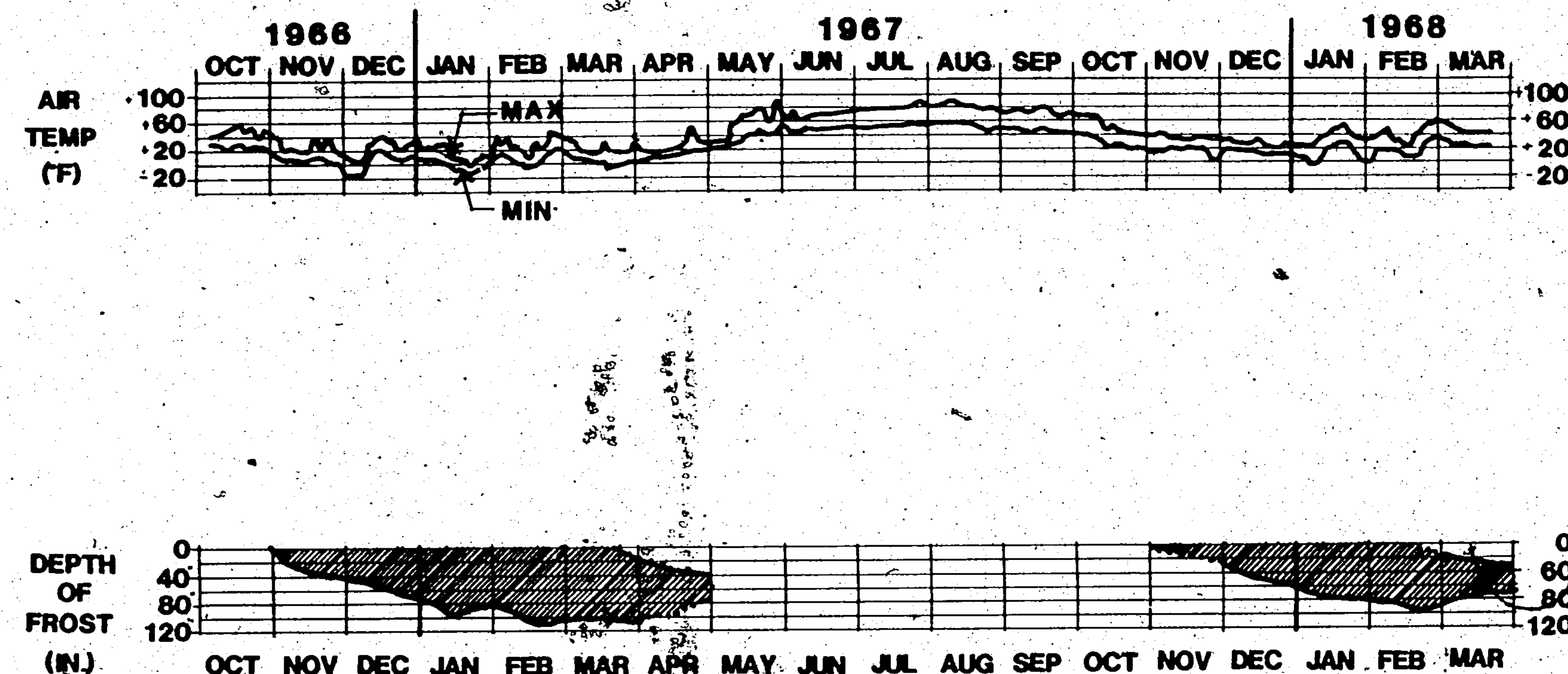


Fig. 4.4 Mean monthly air temperature profile and frost penetration depths measured during the period 1966-1968 at roadway section 2:28

Period	Fatigue Damage (%)	Permanent Deformation Damage (%)
1st year	2.944+2.703=5.647	1.141+2.317=3.458
2nd year	0.205+0.699=0.904	0.645+3.120=3.765
3rd year	1x0.761=0.761	1x3.575=3.575
4th-8th year	5x0.621=3.105	5x3.502=17.51
9th-16th year	8x1.263=10.104	8x6.466=51.728
Total damage at the end of 16 years	20.521	80.036
Percent damage remaining	79.479	19.964

- Extra years needed to reach 100% fatigue damage = $\frac{79.479}{1.263}$ = 62.9 years
- Extra years needed to reach 100% permanent deformation damage = $\frac{19.964}{6.466}$ = 3.08 years
- Pavement service life based on fatigue = 78 years
- Pavement service life based on permanent deformation = 19 years

