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COMPARATIVE STUDY OF FLEXIBLE PAVEMENT DESIGNS FOR NEW ROAD IN GREAT BRITAIN AND INDONESIA

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Abstract - The objective of this article is to compare the design methods for flexible pavements for new roads and overlays between two countries, Great Britain and Indonesia.
Pavement design procedures provide a structure that can be accepted in a specific environmental condition and able to satisfy the anticipated traffic loading. There are many factors influencing the thickness design of pavement, such as pavement materials, traffic load and climatic conditions, there is no doubt that the method of pavement design in one country may differ to another.
This article reviews the design methods of flexible pavement for new roads in both countries. Simple comparative examples are presented together with discussion and comparision of the design methods. The results are then summarised in the concluding section.

Keywords: flexible pavement, new roads, design methods

INTRODUCTION

Natural soil is not able to support wheel load directly imposed on it without significant deformation. In order to reduce the stress intensity on the natural soil, a pavement is necessary.

A pavement is a structure placed between the natural soil and the wheel, its function is to disperse traffic stresses transferred to the natural soil which will not exceed the strength of the natural soil, and further to carry traffic comfortably, convenciently, safely and economically.

Generally, pavements are divided into two catagories, flexible and rigid pavements. The difference between the two types of pavements lies principally in the way in which the wheel loads are distributed over the natural soil.

The flexible pavement is defined as a structure which maintains intimate contact with and distributes loads to the natural soil and depends upon aggregate interlock, particle friction and cohesion for stability. Hence, it is composed of a series of layers with the highest quality material at or near the surface. As load is transferred through the layers to the natural soil, the stress intensity progressively decreases with depth. The thickness design of this pavement is largerly influenced by the strength the natural soil (see Figure 1-1a).

The rigid pavement is considered to as a concrete slab, reinforced or unreinforced, laid either directly on the natural soil or on a thin layer of granular material. Because of its rigidity and high modulus of elasticity, load tends to be distributed over a relatively large area on the natural soil. A major part of the structural capacity of this type of pavement is supplied by the slab its self (see Figure 1-1b).

The components of a flexible pavement basically consist of four elements, namely a wearing Surface, roadbase, subbase and subgrade, as shown in Figure 1-1a.

The wearing surface is the uppermost layer of a flexible pavement and designed to provide a safe and smooth riding surface skid resistance, waterproofing and also to withstand the direct traffic loads. This element is commonly provided in two layers

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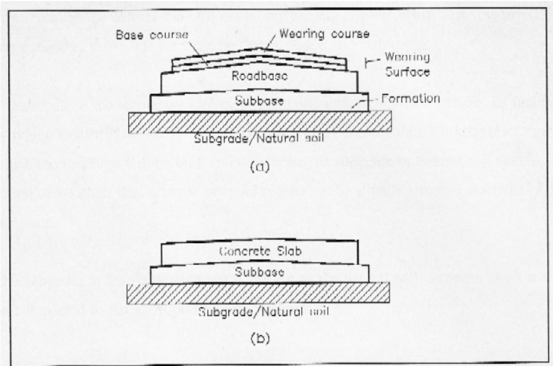

Figure 1-1 Components of (a) flexible and (b) rigid pavements

known respectively as the wearing course and the base course.

The wearing course should provide the general properties of the wearing surface described above, while the base course is essentially an extension of the roadbase which provides a good shaped surface and distributes the traffic load over the roadbase.

The roadbase is the principal load spreading layer of the pavement. It is proposed to distribute or spread the traffic load from the wearing surface such that the load transferred to the natural soil will not be sufficiently large to result in deformation. As consequently, this roadbase must be dense, stable and watertight.

The subbase is the secondary load spreading layer underlying the roadbase. Its function is to help to distribute traffic load to the natural soil, to provide a working platform on which the roadbase will be built and to insulate the subgrade or natural soil against the action of weather. The subbase material requires to be slightly stronger than that of the subgrade.

The subgrade is the foundation layer, known as the natural soil. In some cases it will be compacted to the appropriate level and profile.

In addition to those previously mentioned, the capping layer is used in some countries, and is placed between the subgrade and the subbase. The use of the capping layer will be described in the design thickness section.

The formation is defined as the surface on which the subbase layer is laid. When the capping layer is used, the formation will be the line separating the subbase and the capping layer.

In the case of rigid pavement, the cross section of this pavement is composed of a concrete slab, subbase and subgrade (see Figure 1-1b). The function of subbase and subgrade is basically the same as that in the flexible pavement, while the function of the wearing surface and roadbase are combined in the concrete slab.

However, for any types of pavements, the objective of pavement design procedure is to provide a structure that will be accepted in a specific environment and able to satisfy the anticipated traffic loading. There are many factors influencing the thickness design of pavement, such as the soil characteristic, paving material, wheel loads and climatic condition. The complexity of these factors has resulted in many thickness design methods of pavement. It is believed that the method of pavement thickness design in one country may differ to another.

This project is proposed to identify the differences and similarities of flexible pavement design methods between two countries, Great Britain and Indonesia. It reviews the design of flexible pavements proposed for new roads and for overlay for both countries. It will also include an example of the design calculation, followed by the comparison of the methods adopted in the two countries. Finally the work will be summarised in the concluding section.

