SELECTION OF PROPER MODE FOR PAVEMENT REHABILITATION – A CHALLENGE FOR ENGINEERS

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Abstract

The Government of Sri Lanka is investing heavily on infrastructure development which is the gateway to the economic development of the country. Accordingly, a significant number of national, provincial and rural road projects have been implemented. Designs for most of these road projects are done according to the international standards. There are a number of techniques available to rehabilitate or upgrade pavements and the intended benefits cannot be reaped unless a judicious technique is followed. This paper covers an example of an estimating design equivalent standard axles and a comparison of two common surfacing techniques, viz., thin bituminous surfacing and thin asphalt surfacing. Pavement structure is analysed using mechanistic method. The significance of mechanistic method over empirical approach is highlighted. The fallacy of the belief that the design life of pavement increases monotonically with the thickness of asphalt layer is revealed. The analysis leads to the selection of suitable pavement structure.

Keywords: Pavement design review, failure modes of pavements, equivalent standard axels (ESAs), mechanistic approach, asphalt

1. Overview

The Government of Sri Lanka is planning to invest heavily on infrastructure development which is the gateway to the economic development of the country, and accordingly a significant number of national, provincial and rural road projects have been implemented. Funds for these projects have been obtained from donors, mainly, Asian Development Bank, World Bank, Japan International Corporation Agency in the form of development loans, and from Chinese Banks as commercial loans. The main objectives of obtaining these loans are: (a) speedy road development; (b) improve quality and durability of road assets; (c) provide safe, easy, speedy and comfortable transport for people and goods; (d) cost effective design, construction and maintenance; and (e) provide value for money.

Designs for most of these road projects are done in accordance with the international standards, and the type of pavement, width of carriageway, surface texture, construction material and techniques are specified, based on the existing road conditions, future traffic volumes and annual traffic growth, socio-economic development factors, environmental factors, maintenance aspects and return on investment. Any modifications to the design, if needed, should be carried out after careful investigation of the existing pavement conditions, traffic volumes, and other relevant parameters. But this aspect has not been seriously considered during the construction stage in some projects, and modifications are introduced on ad-hoc basis.

Study on a similar design modification done for a well-designed donor funded provincial road improvement project in a rural area in the dry zone of Sri Lanka has been carried out to find out the suitability of the modified design with respect to traffic volumes and structural design.

2. Estimation of Traffic Volume

The 12 hour manual classified traffic volume count was performed in four days in a provincial road in a rural area of Sri Lanka. Since the road falls in to “C-Class” with an average carriageway width of less than 5 metres and low traffic volumes, it is assumed that wheel path of the vehicles are at the middle of the road. Thus, traffic count is performed regardless of the direction of movement. The extracted 12 hour traffic volume data are converted to 24 hour traffic volume by multiplying with the calculated conversion factor. The resultant average number of 24 hour (daily) traffic volume was obtained as shown in Table 2.1.

Table 2.1: Average daily traffic volume in a Provincial Road.

Vehicle Type		*24 hour volume*
Motor Cycles	*Light Vehicles*	*429*
Three Wheelers		*73*
Cars/Vans/Jeeps		*54*
Light Goods Vehicles	*Commercial Vehicles*	*2*
Medium Goods Vehicles		*103*
Heavy Trucks		*35*
Medium Buses		*4*
Large Buses		*24*
Land Vehicles		*10*

3. Estimation of Equivalent Standard Axles (ESAs)

Since the axel load survey was not carried out for this road, axle load data in a similar road is considered to calculate the ESAs. The pavement performance is influenced only by the heavy end of the traffic spectrum, thus heavy commercial vehicles only were considered in the calculation of axle loading. To convert axle loads to an average equivalency factor (EF) per axle, the relationship given in equation (1) is used.

$$F = \left(\frac{L}{L_s} \right)^4 \tag{1}$$

In this study, it is assumed that fourth power rule applies in causing the structural damage to the flexible pavement. The rule can be stated thus; the structural damage caused by an axle load, L varies as the fourth power of its ratio to the standard axle load, L_s which is equivalent to 80 kN with single axle dual tyre configuration. Table 3.1 shows the loads on axle configurations that cause the same amount of damage as the standard axle.

Table 3.1 Axle loads which cause equal damage

Axle configuration	*Single Single*	*Single Dual*	*Tandem Dual*	*Triaxle Dual*
Load (kN)	*53*	*80*	*135*	*181*

Whether fully loaded, partially loaded or empty, the average EF per particular vehicle type is calculated. Average EF values obtained for each vehicle type is shown in Table 3.2.