Fig. 4.5 Computation of pavement service life

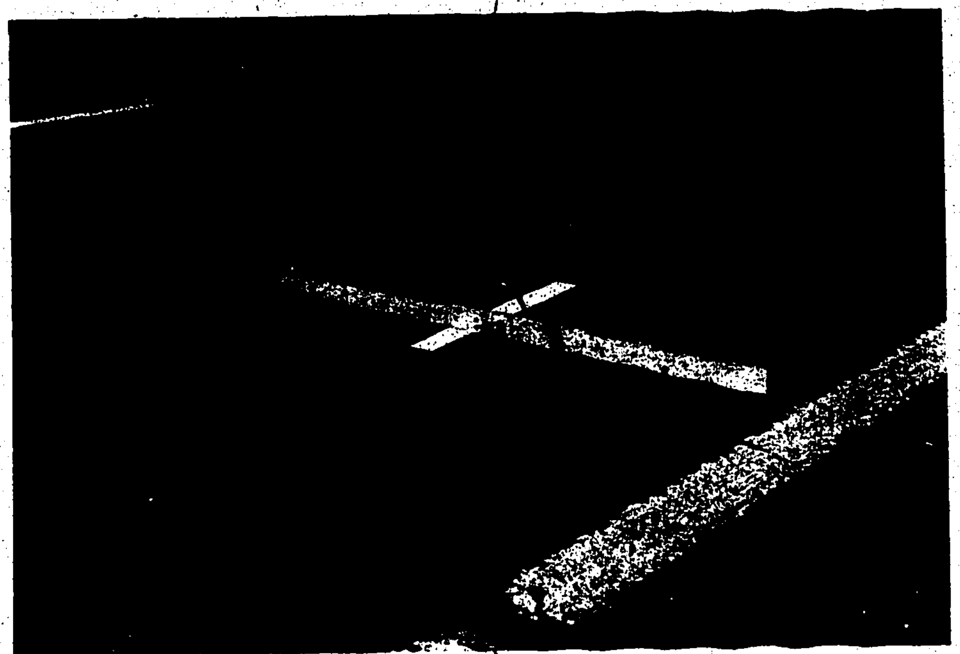


Fig. 4.6 Permanent deformation within the outer and inner wheel paths of the outer lane - Highway section 2:28

Fig. 4.7 Overlaid roadway section - Highway 2:28

Table 4.1(a) Summary of the gradation and mix characteristics of the asphalt concrete

Gradation		Mix Characteristics		
Sieve Size µm	Avg. % Passing			
20,000	100	Asphalt content*	%	5.4
5,000	50	Density	kg/m^3	2314
1,250	34	Stability	N	5916
315	14	Air voids	%	3.9
80	6.3	V.M.A.	%	14.1
		Voids filled with asphalt	%	72.5
		Flow	mm	2.1

* Asphalt absolute viscosity @ 21°C, poises x 10^6 = 0.60

Table 4.1(b) Summary of asphalt concrete properties before and after construction

Condition	Bitumen Content %	Air Voids %
As Designed	5.4	3.9
As Constructed	5.2	8.3

Table 4.2 Summary of subgrade and base course properties for highway 2:28, northbound km 2.4-14.4 (1966)

Description	Sieve Analysis (sieve sizes in µm)						Atterberg Limits		P.I.
	40000	20000	5000	1250	315	80	L.L.	P.L.	
Subgrade Layer	100	100	98.5	97.6	92.0	56.1	31.3	14.8	16.5
Granular Base	100	79.1	46.2	35.9	16.0	7.7	20.8	15.6	5.2

Table 4.3(a) Monthly damage for the period October 1965 - March 1966

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1965	Oct.	0.806	0.159
	Nov.	0.213	0.010
	Dec.	0.111	0.002
1966	Jan.	0.066	0.001
	Feb.	0.047	0.000
	March	1.701	0.969
6 mo. period cumulative damage		2.944	1.141

* ESAL/month = 5400 ; vv = 8.3% ; Tire pressure = 80 psi

Table 4.3(b) Monthly damage for the period April 1966 - September 1966

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1966	April	0.392	0.592
	May	0.839	0.011
	June	0.494	0.771
	July	0.265	0.367
	Aug.	0.362	0.318
	Sept.	0.351	0.258
6 mo. period cumulative damage		2.703	2.317

* ESAL/month = 6800 ; vv = 5.0% ; Tire pressure = 80 psi

Table 4.3(c) Monthly damage for the period October 1966 - March 1967

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1966	Oct.	0.069	0.141
	Nov.	0.017	0.009
	Dec.	0.009	0.002
1967	Jan.	0.006	0.001
	Feb.	0.004	0.000
	March	0.100	0.492
6 mo. period cumulative damage		0.205	0.645

* ESAL/month = 6[illegible]00 ; vv = 2.8% ; Tire pressure = 80 psi

Table 4.3(d) Monthly damage for the period April 1967-September 1967

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1967	April	0.081	0.531
	May	0.200	0.965
	June	0.147	0.738
	July	0.083	0.354
	Aug.	0.100	0.299
	Sept.	0.088	0.233
6 mo. period cumulative damage		0.699	3.120

* ESAL/month = 6800 ; vv = 2.5% ; Tire pressure = 80 psi

Table 4.3(e) Monthly damage induced during the third year after construction

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1967	Oct.	0.050	0.136
	Nov.	0.012	0.008
	Dec.	0.007	0.002
1968	Jan.	0.004	0.001
	Feb.	0.003	0.000
	March	0.069	0.450
	April	0.070	0.410
	May	0.175	0.953
	June	0.131	0.735
	July	0.074	0.352
	Aug.	0.088	0.297
	Sept.	0.078	0.231
12 mo. period cumulative damage		0.0761	3.575

* ESAL/month = 6800 ; vv = 2.3% ; Tire pressure = 80 psi

Table 4.3(f) Monthly damage induced during the fourth year after construction

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1968	Oct.	0.040	0.132
	Nov.	0.010	0.008
	Dec.	0.005	0.002
1969	Jan.	0.003	0.001
	Feb.	0.003	0.000
	March	0.055	0.427
	April	0.056	0.393
	May	0.142	0.935
	June	0.109	0.730
	July	0.062	0.351
	Aug.	0.073	0.295
	Sept.	0.063	0.228
12 mo. period cumulative damage		0.621	3.502