LITERATURE STUDY

Flexible Pavement Design for New Roads Design in Great Britain

The development of the design of flexible pavements in Great Britain was, and still is, based on the Road Note 29[10] published by the Road Research Laboratory. The method has been derived from the observation of the full-scale performance of a number of experimental roads built in the public road network. Design curves were developed to relate the overall pavement thickness and the thickness of each layer to the traffic to be carried and to the strength of the subgrade.

In order to satisfy the various conditions that influence the design of pavements, the most recent research by Transport and Road Research Laboratory (TRRL), presented in TRRL LR ll32[17], re-analyses the behaviour of the full-scale experiments of the Road Note 29. It has been possible to develop some theoretical design concepts leading to a design method which takes into account the variability of material properties, subgrade strength and difference cumulative traffic flows.

According to the TRRL researchers, the design of pavement requires the pavement to satisfy the following structural criteria (see also Figure 2-1).

a. The subgrade must be able to sustain traffic loading without excessive deformation. This is controlled by the vertical compressive stress or strain at formation level.
b. Bituminous materials and cement-bound materials used in roadbase designs for long life must not crack under the influence of traffic. This is controlled by the horizontal tensile stress or strain at the bottom of road base.

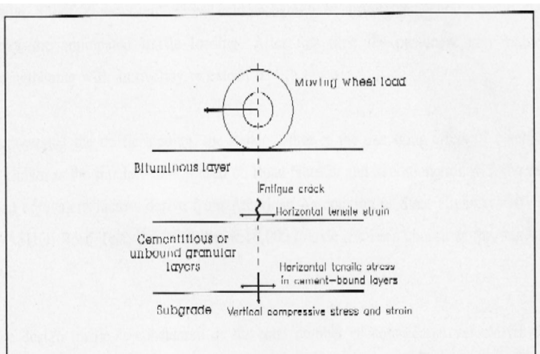

Figure 2-1 Critical stresses and strains in bituminous (flexible) pavement considered by the TRRL Researchers

c. In pavements containing a considerable thickness of bituminous material the internal deformation of this materials must not be limited. Their deformation is function of their creep characteristics.
d. The load spreading ability of granular subbase and capping layer must be adequate to provide a satisfactory construction platform.

However, the Department of Transport of Great Britain issued a new standard design for the structural design of new road pavements (HD 14/87)[7], together with the accompanying advice note (HA 35/87)[8]. This standard design takes account of all developments in recent years and provides the selection of the materials and thicknesses for road pavements. A consideration has also been given to the higher traffic loading to be predicted for many motorways and other major trunk roads. It also includes the most recent TRRL research reported in TRRL LR 1132.

Assessment of Traffic Loading, Design Life and Traffic Growth

The first stage of pavement design is a decision on the design life during which the pavement will carry the traffic loading. The advice note HA 35/87 and, in more detail, the TRRL LR 1132 show thatthe onset of the critical structural conditionof flexible pavements is when rutting of 10 mm in the wheel paths or cracking in the wheel paths occurs. Taking into account the construction, reconstruction and traffic delay costs, a design life of 20 years with 85 per cent probability of survival is generally assumed to carry the anticipated traffic loading. After this time the pavement may require strengthening with an overlay to extend its life.

In assessing the traffic loading, the standard unit of the damaging effect of a vehicle is known as the standard axle. Based on Road Note 29 and in conjunction with the axle load equivalent factors derive from American Association of State Highway Official (AASHO) Road Test, the 8,200 kg or 18,000 lb axle has been chosen as the standard axle.

The design traffic is considered as the total number of commercial vehicles in the nearside lane in one direction only over a selected design life period, which is estimated from existing traffic with an expected rate of traffic growth. It has been found that the loads imposed by private cars do not contribute significantly to the structural damage of pavement. Therefore, for the purpose of the structural design, only the number of

commercial vehicles over 1,500 kg gross vehicle weight are considered.

A study of the distribution of commercial vehicles across the width of a carriageway has been enabled a conversion of the total commercial vehicles in one direction into the number of commercial vehicles in nearside lane. Moreover, an examination of the number of axles per cofilmercial vehicles and their weights, together with information on the relative damaging effects, has provided the factors to conveft the number commercial vehicles in nearside lane to the equivalent number of standard axles 8,200 kg or 18,000 lb.

Usually, an assessment of traffic loading is required to estimate the cumulative total number of standard axles from the commercial traffic over the projected life of the road. However, the standard design HD 14/87 uses the Annual Average Daily Flow (AADF) of commercial vehicles (cv/d) at opening passing over the carriageway in one direction rather than the cumulative total number of standard axles.

Simply, these commercial vehicle flows represent the cumulative total number of standard axles since the thickness design procedure of pavement is linked to the assumed traffic growth, damaging factor and also design life.

For maintenance purpose, the cumulative standard axles is still used as the method of assessment of design traffic loading,

The advice note suggests that if the traffic flow obtained is a two way flow, where possible the directional split in the flow of commercial vehicles will be appropriate to assume to be 50/50, unless the studies show a significant bias.

The growth rate for the commercial traffic will not necessarily be the same as that for other traffic. In the case of census counts are available, they can be used in applying a growth rate for commercial traffic.

However, currently, an annual growth rate of 2 per cent is assumed without serious loss of accuracy for trunk roads and motorways.