Table 3.2 Average Equivalent Factor value for different vehicle types

Vehicle Type	*EF*
Light Goods Vehicles	*0.019*
Medium Goods Vehicles	*0.344*
Heavy Trucks	*1.524*
Medium Buses	*0.026*
Large Buses	*0.979*
Land Vehicles	*0.186*

The design parameter required is the number of equivalent standard axles (ESAs). The initial daily number of ESAs, N_E is calculated as follows;

$$N_E = \sum_j N_{Aj} * EF_j \tag{2}$$

where N_{Aj} is the daily number of vehicles of type j
(j=1 for light goods vehicles, j=2 for medium goods vehicles, j=3 for heavy trucks, j=4 for medium buses, j=5 for large buses, j=6 for land vehicles)

EF_j is the equivalent factor for vehicle type j

The design number of ESAs is then calculated as:

$$ESAs_{(Design)} = N_E * 365 * GF \tag{3}$$

where GF is the cumulative growth factor calculated using equation (4)

$$GF = \frac{(1+\mathrm{r})^{n} - 1}{r} \tag{4}$$

where r is the annual growth rate of traffic
n is the assigned design life for the road

Table 3.3 shows the calculated base year ESAs.

Table 3.3 calculated ESAs for the base year

Vehicle Type	*EF*	*ADT*	*ESAs*
Light Goods Vehicles	*0.019*	*2*	*0.039*
Medium Goods Vehicles	*0.344*	*103*	*35.423*
Heavy Trucks	*1.524*	*35*	*53.353*
Medium Buses	*0.026*	*4*	*0.106*
Large Buses	*0.979*	*24*	*23.506*
Land Vehicles	*0.186*	*10*	*1.864*
			114.291
	ESAs (Base)		*4.17E+04*

The following two assumptions were considered while calculating the cumulative ESAs:

1. Annual vehicle growth rate is 5%, and no traffic is generated or diverted due to rehabilitation work
2. No major construction activities or developments are taken place at either sides of the road during the life time of the road (i.e., 20 years)

Though above assumptions look much reasonable, unplanned commercial developments may take place, and some abrupt changes in traffic may occur during design life time of the road.

The cumulative ESAs are calculated for assigned design life of 20 years, and is shown in Table 3.4.

Table 3.4 Cumulative values of ESAs

Years from Base year	*Cumulative ESAs*
ESAs (Base Year)	*4.17E+04*
5 years	*2.31E+05*
10 years	*5.25E+05*
15 years	*9.00E+05*
20 years (Design Life)	*1.38E+06*

4. Structural Design of Pavement

4.1 Introduction

In the past, the design life of flexible pavements is estimated by empirical methods. In a way, this is not different in approach in the other fields of engineering as well. However, such methods do not give any idea of the stress and strain distributions in the pavement, and thereby the users are kept unaware of some of the hidden failure modes which could likely to occur.

The current trend, therefore, is to use mechanistic methods (Austroads, 2004, 2008) or mechanistic-empirical methods (AASHTO, 2008) by which the designer is informed of the stress, strain and deformation at any given point of the pavement. By applying specific failure criteria for each component of the pavement, its service life can be assessed.

4.2 Failure Modes

(i) Pavements with bituminous seal

Falls under this category are the pavements having unbound granular bases with bituminous seal such as spray seal, chip seal, DBST, etc. The mode of failure considered in the design is rutting or the loss of surface shape. Empirical methods are still applied for the design of this kind of pavements.

(ii) Pavements with thick layer of asphalt base and wearing course

Two major modes of failure are considered in the structural design for this kind of pavements. One is rutting or loss of surface shape, and the other is cracking of asphalt initiated from its bottom. Endurance of a pavement against rutting failure depends predominantly on the vertical compressive strain at the top of the subgrade, caused by axle loading, while that against cracking failure is considered dependent on the horizontal tensile strain at the bottom of the asphalt layer. If not by coincidence, the failure in both modes does not occur simultaneously. Commonly, the cracking failure takes precedence. Mechanistic methods are considered superior for the design of this type of pavements.

(iii) Pavements with thin asphalt surfacing

The horizontal tensile strain at the bottom of the asphalt layer is generally considered the parameter governing the cracking of thick layers of asphalt. In the event of thin asphalt layers, the design life is overestimated due a number of causes (Austroads, 2008). In thick asphalt layers, tensile strains exist at the bottom and compressive strains at the top; whereas, in thin asphalt layers, the entire layer is in tension. This difference can be considered a reason for over-estimation of life based on the tensile strain at bottom.