* ESAL/month = 6800 ; vv = 2.0% ; Tire pressure = 80 psi

Table 4.3(g) Monthly damage induced during the ninth year after construction

Year	Month	Max. Monthly Damage (percent)* Fatigue	Rutting
1974	Oct.	0.076	0.245
	Nov.	0.019	0.015
	Dec.	0.010	0.004
1975	Jan.	0.006	0.001
	Feb.	0.005	0.001
	March	0.095	0.765
	April	0.099	0.710
	May	0.275	1.727
	June	0.249	1.367
	July	0.145	0.657
	Aug.	0.156	0.550
	Sept.	0.128	0.424
12 mo. period cumulative damage		1.263	6.466

* ESAL/month = 12500 ; vv = 1.8% ; Tire pressure = 95 psi

Table 4.4: Summary of rut depths for highway 2:28, northbound from km 2.4 to km 14.4 (From Alberta Research Council Survey data on May 2nd, 1982)

	Inner Lane		Outer Lane	
km to km	OWP	IWP	OWP	IWP
2.4-4[illegible]	7.4 (0.8)*	7.0 (0.9)	19.7 (4.1)	20.4 (3.5)
4.4-6.4	7.2 (0.7)	8.0 (1.4)	20.2 (2.6)	20.9 (2.3)
6.4-8.4	7.5 (1.3)	7.9 (1.0)	18.5 (1.6)	21.8 (2.7)
8.4-10.4	7.4 (1.0)	8.4 (1.3)	20.5 (3.2)	23.1 (4.4)
10.4-12.4	8.1 (2.0)	7.0 (1.6)	20.5 (3.8)	23.2 (4.8)
12.4-14.4	6.6 (1.0)	6.7 (1.3)	17.9 (4.1)	21.2 (3.5)

* One standard deviation
Measurements at 0.16 km frequency, all w/ps with 6 ft long rod.

J.T. Christison
Civil Engineering Dept.
Alberta Research Council

References

1. M.A. Karan, T.J. Christison, A. Cheetham, and G. Berdahl, "Development and Implementation of Alberta's Pavement Information and Needs System", TRB Research Record 938, 1983, pp. 11-20.

2. B.P. Shields, K.O. Anderson and J.M. Dacyszyn, "An Investigation of Low Temperature Transverse Cracking of Flexible Pavements", Proceedings of the 1969 Convention, CGRA, Edmonton, Sept. 29-Oct. 2, 1969, pp. 273-308.

3. K.O. Anderson and B.P. Shields, "Rheological Changes of Asphalt Cements in Service in Alberta", Proceedings, Twentieth Annual Conference, CTAA, Toronto, Nov. 1975, pp. 389-409.

4. K.O. Anderson and D.P. Palsat, "Recycling of Asphalt Pavements in Alberta", Proceedings, CTAA, Edmonton, Alberta, Nov. 1982, Vol. 27, pp. 188-206.

5. K. Hadipour, "Recycling of Asphalt Concrete Pavements", University of Alberta, Edmonton, M.Sc. Thesis, 1983.

6. J.T. Christison, Alberta Research Council, Private Communications, June 1987.

7. Alberta Transportation, "1985 Traffic Volume on Primary, Secondary and Other Roads", Report 1, Alberta Transportation, Edmonton, April 1986.

8. Alberta Transportation, "1985 Traffic Volume and Classification Breakdown on Primary, Secondary and Other Roads", Report 2, Alberta Transportation, Edmonton, April 1986.

9. Alberta Transportation, "1985 18-Kip ESAL Statistics - Primaries", Alberta Transportation, Edmonton, Aug. 15, 1986.

5.0 PARAMETRIC STUDY OF FACTORS AFFECTING PAVEMENT LIFE

5.1 Introduction

Among the several available mechanistically-based design methods, the Asphalt Institute thickness design procedure (1,2) was chosen to accomplish the objectives of this study. In this design procedure, the pavement is considered as a multi-layered elastic system with the materials in each of these layers being characterized by a modulus of elasticity, E, and a Poisson's ratio, ν. Loads on the surface of the pavement are considered responsible for producing two strains which are believed critical for design purposes. A computer program, DAMA, was used to relate these mechanistic responses to the amount of cumulative damage induced within the pavement structure in the form of fatigue cracking and/or excessive permanent deformation (2,3).

The investigation was conducted to achieve three objectives. First, to identify the significant factors that influence the design thickness requirements. Second, to quantify the individual and/or interactive effects of such factors on the pavement design life. And third, to provide guidelines for future implementation of the design procedure.

Since program DAMA analyzes a given pavement cross-section structure and thus not a design program per se, a pavement test section of known performance history, located in central Alberta, was selected for the purposes of the study (4,5,6). This test section consisted of a 102 mm-thick asphalt concrete layer followed by a 51 mm-thick asphalt bound layer and a 306 mm-thick aggregate base layer resting on an alluvium subgrade with medium plasticity.

The study is comprised of two phases. In the first phase, a screening analysis using statistical techniques (7) was used to identify and isolate the factors that are considered to significantly affect pavement design life. The second phase consisted of conducting a sensitivity analysis for the significant factors to enable quantifying their individual and interactive influences on the design life.

In the following sections, the details and findings from both phases of the investigation will be described.

5.2 Major Factors Influencing Design Thickness Requirements

This section describes the screening analysis performed, using program DAMA, to isolate the significant factors that affect the design thickness requirements.