The design traffic loading is then used to determine the thickness design of pavement. It has been simplified in the design charts with the flow of commercial vehicles expected in the year of opening of the road as the parameter. The charts include the 2.0 per cent growth rate in the flow of commercial vehicles throughout the design life and this, coupled with the rounding up of pavement thickness to the nearest 10 mm, is intended to give conservative design. An allowance has also been made for commercial vehicles travelling in one lane or single carriageway. In this case, the design traffic loading should be corrected using chart as shown in Figure A-1 of Appendix A.

Thickness Design Layers

As described previously, the pavement is used to reduce traffic stresses transferred to the natural soil or the subgrade, and the design of flexible pavement largely depends upon the strength of the subgrade.

In determining the strength of the subgrade, the California Bearing Ratio (CBR) has been used. The CBR value depends on the structure of the soil, which is influenced by the moisture condition and also the depth of the water table. A high water table produces a lower subgrade strength. The high water table is defined as at 300 mm beneath the formation level, where as the lower water table is L metre down. The CBR value should be the equilibrium value expected in the long term when the road has been completed. For differing soil conditions, the equilibrium suction-index CBR values are recommended in the TRRL LR 1132 and given in Table 2-1 below.

In assessing the subbase layer, the strength of the subbase material is influenced by the anticipated traffic loading. The design thickness of this layer should also be based on the likely CBR value of the subgrade and can be estimated from Figure A-2 of Appendix A.

However, in cases where the CBR of the subgrade is less than 5 per cent, the Department of Transport of Great Britain recommends the use of a capping layer of low-cost material to a lower specification than that the subbase.

TYPE OF SOIL	PI	HIGH WATER TABLE						LOW WATER TABLE					
		CONSTRUCTION CONDITIONS						CONSTRUCTION CONDITIONS					
		POOR		AVERAGE		GOOD		POOR		AVERAGE		GOOD	
		THIN	THICK	THIN	THICK	THIN	THICK	THIN	THICK	THIN	THICK	THIN	THICK
HEAVY CLAY	70	1.5	2.0	2.0	2.0	2.0	2.0	1.5	2.0	2.0	2.0	2.0	2.5
	60	1.5	2.0	2.0	2.0	2.0	2.5	1.5	2.0	2.0	2.0	2.0	2.5
	50	1.5	2.0	2.0	2.5	2.0	2.5	2.0	2.0	2.0	2.5	2.0	2.5
	40	2.0	2.5	2.5	3.0	2.5	3.0	2.5	2.5	3.0	3.0	3.0	3.5
SILTY CLAY	30	2.5	3.5	3.0	4.0	3.5	5.0	3.0	3.5	4.0	4.0	4.0	6.0
SANDY CLAY	20	2.5	4.0	4.0	5.0	4.5	7.0	3.0	4.0	5.0	6.0	6.0	8.0
	10	2.5	3.5	3.0	6.0	3.5	7.0	2.5	4.0	5.5	7.0	6.0	>8
SILT	-	1.0	1.0	1.0	1.0	2.0	2.0	1.0	1.0	2.0	2.0	2.0	2.0
SAND (POORLY GRADED)	-	<----- 20 ----->											
SAND (WELL GRADED)	-	<----- 40 ----->											
SANDY GRAVEL (WELL GRADED)	-	<----- 60 ----->											

Table 2-1 Equilibrium suction-index CBR values
Note : PI = Plasticity Index

The capping layer provides a working platform on which subbase construction can be built with minimum interruption from wet weather, to minimise the effect of weak subgrade on road performance and to reduce the risk of damage during construction to any materials above the capping layer, which improves contribution of these layers.

Advice given in TRRL LR 1132 indicates that the use of 150 mm of subbase laid on 350 mm of capping layer with a CBR of more than 15 per cent enables the load spreading ability of the combined layers to satisfy the requirement for a construction over subgrade with the CBR values from 2 up to 5 per cent. An additional thickness of capping layer of 250 mm is required on the subgrade with the CBR value less than 2 pet cent. In this condition, the design of roadbase and wearing surface is then the same as that for pavement constructed on soil with CBR of 5 per cent or more and a subbase of 225 mm.

The standard designs of roadbase are derived from the TRRL LR 1132, which have been developed based on the long-term performance of 144 sections of experimental road in which the construction details and structural properties were carefully measured. The structural performance of each section was assessed by monitoring systematically the development of rutting and cracking of the road surface. The development of rutting was related to the design of traffic loading. The multiple regression techniques were then used to correlate the pavement life, as defined by the critical condition of 10 mm rutting, with the thickness of pavement layers and CBR of the subgrade.

The design chart was then developed as shown in Figure A-3 of Appendix A for flexible pavements consisting of bituminous roadbases and wearing surfaces. Also developed, were the design charts for flexible composite pavements where the upper pavement layers are constructed from bituminous material and are supported on a cemented roadbase, as shown in Figure A-4 and Figure A-5 of Appendix A.

As previously mentioned, the design standard HD 14/87, described in advice note HA 35/87, takes into account higher traffic loadings in order to accommodate recent increased traffic levels. This has enabled designs to be developed for heavily trafficked roads, defined as the traffic in excess of 3,000 commercial vehicles per day at the opening.

The approach underlying the design for heavily trafficked roads is given on the basis that the reconstruction of a pavement which requires the removal or replacement of the roadbase will normally result in high traffic delay costs. Therefore it is desirable to avoid fatigue cracking in the roadbase which would usually require the replacement of both the roadbase and wearing surface.