Some of the other reasons for the amplification of tensile strain and reduction of actual life:

- non-uniform vertical stress applied by tyres, with greater stress towards the edge;
- horizontal stresses caused by braking, accelerating, turning and climbing;
- inferior bonding between the thin asphalt layer and the underlying unbonded granular layer;
- rapid cooling of thin layer during placing, and thereby causing non-uniform, poorly compacted layer;
- fast oxidation and hardening of thin layers of asphalt;
- effect of moisture in the granular layer; and
- variability of thickness.

4.3 Design under review

A pavement design review for a well-designed donor-funded provincial road improvement project has been carried out to find the suitability of the modified design with respect to traffic volumes and pavement design. The structure proposed by the designer consists of 40 mm thick asphalt wearing course, 150 mm thick granular base course, 175 mm thick granular sub-base laid on soil subgrade. It was reported to have been for a 20-year life. The design traffic reported to have been considered is $1\text{x}10^6$ ESAs.

4.4 Review of Design

The pavement structure was analysed by kenpave software using linear-elastic option (Huang 2003). The standard axle considered was 80 kN single axle dual tyre (SADT) type (Austroads, 2008), and is shown in Fig.4.1. The tyre contact pressure was considered as 750 kP

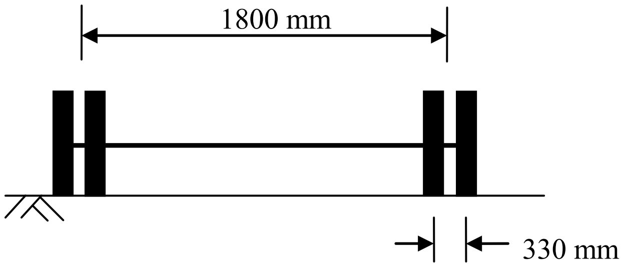

Figure 4.1 Configuration of the standard axle load

The weighted mean annual pavement temperature (WMAPT), based on weather records of the region was taken as 40 °C. Elastic properties of the layers (*E* and *v*), method of sub-layering, and fatigue criteria as given in Austroads (2008) was applied. The CBR of subgrade was assumed as 5%. The thickness of the asphalt layer was varied, while keeping that of the others constant, and corresponding effect on the strain in the asphalt layer and the life of pavement in terms of equivalent standard axles (ESAs) was studied.

The effect of the asphalt layer thickness on the life of the pavement is shown in Fig. 4.2. It is to be noted that the life was calculated ignoring the factors listed in Section 4.2 (iii), which contribute to the amplification of tensile strain in the asphalt layer and the reduction of the actual life.