5.2.1 Screening Analysis

Among the very many input design parameters that program DAMA utilizes in calculating the design life of a certain pavement structure, the following are the ones that have been selected for inclusion in the screening analysis:

1. Tire pressure of the wheel load system (TP).

2. Percent volume of bitumen in the asphalt concrete layer (VB).
3. Percent volume of air voids in the asphalt concrete layer (VV).
4. Expected frequency of traffic load application in Hz (f).
5. Type of bitumen used expressed as the asphalt absolute viscosity at 21°C ($\eta_{70°F, 10^6 \text{ poises}}$).
6. Mean Annual Air Temperature of the geographical region in which the pavement structure is analyzed (MAAT).
7. Quality of aggregate materials used in the base and subbase layers of the pavement system (K_1).
8. Resilient modulus of the subgrade layer (E_{SG}).

These factors were selected because of the many available reseach results which indicate their probable potential influences on pavement design life.

In order to isolate the most significant factors that affect the pavement design life, a 2-level fractional factorial design was performed on the above mentioned factors. The levels chosen for each factor were selected as follows:

Factor	Level 1	Level 2	
TP	70	120	(psi)
VB	9	12	(%)
VV	3	10	(%)
f	1	16	(Hz)

$\eta_{70}, 10^6$ poises	0.5	5.0	($x10^6$ poises)
MAAT	7	29	(°C)
K_1	8000	12000	(-)
E_{SG}	4500	22500	(psi)

The details of the analysis are explained in Appendix C.

5.2.2 Findings

The fractional factorial statistical technique used in the screening analysis has disclosed the following significant factors:

1. Mean Annual Air Temperature (MAAT).
2. Subgrade resilient modulus (E_{SG}).
3. Volume of air voids, % (VV).
4. Volume of bitumen, % (VB).
5. Tire pressure (TP).
6. Quality of aggregate material to be used in base or subbase layers (K_1).

On the other hand, changing the asphalt type and/or frequency of load application has shown little or no effect on the pavement design life.

5.3 Sensitivity Analysis

Having isolated the significant input parameters that influence the estimation of the pavement design life, the next step involved conducting a sensitivity analysis for those factors to quantify their individual and/or interactive effects on the design life.

Only four out of the six significant factors evolved from the screening analysis were selected for inclusion in the sensitivity analysis. These are: (a) tire pressure, (b) volume of air voids; (c) subgrade resilient modulus; and (d) the mean annual air temperature of the climactic region in which the pavement facility was situated. The other two factors, namely, volume of bitumen used in the asphalt concrete layer and the K_1 factor, were excluded from the analysis because of two reasons. First, to decrease the number of computer runs required for the analysis. And second, to account for other constraints imposed by mix and other design requirements on these factors. In other words these later two factors are usually controlled by proper mix design procedures which are not expected to vary largely in practice.

The sensitivity analysis described herein consists of determining the change in the dependent variable (pavement design life, DL) resulting from a unit change in an independent design parameter (TP, VV, E_{SG}, or MAAT).

The change in DL resulting from an assumed error in the estimation of any of the three design parameters considered i.e. TP, VV and E_{SG}, were computed at three levels for each parameter and at three levels of error magnitude. The different levels of the design parameters selected were based on practical considerations. The tire pressure levels selected were meant to include the current ranges within which such parameter fall. The volume of air voids levels were chosen to reflect the degree of quality control maintained during construction. For example, a low volume of air voids indicates a proper degree of compaction being achieved, whereas a high level indicates a poor compaction. The subgrade resilient moduli values selected indicate poor, medium, and good support conditions as represented by the 4500 psi, 12000 psi, and 22500 psi levels respectively. The magnitudes of each parameter considered are given in factorial form in Tables 5.1 and 5.2 for the two climatological regions considered. The three levels of error selected for each of the three design parameters are shown in Table 5.3.

Using the program DAMA, the design life of the selected pavement section at each combination of design parameters given in Tables 5.1 and 5.2 was obtained. These data were then plotted on graphs to show the resultant change in DL caused by an induced error in each parameter. The percent change in the design life was computed as follows:

$$E_i = \frac{[DL_{(j,i\pm\Delta)} - DL_j]}{DL_j} \qquad (5.1)$$

where,

E_i = percent change in the design life (DL) due to a variation in the design parameter i;

DL_j = design life for the factorial; j indicates block of factorial listed in Tables 5.1 and

5.2; and

$DL_{(j,i\pm\Delta)}$ = design life for factorial block j with an assumed error $\pm\Delta$ in parameter i.

A positive value of E_i indicates a resultant increase in the pavement design life, whereas a negative value indicates a decrease.

5.4 Presentation of Results

The percentage change in DL was calculated for each of the 27 factorial cells at the indicated errors in tire pressure (TP), volume of air voids (VV), and subgrade resilient modulus (E_{SG}). This was accomplished for the two climatic regions considered i.e. MAAT of 7°C and 29°C. In each cell of the two factorials, 19 independent solutions were made, one at the actual value plus six each at the plus and minus variations about the three parameters, for a total of 1026 solutions.

The results of the sensitivity analysis are presented in tabular and graphical forms. In the tabular form the percent change in DL resulting from induced variations in any of the design parameters is shown in Tables 5.4 to 5.21. It should be noted that a positive value indicates an increase in the DL, whereas a minus sign (-) indicates a decrease.

The graphical presentation is another form of illustrating the percent increase or decrease in DL as a function of variations in the design parameters. This is shown in Figs. 5.1 to 5.18. In these figures, a plus sign between the brackets indicates a positive variation in the design parameter, whereas a minus sign indicates a negative variation.

A detailed discussion of the effect of each of the design parameters on DL will now follow.

5.4.1 Cold Climatic Region (MAAT=7°C)

The percentage increase (or decrease) in DL resulting from subsequent changes in the design parameters when the pavement structure was placed in a cold environment, will be presented here.