Overcoming this problem is ensured by using a standard thickness of rolled asphalt lower roadbase which is proven to have a higher fatigue cracking resistance. The standard thickness of this special layer is 125 mm. Hence, the design chart in Figure A-3 of Appendix A should be added to the 125 mm lower roadbase layer of hot rolled asphalt, as shown in note 2 of Figure A-3 of Appendix A.

The use of this design ensures that the structure deteriorates primarily through deformation with the little risk of fatigue cracking in the roadbase.

Design in Indonesia

The flexible pavement design procedure recommended by Bina Marga(2), Directorate General of Highways of Indonesia, is mainly adopted from the results of the American Association of State Highway Officials

(AASHO)(l) Road Test with modification in order to take into account the specific conditions in Indonesia.

The AASHO design is based on a road user definition of pavement failure rather than based on structural failure concepts, such as cracking and deformation. Simply, the function of any roads is to carry safely and smoothly vehicular traffic from one point to another.

The concept is that serviceability, which is defined as the ability of the pavement to carry the designed traffic. Performance is the ability of the pavement to satisfy the design traffic over a period of time. Hence performance can be interpreted as the integral of serviceability with time (load repetitions).

Initially, this performance can be obtained from the serviceability at the time of construction and also at various times after construction. These periodic ratings of serviceability can be made on the basis of subjective rating of the riding surface by individuals who travel on it. The serviceability rating is on a scale of 0 to 5 with 5 corresponding to a very good rating

It has been found that the serviceability index (IP) at any time is a function of slope variance (a measure of longitudinal roughness), the extent and type of cracking and patching, and the rutting displayed at the surface.

It is also assumed that an initial value of serviceability index for new flexible pavement at the Road Test is 4.2, and the potential loss in serviceability taken as a value of 1.5.

The basic equations adopted by Bina Marga are the same as that developed from the AASHO Road Test, as the following.

$$G1 = \beta\,(\log Wt - \log \rho) \quad \ldots \qquad 1$$

$$G1 = \log \frac{IPo - IPt}{4.2 - 1.5} \quad \ldots \qquad 2$$

$$\beta = 0.40 + \frac{0.081\,(L1 + L2)^{3.23}}{(ITP + l)^{5.19}\,L2^{3.23}} \quad \ldots \quad 3$$

$$\log \rho = 5.93 + 9.36 \log (ITP + 1) - 4.79 \log (L1 + L2) + 4.33 \log L2 \quad \ldots \qquad 4$$

where :

G1 = a logarithm function of the ratio of loss in serviceability during the time t to the potential loss taken from a point of 4.2 to 1.5

β = a function of design and load variables that influences the shape of IP versus W serviceability curve

ρ = a function of design and load variables that presents the expected number of axle load applications to IP of 1.5

Wt = the axle load applications at the end of time t

IPo = initial serviceability, the value of IPo is influenced by the type and quality of surfacing material. Simply this value is measured by the roughness

IPt = serviceability at the end of time t

L1 = load on one single axle or on one tandem axle set (kip)

L2 = axle code, in which L2 = 1 for single axle and L2 = 2 for tandem axle

ITP = structural number of pavement, which is an index number as a conversion to the thickness of various flexible pavement layers

To simplify the solution, all the load factors are expressed in term of standard axle load, which is 18,000 pounds, so that L1 = 18 kip and L2 = 1.

As a result, the number of 18 kip single axle load application to time t is expressed as below :

$$\log W_{t18} = 9.36 \log(\underline{ITP}+1) - 0.20 + \frac{\log \dfrac{IP_o - IP_t}{4.2-1.5}}{0.40 + \dfrac{1094}{(\underline{ITP}+1)^{5.19}}} \quad \ldots 2\text{-}5$$

In order to satisfy a various conditions, a soil support (DDT) and regional factor (RF) are included into the design equation, such that :

$$\text{Log } W_{t18} = \text{Log } N'_{t18} + K(\,Si - So) \quad \ldots \qquad 6$$

$$W_{t18} = N_{t18}\,(1/RF) \quad \ldots \qquad 7$$

where :

Si = Soil support value for any condition i

So = Soil support value for Road Test condition
N'_{t18} = total application for road test condition
N_{t18} = total unweighed load application
K = regression constant (K : 0.372)

Hence the final flexible pavement design is given by :

$$\log W_{t18} = 9.36 \log (ITP+1) \quad 0.20 + \frac{\log \dfrac{IP_o - IP_t}{4.2-1.5}}{0.40 + \dfrac{1094}{(ITP+1)^{5.19}}} + \log (1/RF) + 0.372 \; (S_i - 3.0) \qquad \ldots 2\text{-}8$$

This equation relates the number of 18 kip single axle load repetitions (W_{t18}) required to reach predefined terminal serviceability index (IP) for any given pavement structure (ITP), climatic condition (RF) and subgrade soil (DDT).

Assessment of Traffic Loading, Design Life and Traffic Growth

As with the AASHO design method, the mixed traffic for a given period of time is accounted for by equivalent damage factors relative to the standard 18 kip singe axle load (8160 kg).

In assessing the traffic loading, basically it is proposed to determine the number of equivalent 18 kip single axle load applications per day on the design lane for the given design life (LER).

The term design lane is defined as one of the number of lanes on the highway which has the greatest traffic capacity. In the case where there is no lane line marking, the number of lanes are estimated on the basis of the width of the pavement as shown in Table 2-2 below.