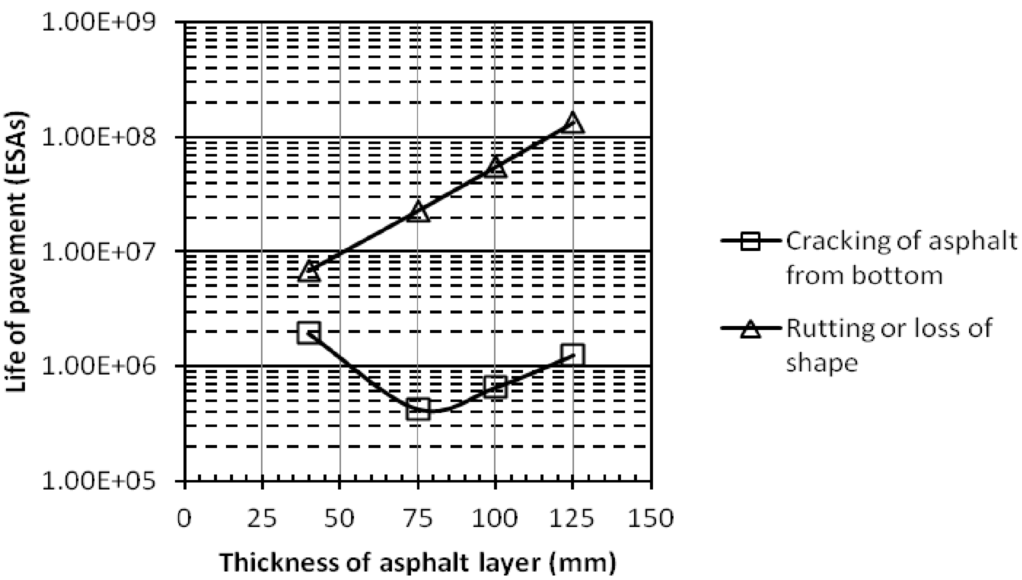

Fig. 4.2 Effect of the thickness of asphalt layer on the life of Pavement

Fig. 4.2 shows that within the range of asphalt thickness covered in this study, the pavement reaches its end of life by cracking the asphalt layer, and not by rutting. When the tensile strain amplification factors are not considered, the life of the pavement with 40 mm thick asphalt layer is 1.9×10^6 ESAs. In reality, with the strain amplification, the life should be less. A trend of decreasing the pavement life with increasing the asphalt layer thickness up to 75 mm is also an important observation. In laying asphalt, the thickness of it may vary, and if it becomes 75 mm, the life drops to 4.2×10^5 which is well below the expected design life for 20 year period. According to the original design, the expected design life is around 1.0×10^6, whereas according to the reviewers' estimate, it is 1.38×10^6. The consultant's claim of 6.0×10^6 is erroneous, because such a high value is possible only if the non-critical mode of rutting is considered, which may be a reality if spray seal or chip seal is used instead of asphalt.

The effect of the thickness of asphalt layer on the strain at critical locations in the pavement is shown in Fig. 4.3. The sign convention applied for strain is: compressive – positive; and tensile – negative.

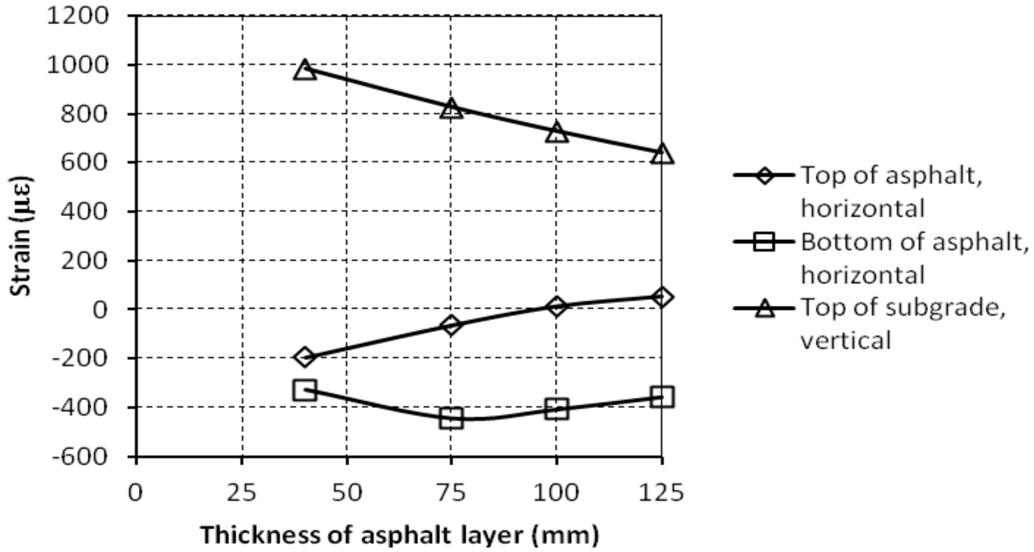

Fig. 4.3 Strain at the critical points in pavement

It is a known fact that the tensile strain in asphalt controls cracking of it, whereas the vertical compressive strain at the top of the subgrade controls rutting or the loss of shape of the wearing surface (Huang, 2003). Fig.4.3 representing the results of analysis shows that, the vertical compressive strain at the top of the subgrade decreases monotonically, reducing the chances of failure by rutting, with the increase in thickness of the asphalt layer. Bottom of asphalt layer is in tension as expected, with the largest tensile strain in it occurring when the thickness of it is around 75 mm. In other words, the greatest tendency to initiate cracking at the bottom of the asphalt layer is when its thickness is about 75 mm.

The most interesting behaviour which will be described next would not be seen unless the mechanistic approach is followed. That is, the top of the asphalt layer is also in tension unless the thickness of the asphalt layer is less than 100 mm. For example, with 40 mm thick asphalt, the bottom of the asphalt has a horizontal strain of 328×10^{-6} (tensile) and top has a strain of 194×10^{-6} (tensile). Hence, it is clear that since the entire asphalt layer is in tension, the tendency to crack the asphalt is high. If the strain amplification factors mentioned previously were taken into account, the situation would be seen more severe. Therefore, in the case presented, if asphalt is used as surfacing, the minimum thickness of it should be 100 mm.

5. Conclusions

The 40 mm asphalt wearing course is in danger of premature cracking, and hence is unsuitable. However, if instead of asphalt, bituminous seal, such as spray seal, chip seal, DBST is applied, the presently proposed structure for the base and subbase is adequate to achieve the design life. This highlights the fallacy of the belief that thin asphalt surfacing is superior to bituminous seal.

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