5.4.1.1 Effect of Changing Tire Pressure

The percentage change in DL resulting from variations in the tire pressure (TP) is shown in Figs. 5.1 through 5.3 and Tables 5.4 to 5.6. The results show that an increase in the tire pressure will result in a decrease in the design life and vice versa. For example, if a pavement were designed for a TP of 70 psi (with E_{SG} = 12000 and VV = 5%) and a subsequent check indicated TP to equal 90 psi (an error in TP of 20 psi), the reduction in DL would be about 20 percent. This is not surprising since ~~on one hand~~, increasing the tire pressure will tend to increase the amount and rate of propagation of fatigue cracking in the asphalt concrete layers and hence reduce the design life of the pavement system. On the other hand, once fatigue cracks have been developed, a further reduction in the capability of the asphalt concrete layer to distribute the load will result which will lead to rapid failure due to overstressing the subgrade layer.

The results also indicate that a plus or minus error in TP has a different effect on DL, and that underestimating the value of the tire pressure at any level will result in a larger change in the design life than if the tire pressure was overestimated. Moreover, for a given error in TP, the percentage change in DL increases as the magnitude of TP decreases. The deviation from this trend at intermediate levels of E_{SG} is believed to be due to changes in failure mode from being one of fatigue to one of permanent deformation or vice versa.

From the information contained in Fig's. 5.1 through 5.3, it is also evident that at all levels of TP, the percentage change in DL that results from an error in TP is dependent upon both the subgrade resilient modulus and the volume of air voids in the asphalt concrete layer. However, such interaction effects are more pronounced at intermediate levels of VV and E_{SG}.

5.4.1.2 Effect of Changing Volume of Air Voids

The percentage change in DL resulting from errors in the volume of air voids is shown in Figs. 5.4 to 5.6 and Tables 5.7 to 5.9. At all levels, the change is positive if VV is underestimated and negative if VV is overestimated.

The results show that the change in DL resulting from a change in VV is dependent upon the levels of the subgrade modulus and less dependent on the tire pressure except at high levels of subgrade modulus where the change in DL is equally dependent upon both design parameters.

The results also indicate that a plus or minus variation in VV has a different effect on DL, with the minus variations producing larger changes (or increases) in the design life. Such an observation is found to be more pronounced at higher levels of subgrade resilient modulus.

In most cases, the change in DL induced by an error in VV is found to be greater at low levels of VV i.e. at 3 and 5%. This tends to support the argument that well compacted mixes at 3% and 5% air voids could be expected to increase the design life considerably.

5.4.1.3 Effect of Changing Subgrade Resilient Modulus

The effect of changing the subgrade resilient modulus, E_{SG}, on the design life is depicted in Figs. 5.7 to 5.9 and Tables 5.10 to 5.12. The change is found to be negative if E_{SG} is underestimated and positive if it is overestimated. This can readily be explained by the fact that lower subgrade moduli values tend to enhance the amount of damage accumulated within the pavement structure in the form of excessive rutting and hence provide for premature structural failure.

The results show that the change in DL due to a change in E_{SG} is again dependent on VV but less dependent on TP. It is also recognized, in most cases, that a plus variation in E_{SG} is more serious in affecting the percent change in DL than a minus variation. This is found to be true in all cases especially at low levels of the design parameter. This can be illustrated by the following example: from Fig. 5.8(c), the percentage change in DL due to a 20% increase in a subgrade modulus of 4500 psi is found to be 750%, whereas for he same structure under the same conditions, a 20% reduction in the subgrade modulus value is found to produce only 80% change in DL. In other words, the effect of a 20% increase in a subgrade modulus of 4500 psi is found to be in the order of 9 times greater than the effect of an equal decrease.

5.4.2 Hot Climatic Region (MAAT=29°C)

The results of changing tire pressure, volume of air voids, and subgrade resilient modulus on the design life of the same pavement structure placed in a hot climate characterized by a mean annual air temperature of 29°C will be presented in the following sections.

5.4.2.1 Effect of Changing Tire Pressure

The percentage change in DL due to an induced error in TP under the hot climatical regime is shown in Figs. 5.10 to 5.12 and Tables 5.13 to 5.15. The change is again positive for a decrease in TP and negative for an increase in TP.

The percent change in DL due to a change in TP is found to depend largely on the level of both subgrade resilient modulus and volume of air voids. Unlike the case when the structure is being placed in a cold environment, the interaction effects of both E_{SG} and VV with TP in affecting the design life are found to be very pronounced at all levels of the design parameters. In other words, the effect of TP is not only a function of VV and E_{SG} but also of the mean annual air temperature of the particular regime in which the structure is situated.

The effect of underestimating TP appeared to be more influential in changing the design life than if the tire pressure was overestimated.

5.4.2.2 **Effect of Changing Volume of Air Voids**

The change in DL due to a variation in VV is depicted in Figs. 5.13 to 5.15 and Tables 5.16 to 5.18. The general trend observed before under the cold climatic regime is again identical with a positive change in DL for a decrease in VV and a negative one for an increase in VV. However, under the hot climate less change in DL is observed to occur due to an induced variation in VV.

At all levels, a plus or minus variation in VV is observed to produce a different percent change in the design life.

The results also indicate that the percentage change in DL is not only dependent on the amount of variation induced in VV but is also dependent on the levels of the other design parameters, namely, subgrade modulus, mean annual air temperature and tire pressure.

It is also found that, at both low and high levels of subgrade modulus, the change in DL due to an error in VV is more pronounced at low levels of VV than at high levels. However, at intermediate levels of subgrade modulus, the results are inconclusive.