WIDTH OF PAVEMENT (L) L in metres	NUMBER OF LANE (n)
L < 5.50	1
5.50 ≤ L < 8.25	2
8.25 ≤ L < 11.25	3
11.25 ≤ L < 15.00	4
15.00 ≤ L < 18.75	5
18.75 ≤ L < 22.00	6

Table 2-2 The number of lanes related to the width of pavement

NUMBER OF LANE	LIGHT VEHICLE*		HEAVY VEHICLE**	
	ONE DIRECTION	TWO DIRECTIONS	ONE DIRECTION	TWO DIRECTIONS
1	1.000	1.000	1.000	1.000
2	0.600	0.500	0.700	0.500
3	0.400	0.400	0.500	0.475
4		0.300		0.450
5		0.250		0.425
6		0.200		0.400

* Total vehicle weight < 5 ton, for example : cars, van, pick up
** Total vehicle weight ≥ 5 ton, for example : bus, truck, tractor, trailer

Table 2-3 Lane distribution factor (C)

The number of lanes are then used to determine the lane distribution factor (C), which is proposed as a consideration of the variation of traffic within the lanes.

The lane distribution factor is influenced by the number of lanes, vehicles weight and road directions whether one or two directions. Table 2-3 shows the various values of the lane distribution factors (C).

In estimating the traffic and its axle load distribution, the average daily traffic should be converted to the equivalent number of standard axles.

The equivalent factor (E) for each axle load is determined by the equations below :

For single axle :

$$E = \left(\frac{\text{Load on one single axle in kg}}{8160} \right)^4$$

For tandem axles :

$$E = 0.086 \left(\frac{\text{Load on one single axle in kg}}{8160} \right)^4$$

The equivalent factor can be seen clearly in Table 2-4 below. For more details, the type of vehicles and their axle loads commonly used in Indonesia are given in Table B-1 of Appendix B.

The design number of equivalent 18 kip single axle load applications per day (LER) is determined on the basis of forecasting the initial average daily traffic counted in both directions of road without directional split and in each direction of road with directional split (LHR).

AXLE LOAD		EQUIVALENT FACTOR	
KG	LBS	SINGLE AXLE	TANDEM AXLES
1000	2205	0.0002	-
2000	4409	0.0036	0.0003
3000	6614	0.0183	0.0016
4000	8818	0.0577	0.0050
5000	11023	0.1410	0.0121
6000	13228	0.2923	0.0251
7000	15432	0.5415	0.0466
8000	17637	0.9238	0.0794
8160	18000	1.0000	0.0860
9000	19841	1.4798	0.1273
10000	22046	2.2555	0.1940
11000	24251	3.3022	0.2840
12000	26455	4.6770	0.4022
13000	28660	6.4419	0.5540
14000	30864	8.6647	0.7452
15000	33069	11.4184	0.9820
16000	35276	14.7815	1.2712

Table 2-4 Equivalent factor to standard axle 18 kip (8160 kg)

This LHR should be then multiplied by the lane distribution factor (C) and by the equivalent factor (E) in order to determine the initial number of equivalent 18 kip single axle load applications per day (LEP), as shown below :

$$LEP = \sum_{car}^{trailer} LHR\ C\ E \quad ...\ 2\text{-}11$$

The projected number of equivalent 18 kip single axle load applications per day at the end of design period (LEA) is given by :

$$LEP = \sum_{car}^{trailer} LHR\ (i+1)^{UR}\ C\ E \quad ...\ 2\text{-}12$$

where :
i = growth rate (%)
UR = design life (years)

The average number of equivalent 18 kip single axle load applications per day over the design period (LET) is then calculated as below :

$$LET = (LEP + LEA) / 2 \quad …\ 2\text{-}13$$

Hence, the LER is determined as :

$$LER = LET\ FP \quad ...\ 2\text{-}14$$

where FP is the adjustment factor. Since LER adopted from the equation 2-8, denoted by W_{t18}, is only for 10 years design period, the adjustment factor for period other than 10 years is considered such that :

$$FP = UR/10 \quad ...2\text{-}15$$

And then :

$$LER = LET\ \ UR/10 \quad ...\ 2\text{-}16$$

As described previously that the design traffic is based upon forecasting the initial average daily traffic. Since Indonesia is a developing country, this appears to the difficulty in predicting traffic growth rates and for further the proposed design life of the pavement.

A suggestion has been made that the prediction of traffic can be obtained from past trends of traffic growth (over last 2 years). Consideration should also be given to the possibility of the land use pattern that may be influenced to the projected highways.

The design life decision should be based upon the road function, pattern of traffic or land use, and also the economic value from the projected highways.

Life studies suggest that for developing countries, the period of time of 10 up to 20 years is usually appropriate as the design life.

Thickness Design Layers

The equation 2-8, derived from the AASHO Road Test, is adopted by Bina Marga as the basic design to determine the thickness layers of pavement.

It has been simplified on the nomographic solution, as shown in Figure B-1 up to Figure B-9 of Appendix B, with the consideration to a various values of IPo, IPt, and the effects of climate condition which is denoted in term of FR.

The CBR is used to determine the strength of the subgrade, which is denoted by DDT a rating representing the subgrade strength. This CBR can be obtained from a field or laboratory test according to a standard soil test in Indonesia'.

However it has been suggested that in designing pavements for new roads, the CBR laboratory test must be used.

The correlation between CBR and DDT has been made and is expressed by :

$$DDT = 4.3 \log (CBR) + 1.7 \quad ...\ 2\text{-}17$$

This correlation is also shown in Figure B-10 of Appendix B.