5.4.2.3 Effect of Changing Subgrade Resilient Modulus

The results of changing E_{SG} on the DL are shown in Figs. 5.16 to 5.18 and Tables 5.19 and 5.21. From these results, similar trends as those described in section 5.4.1.3 are observed.

Furthermore, the effect of increasing the subgrade modulus by 20% or more on the design life is more recognized at lower levels of parameters.

It is also noticed that, except at high levels of TP and VV, the change in DL due to an induced change in E_{SG} is independent of the other design parameters. However, the sensitivity of the design life to changes in the subgrade modulus is greatly influenced by the mean annual air temperature of the climatological region in which the pavement structure is placed.

5.5 General Discussion of the Results

Of the four design parameters considered, an error in the estimation of the subgrade resilient modulus has the most pronounced effect on the design life especially at low levels of the design parameter. Next in order are the volume of air voids and the tire pressure, although the two have approximately equal influence at almost all levels of the parameters.

The effect of the environmental conditions in terms of the mean annual air temperature of the particular regime, is more difficult to quantify. However, such an effect was found to significantly influence the percentage change in the design life induced by variations in the other design parameters. For example, for a pavement structure being placed in a hot climate under the conditions of 4500 psi subgrade modulus, 70 psi tire pressure, and 3% air voids, the change in the design life due to an error of 20% incrase in the subgrade modulus will be 330%. Now if the same structure was placed in a colder climate under the same conditions, the percent change in the design life will be 730. This means that the sensitivity of the pavement design life to change due to an error in estimating E_{SG} has increased by more than twofold for the same structure when placed under the two extreme climates.

It has also been observed that changes in DL due to changes in any of the design parameters follow the general trends depicted in Figs. 5.19 and 5.20. Deviations from these general trends occurring at intermediate levels of E_{SG} are shown in Figs. 5.19(a) and (b) and 5.20(a) and (b).

This deviation is believed to be brought about by changes in the failure mode from being one of fatigue to one of permanent deformation or vice versa.

For weak pavements, as characterized by a low subgrade modulus of 4500 psi, in cold climates, the design life is very sensitive to changes in the mean annual air temperature at all levels of the design parameters and less sensistive to changes in air voids content and tire pressures.

For well compacted subgrades of medium strength, tire pressure has little effect on DL at 3% air voids. However, at 5% air voids the influence of increasing the tire pressure is very much pronounced under both hot and cold climates. At VV of 11%, design life is already low but tire pressure changes are felt more at high temperatures.

For strong subgrades, the effect of increasing the tire pressure is found to appreciably reduce the design life at both 3% and 5% air voids, and in both hot and cold climates. At 11% air voids, however, the effect of increasing the tire pressure is much less pronounced for both climatic regimes.

From the above discussion and the results of the main factorials contained in Tables 5.1 and 5.2, and shown in Figs. 5.21 to 5.23, it is apparent that longer design lives are expected for the same pavement structure when placed in a hotter climate. This tends to support the argument that the seasonal changes in the subgrade modulus in areas experiencing freezing and thawing effects will have more adverse effects on the overall performance of the pavement structure, and that the increased subgrade strength while in the winter time is not beneficial enough to compensate for the amount of damage accumulated within the pavement during the spring period when the subgrade modulus is at a minimum value.

It is also evident that the subgrade support value, expressed in terms of subgrade modulus, is the key factor in controlling the failure mode. Poor subgrades will tend to force the mode of failure to be one of permanent deformation, while pavements supported by stronger subgrades will most likely fail due to excessive fatigue cracking.

5.6 Summary of the Parametric Study

The significance study was conducted to accomplish the following objectives:

1. To identify the significant factors that influence the design thickness requirements.
2. To quantify the individual and/or interactive effects of such factors on the design life of flexible pavement structures.
3. To provide guidelines for future implementation of the structural design procedure used in the study.

Two phases of the study were completed. A screening analysis comprised the first phase which disclosed the following significant factors:

1. Subgrade resilient modulus, E_{SG}.
2. Mean annual air temperature of the climatic region in which the pavement structure was placed, MAAT.
3. Percent volume of air voids contained in the asphalt concrete layer, VV.
4. Tire pressure of the wheel load system, TP.
5. Percent volume of bitumen contained in the asphalt concrete layer, VB.
6. Quality of aggregate materials used in base and subbase layers of the pavement system, K_1.

Only the first four factors were selected for inclusion in the second phase of the study.

The second phase of the investigation consisted of conducting a sensitivity analysis to quantify the effects of the different design parameters that were concluded significant from the screening analysis. This part of the study revealed the following facts:

1. Of the four design parameters considered, a variation in the subgrade modulus has the most pronounced effect on the design life especially at low levels of the design parameter.
2. Changes in the volume of air voids and tire pressure have approximately equal influence on the design life, and come second in order to the effect of changing the subgrade modulus.
3. The change in the design life resulting from a

change induced in any of the design parameters is not only dependent on the change in that factor, but also on the levels of all other factors.

4. Longer design life was obtained for the same structure when placed in a hotter climate.

5. The failure mode was found to be controlled by the subgrade modulus value. Excessive permanent deformation failure will result as a consequence of constructing the pavement over a weak subgrade while a strong subgrade will result in the mode of failure being one of fatigue cracking.

In conclusion, the significance study achieved its objectives, and served well to point out some of the areas which needed further research to better understand the influence of the various parameters that affect the design life of flexible pavements.