As described previously that this design method is based on serviceability, expressed by serviceability index (IP), which has the rating scale from 0 to 5.

Some examples of the rating scale are described below :

a. IP = 1.0 denotes a serious damage on road surface such that traffic is severely disturbed

b. IP = 1.5 denotes the possible lowest level of services that road is not cut off

c. IP = 2.0 denotes the lowest level of services for stable road

d. IP = 2.5 denotes the road surface is in good and stable condition

To select the appiopriate terminal serviceability for design (IP), it is necessary to take account the classification of road and design number of equivalent 18 kip single axle load applications (LER).

The various value of IP, are given in Table 2-5 below :

LER	ROAD CLASSIFICATION			
	LOCAL	COLLECTOR	ARTERIAL	TOLL WAY
< 10	1.0 - 1.5	1.5	1.5 - 2.0	-
10 - 100	1.5	1.5 - 2.0	2.0	-
100 - 1000	1.5 - 2.0	2.0	2.0 - 2.5	-
> 1000	-	2.0 - 2.5	2.5	2.5

Table 2-5 The terminal serviceability values

In order to estimate the initial serviceability for design (IP0o), consideration should be given to the type and roughness of surface. This can be seen in the Table 2-6 below, and the definition of the pavement materials are given in Appendix E.

TYPE OF PAVEMENT SURFACE	IPo	ROUGHNESS (mm/km)
LASTON	≥ 4 3.9 - 3.5	≤ 1000 > 1000
LASBUTAG or HRA	3.9 - 3.5 3.4 - 3.0	≤ 2000 > 2000
BURDA	3.9 - 3.5	≤ 2000
BURTU	3.4 - 3.0	> 2000
LAPEN	3.4 - 3.0 2.9 - 2.5	≤ 3000 > 3000
LATASBUM or BURAS or LATASIR	2.9 - 2.5	--
SAND	≤ 2.4	--
GRAVEL	≤ 2.4	--

Table 2-6 The initial serviceability values

The regional factor (RF) has been included in the AASHO design procedure in order to allow for the various conditions other than those occurring during the AASHO Road Test. The modification has been made in order to take account the specific conditions in Indonesia. In this case, the conditions are interpreted as the field conditions and climate which affect to the traffic loading and soil support.

The field conditions will include the permeability of the soil, drainage, alignment, percentage of heavy vehicles with weight more than 13,000 kg, and the percentage of stopping movements. The climate condition is expressed in term of an average annual rainfall in mm/year.

According to Indonesian Highways Construction Regulations, the effect of permeability of the soil and the drainage is assumed to be identical, hence this regional factor depends only upon the alignment (gradient), percentage of heavy vehicles and of stopping movements, and the average annual rainfall.

The regional factor (FR) is then presented as below :

AVERAGE ANNUAL RAINFALL (mm/year)	GRADIENT < 6 %		GRADIENT 6 - 10 %		GRADIENT > 10 %	
	% HEAVY VEHICLE		% HEAVY VEHICLE		% HEAVY VEHICLE	
	≤ 30 %	> 30 %	≤ 30 %	> 30 %	≤ 30 %	> 30 %
< 900	0.5	1.0 - 1.5	1.0	1.5 - 2.0	1.5	2.0 - 2.5
≥ 900	1.5	2.0 - 2.5	2.0	2.5 - 3.0	2.5	3.0 - 3.5

Table 2-7 Regional Factor values

In determining the thickness of layers, as previously mentioned, it is expressed in term of the structural number (ITP), which is an index number derived from the analysis of traffic, soil conditions and regional factors that may be converted to the thickness of various flexible pavement layers. This value of ITP is obtained from the nomographic solution in Figure B-1 up to Figure B-9 of Appendix B. The relationship between the ITP and layer thicknesses is given by the following formula :

$$ITP = a1\ D1 + a2\ D2 + a3\ D3 \quad \ldots\ 2\text{-}18$$

where :

a1, a2, a3 = layer coefficients corresponding to wearing surface, roadbase and subbase respectively

D1, D2, D3 = the actual layer thicknesses corresponding to wearing surface, roadbase and subbase respectively

The layer coefficients are used as the empirical relationship between ITP for a pavement structure and layer thicknesses which express the relative ability of a material being used in each layer as a structural component of the pavement.

Table 2-8 shows the various values of layer coefficients for given types of materials, derived from the Marshall Test for asphalt material, Compressive Strengths for material stabilised by cement or lime and CBR for roadbase or subbase material.

However since the flexible pavement is a layered structure, each layer should be examined so that an adequate thickness is provided as cover. Consequently this appears as the minimum allowable thickness of any given layers.

The minimum layer thicknesses are then given in the Table 2-9.

LAYER COEFFICIENT			STRENGTH OF MATERIAL			TYPE OF MATERIAL
a1	a2	a3	MS (kg)	CS (kg/cm)	CBR (%)	
0.40	-	-	744	-	-	Laston
0.35	-	-	590	-	-	
0.32	-	-	454	-	-	
0.30	-	-	340	-	-	
0.35	-	-	744	-	-	Lasbutag
0.31	-	-	590	-	-	
0.28	-	-	454	-	-	
0.26	-	-	340	-	-	
0.30	-	-	340	-	-	H R A
0.26	-	-	340	-	-	Asphalt Macadam
0.25	-	-	-	-	-	Lapen (mechanic)
0.20	-	-	-	-	-	Lapen (manual)
-	0.28	-	590	-	-	Upper Laston
-	0.26	-	454	-	-	
-	0.24	-	340	-	-	
-	0.23	-	-	-	-	Lapen (mechanic)
-	0.19	-	-	-	-	Lapen (manual)
-	0.15	-	-	22	-	Soil cement stabilised
-	0.13	-	-	18	-	
-	0.15	-	-	22	-	Soil lime stabilised
-	0.13	-	-	18	-	
-	0.14	-	-	-	100	Crushed stone (Class A)
-	0.13	-	-	-	80	Crushed stone (Class B)
-	0.12	-	-	-	60	Crushed stone (Class C)
-	-	0.13	-	-	70	Granular subbase (Class A)
-	-	0.12	-	-	50	Granular subbase (Class B)
-	-	0.11	-	-	30	Granular subbase (Class C)
-	-	0.10	-	-	20	Sand / sandy clay

Note : MS : Marshall Test
CS : Compressive Strength
CBR : California Bearing Ratio

Table 2-8 Layer coefficients

Stage Construction

As previously mentioned, in developing countries there appears to be a difficulty in predicting traffic growth, moreover the design life of the pavement. In some cases, change of grade and alignment may cause the road to become out-of-date before reaching the physical condition that it should be replaced.

Another problem is to limit of pavement construction cost especially when a longer design life is proposed.

ITP	MINIMUM THICKNESS (cm)	TYPE OF MATERIAL
WEARING SURFACE		
< 3.00	5	Buras, Burtu, Burda
3.00 - 6.70	5	Lapen/Asphalt Macadam, HRA, Lasbutag, Laston
6.71 - 7.49	7.5	Lapen/Asphalt Macadam, HRA, Lasbutag, Laston
7.50 - 9.99	7.5	Lasbutag, Laston
≥ 10.00	10	Laston
ROADBASE		
< 3.00	15	Crushed stone, soil cement or lime stabilised
3.00 - 7.49	20	Crushed stone, soil cement or lime stabilised
	10	Upper Laston
7.50 - 9.99	20	Crushed stone, soil cement or lime stabilised, macadam
	15	Upper Laston
10.00 - 12.14	20	Crushed stone, soil cement or lime stabilised, macadam, lapen, upper laston
≥ 12.25	25	Crushed stone, soil cement or lime stabilised, macadam, lapen, upper laston
SUBBASE		
All	10	----

Table 2-9 Minimum layer thicknesses

Yoder and Witczak[21] stated that in pavement design decision, the balance between increasing the maintenance costs and the initial (construction) costs should be considered. For example, if an initial design is to be minimised by providing thin pavement section, the maintenance cost increases.

On the other hand, by increasing the initial cost in which building a stronger pavement proposed to a longer design life, the maintenance cost decreases accordingly. However, they suggested that in developing areas the most economical approach is to build the road in stages rather than to incur a large initial cost.

This suggestion seems to be adopted in some cases in Indonesia as an alternative to the design of the flexible pavement. The method of design for stage construction is based on the concept of remaining life. Since the procedure involves planned stage construction, the pavement is designed on the assumption that the planned overlay will be placed before the pavement has used of all of its fatigue life.

The design procedure is such that the pavement can be designed in two stages. It is

necessary to select the first stage design period equal to 25 to 50 per cent of the total design period selected for the project. For example, if the design period is to be 20 years, then the first stage design period should be in the range of 5 to 10 years and the second stage in the range of 10 to 15 years.

To ensure that the first stage layer of the flexible pavement will operate effectively with the second, the second layer must be applied when the cumulative damage in the first layer, according to the hypothesis, reaches 60 per cent, so that the remaining life in the existing layer is 40 per cent.

If the residual life at the end of the first stage is 0 per cent or there is no remaining life at the end of the first stage, the design number of equivalent 18 kip single axle load applications per day for the first stage is denoted by LER1.

If the residual life at the end of the first stage is expected to be 40 per cent, the thickness design of the pavement at the first stage should be increased by applying the design number of equivalent 18 kip single axle load applications per day during the first stage, denoted by x LER1.

It is assumed that there is a linear relationship between the remaining life and the remaining traffic.
Hence :

x LER1 = LER1 + 40 % x LER1 … 2-19

Therefore :
x = l.67
If the design number of equivalent 18 kip single axle load applications per day during the first and second stages is denoted by y LER2, and the design number of equivalent 18 kip single axle load applications per day for the second stage is LER2, and since 60 per cent of y LER2 has been used up in the first stage, so that :

y LER2 = 60% y LER2 + LER2 … 2-20

Therefore :
y = 2.5

For the second stage, the calculated structural number ITP can be determined as below :

ITP2 = ITP – ITP1 … 2-21

where :
ITP is obtained from the nomogram with LER = 2.5 LER2
ITP1 is obtained from the nomogram with LER = 1.67 LER1

Comparative Example Design for New Road

In comparing the methods of flexible pavement design for new roads in the two countries under consideration, the parameters have to be assumed and should be applied in both countries.
The assumed parameters are as the following :

1. The estimated design traffic at the opening is 1400 commercial vehicles per day counted in both directions.
   Based on Transport Statistics Great Britain (1992)[12], these commercial vehicles consist of :
   Buses and coaches
   18.00 % = 252 vehicles
   Goods vehicles 2 axles rigid
   60.1l % = 842 vehicles
   Goods vehicles 3 axles rigid
   5.76 % = 81 vehicles
   Goods vehicles 3 axles articulated
   2.31 % = 32 vehicles
   Goods vehicles 4 axles articulated
   13.82 % = 193 vehicles
   These compositions of traffic can then be used in assessing the traffic loading in Indonesia.
2. It is proposed to design a flexible pavement for a new road for design life of 20 years with 2.00 % traffic growth rate. Hence the design charts in Great Britain can be applied.
3. The CBR is assumed to be 15.0 %, to make the capping layer is not required in Great Britain, in order to enable the comparison with Indonesia.
4. The road is dual carriageway with the maximum gradient of 4 %.