Subgrade Modulus E_{SG} (psi)	Volume of Air Voids VV (%)	Tire Pressure, TP (psi)		
		70	90	120
4500	3	0.493*	0.484*	0.476*
	5	0.381*	0.373*	0.367*
	11	0.220*	0.220*	0.220*
12000	3	2.12**	2.06**	2.01**
	5	1.79**	1.45**	1.05**
	11	1.97**	1.39**	0.097**
22500	3	9.40**	6.77**	4.80**
	5	3.07**	2.18**	1.53**
	11	0.313**	0.220**	0.140**

* number of load repetitions, ESAL's x 10^5
** number of load repetitions, ESAL's x 10^6

Table 5.1 Factorial for Sensitivity Analysis in Terms of Tire Pressure, Volume of Air Voids, Subgrade Modulus for Flexible Pavements in a Cold Climate (MAAT = 7°C)

Subgrade Modulus E_{SG} (psi)	Volume of Air Voids VV (%)	Tire Pressure, TP (psi)		
		70	90	120
	3	2.19*	2.12*	2.06*
4500	5	2.12*	2.05*	1.99*
	11	1.92*	1.86*	1.80*
	3	2.68**	2.58**	2.50**
12000	5	2.60**	2.50**	1.41**
	11	1.25**	0.588**	0.279**
	3	16.70**	10.20**	5.37**
22500	5	8.14**	4.19**	2.15**
	11	1.96**	0.92**	0.43**

* number of load repetitions, ESAL's x 10^5
** number of load repetitions, ESAL's x 10^6

Table 5.2 Factorial for Sensitivity Analysis in Terms of Tire Pressure, Volume of Air Voids, Subgrade Modulus, for Flexible Pavements in a Hot Climate (MAAT = 29°C)

Design Parameter	Levels of Factor	% Errors Selected
TP (psi)	70, 90, 120	± 10, ± 20, ± 40
VV (%)	3, 5, 11	± 6, ± 12.5, ±25
E_{SG} (psi)	4500, 12000, 22500	± 1, ± 5, ± 20

Table 5.3 Factors Levels and Percent Errors Used in the Sensitivity Analysis

Table 5.4 Percent Change in Design Life Due to Variations in Tire Pressure at E_{SG} of 4500 psi and MAAT of 7°C

	VV (%)								
	3			5			11		
% Variation in TP / TP Level (psi)	70	90	120	70	90	120	70	90	120
10	-4.8	0.0	0.0	0.0	-6.3	0.0	0.0	0.0	0.0
-10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	-4.8	0.0	0.0	0.0	-6.3	0.0	0.0	0.0	0.0
-20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
40	-4.8	0.0	0.0	0.0	-6.3	0.0	0.0	0.0	0.0
-40	0.0	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table 5.5 Percent Change in Design Life Due to Variations in Tire Pressure at E_{SG} of 12000 psi and MAAT of 7°C

	VV (%)								
	3			5			11		
% Variation in TP / TP Level (psi)	70	90	120	70	90	120	70	90	120
10	-0.8	-0.5	-0.3	-0.9	-6.1	-4.4	-9.8	-6.9	-5.0
-10	1.0	0.7	0.4	0.9	7.1	4.6	11.0	8.6	5.0
20	-1.6	-0.9	-0.5	-7.0	-11.6	-8.3	-17.1	-12.1	-7.5
-20	2.2	1.3	0.7	2.1	15.2	9.9	22.0	17.2	12.5
40	-2.8	-1.8	-1.0	-19.3	-20.7	-14.9	-29.3	-22.4	-15.0
-40	5.2	2.9	1.6	5.2	23.8	22.5	41.5	41.4	27.5

Table 5.6 Percent Change in Design Life Due to Variations in Tire Pressure at E_{SG} of 22500 psi and MAAT of 7°C

	VV (%)								
	3			5			11		
TP Level (psi) / % Variation in TP	70	90	120	70	90	120	70	90	120
10	-8.8	-6.5	-4.5	-9.1	-6.7	-4.7	-10.0	-6.7	-5.0
-10	10.6	7.5	5.0	11.0	7.8	5.2	12.3	9.0	6.7
20	-16.2	-12.2	-8.5	-16.8	-12.7	-9.0	-18.5	-13.5	-10.0
-20	18.9	16.3	10.6	21.1	16.9	11.0	27.7	19.1	13.3
40	-27.9	-21.7	-15.6	-28.8	-22.3	-16.2	-31.5	-23.6	-18.3
-40	36.8	38.8	23.9	39.9	40.5	24.8	50.0	46.1	28.3

Table 5.7 Percent Change in Design Life Due to Variations in Volume of Air Voids at E_{SG} of 4500 psi and MAAT of 7°C

	Tire Pressure (psi)								
	70			90			120		
VV Level (%) / % Variation in VV	3	5	11	3	5	11	3	5	11
6.0	-9.5	-6.3	0.0	-5.0	-6.3	0.0	-5.0	-6.7	0.0
-6.0	4.8	6.3	0.0	10.0	6.3	0.0	5.0	6.7	0.0
12.5	-14.3	-12.5	0.0	-10.0	-12.5	0.0	-15.0	-13.3	0.0
-12.5	9.5	12.5	11.1	15.0	12.5	11.1	15.0	13.3	11.1
25.0	-23.8	-25.0	0.0	20.0	25.0	0.0	25.0	20.0	0.0
-25.0	28.6	31.3	11.1	30.0	25.0	11.0	30.0	33.3	11.1

Table 5.8 Percent Change in Design Life Due to Variations in Volume of Air Voids at E_{SG} of 12000 psi and MAAT of 7°C

	Tire Pressure (psi)								
	70			90			120		
% Variation in VV / VV Level (%)	3	5	11	3	5	11	3	5	11
6.0	-4.1	-13.9	-13.4	-4.1	-22.0	-13.8	-4.1	-22.2	-12.5
-6.0	4.4	4.1	15.9	4.4	25.3	17.2	4.4	30.3	17.5
12.5	-8.0	-31.8	-24.4	-8.0	-38.4	-24.1	-10.2	-39.0	-25.0
-12.5	8.9	8.6	35.4	9.1	30.6	36.2	9.1	72.5	37.5
25.0	-15.3	-55.9	-41.5	-29.6	-60.4	-43.1	-47.9	-61.0	-42.5
-25.0	19.0	18.0	89.0	19.1	42.1	91.4	19.2	92.0	95.0

Table 5.9 Percent Change in Design Life Due to Variations in Volume of Air Voids at E_{SG} of 22500 psi and MAAT of 7°C

	Tire Pressure (psi)								
	70			90			120		
% Variation in VV / VV Level (%)	3	5	11	3	5	11	3	5	11
6.0	-26.5	-21.7	-13.1	-26.5	-21.9	-13.5	-26.7	-22.2	-13.3
-6.0	38.2	29.4	16.2	38.6	29.7	16.9	39.1	30.2	16.7
12.5	-44.7	-37.9	-23.8	-45.0	-38.3	-24.7	-45.4	-38.7	-25.0
-12.5	94.8	69.7	35.4	96.0	70.7	37.1	97.4	71.9	36.7
25.0	-67.4	-59.6	-41.5	-67.8	-60.1	-41.6	-68.2	-60.7	-43.3
-25.0	186.1	206.7	87.7	284.2	210.5	91.0	326.4	214.5	93.3

Table 5.10 Percent Change in Design Life Due to Variations in the Subgrade Modulus at VV of 3% and MAAT of 7°C

	Tire Pressure (psi)								
	70			90			120		
% Variation in E_{SG} / E_{SG} Level (psi)	4500	12000	22500	4500	12000	22500	4500	12000	22500
1	9.5	5.4	0.4	15.0	5.5	0.4	15.0	5.5	0.4
-1	-9.5	-6.2	-0.6	-5.0	-6.2	-0.5	-10.0	-6.2	-0.5
5	81.0	29.4	2.1	85.0	29.5	2.0	85.0	29.4	1.9
-5	-30.1	-28.3	-2.9	-35.0	-28.2	-2.7	-40.0	-28.2	-2.5
20	723.8	161.2	8.2	745.0	159.8	7.8	725.0	92.2	7.3
-20	-95.2	-77.3	-11.4	-95.0	-77.3	-10.7	-95.0	-77.2	-9.9

Table 5.11 Percent Change in Design Life Due to Variations in the Subgrade Modulus at VV of 5% and MAAT of 7°C

	Tire Pressure (psi)								
	70			90			120		
% Variation in E_{SG} / E_{SG} Level (psi)	4500	12000	22500	4500	12000	22500	4500	12000	22500
1	12.5	5.5	0.3	12.5	1.0	0.3	13.3	0.9	0.3
-1	-6.3	-6.4	-0.6	-12.5	-1.2	-0.6	-6.7	-1.1	-0.6
5	87.5	15.6	2.1	81.3	4.6	2.0	93.3	4.4	1.7
-5	-37.5	-28.9	-2.9	-37.5	-14.4	-2.8	-33.3	-5.3	-2.5
20	781.3	32.0	8.3	756.3	18.4	7.8	800.0	16.7	7.2
-20	-93.8	-78.2	-11.6	-93.8	-73.7	-10.9	-93.3	-64.2	-10.2

Table 5.12 Percent Change in Design Life Due to Variations in the Subgrade Modulus at VV of 11% and MAAT of 7°C

	Tire Pressure (psi)								
	70			90			120		
E_{SG} Level (psi) / % Variation in E_{SG}	4500	12000	22500	4500	12000	22500	4500	12000	22500
1	11.1	1.2	0.8	11.1	1.7	1.1	11.1	2.5	0.0
-1	0.0	-1.2	-0.8	0.0	-1.7	-0.0	0.0	-0.0	0.0
5	66.7	4.9	2.3	66.7	5.2	2.2	66.7	5.0	1.7
-5	0.0	-6.1	-3.1	0.0	-5.2	-2.2	0.0	-5.0	-1.7
20	544.4	20.7	8.5	366.7	19.0	9.0	233.3	20.0	8.3
-20	-88.9	-26.8	-12.3	-88.9	-24.1	-11.2	-88.9	-22.5	-10.0

Table 5.13 Percent Change in Design Life Due to Variations in Tire Pressure at E_{SG} of 4500 psi and MAAT of 29°C

	VV (%)								
	3			5			11		
TP Level (psi) / % Variation in TP	70	90	120	70	90	120	70	90	120
10	-1.1	0.0	0.0	-1.1	0.0	0.0	-1.3	0.0	-10.2
-10	1.1	1.1	0.0	1.1	1.2	0.0	1.3	1.3	10.2
20	-1.1	-1.1	-1.2	-2.3	-1.2	-1.2	-2.5	-1.3	-16.9
-20	2.2	2.3	0.0	2.3	1.2	1.2	2.5	1.3	22.0
40	-3.3	-2.3	-1.2	-3.4	-1.2	-1.2	-3.8	-6.5	-28.8
-40	6.6	3.4	1.2	5.7	3.5	1.2	6.3	3.9	28.8