5. The rainfall is assumed to be 838 mm/year (based on Transport Statistic 1.992, for England and Wales).

British Design

Procedure :

1. Assessment of traffic loading
   Assuming a 50/50 directional split, the design traffic is given by :
   Design traffic : 1400/2 = 700 cv/d in each direction.
2. Determine the thickness of subbase
   From Figure A-2 of Appendix A and with CBR of 15.0 %, the capping layer is not required. Then : Subbase : 225 mm (Granular Type 1)
3. Determine the thickness of roadbase and surfacing
   From Figure A-3 of Appendix A with the design traffic of 700 cv/d, the thickness of surfacing plus roadbase is 300 mm.
   According to note 1 in Figure A-3 of Appendix A, for design traffic less than 3,000 cv/d, the construction should be 40 mm wearing course, 60 mm base course, and the remainder roadbase, then :
   Wearing course= 40 mm (Rolled Asphalt)
   Base course= 60 mm (Rolled Asphalt)
   Roadbase = 300-100
   = 200 mm (Dense Bitumen Macadam)
4. As a result :

Wearing course	= 40 mm
Base course	= 60 mm
Roadbase	= 200 mm
Subbase	= 225 mm
Total	= 525 mm

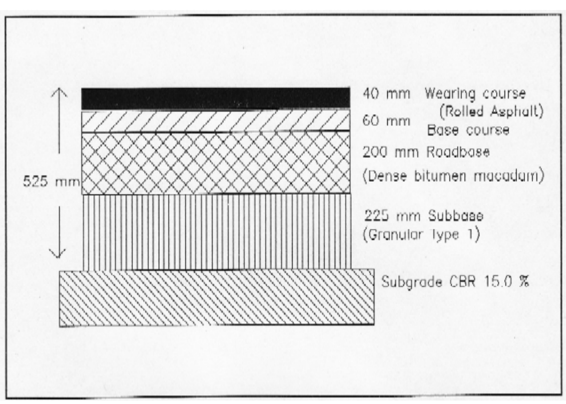

Figure 3.1 Thickness design layers obtained in Great Britain

Indonesian Design

Procedure :

1. Determine the equivalent factor (E)
   Assuming the vehicles weights are as in Table B-1 of Appendix B, the equivalent factors are as below :
   - Buses and coaches 9.0 tonnes (B&C) = 0.3006
   - Truck 2 axles rigid 18.2 tonnes (T2R) = 5.0264
   - Truck 3 axles rigid 25 tonnes (T3R) = 2.7416
   - Truck 3 axles articulated 26.2 tonnes (T3A) = 6.1179
   - Truck 4 axles articulated 31.4 tonnes (T4A) = 4.9283

2. Determine lane distribution factor (C)
   The road is dual carriageway (2 lanes) and the traffic is counted in both directions.From Table 2-3, the lane distribution factor is 0.50.

3. Determine LEP
   From equation 2-11, the initial number of equivalent standard axles per day, LEP, can be calculated as below :
   - B&C = (252) (0.5) (0.3006) = 37.876
   - T2R = (842) (0.5) (5.0264) = 2116.114
   - T3R = (81) (0.5) (2.7416) =111.035
   - T3A = (32) (0.5) (6.1179) = 97.886
   - T4A = (193) (0.5) (4.9283) = 475.581
   LEP = 2838.492 standard axles

4. Determine LEA
   From equation 2-12, with 2 % gtowth rate and 20 years design life, the projected number of equivalent standard axles per day at the end of design period, LEA, is given as the following :
   - B&C = (252) $(l+0.02)^{20}$ (0.5) (0.3006) = 56.281
   - T2R = (842) $(l+0.02)^{20}$ (0.5) (5.0264) = 3144.435
   - T3R = (81) $(l+0.02)^{20}$ (0.5) (2.7416) = 165.004
   - T3A = (32) $(l+0.02)^{20}$ (0.5) (6.1179) = 145.454
   - T4A = (193) $(l+0.02)^{20}$ (0.5) (4.9283) = 706.688
   LEA = 4217.862 standard axles

5. 5. Determine LET
   Using equation 2-13, the average number of equivalent standard axles per day over the design period, LET, is given as below:
   LET = (LEP + LEA)/2 = (2838.492 + 4217 .862)
   LET = 3528.117 standard axles
6. Determine LER
   From equation 2-16, and with 20 years design life, the design number of equivalent standard axles per day, LER, is given by :
   LER = LET UR/10 = 3528.117 (20/10)
   LER = 7056 standard axles
7. Determine DDT (Soil Support Value)
   CBR = 15.0 %
   DDT = 4.3 log(15.0) + 1.7 = 6.8
8. Determine terminal serviceability IP,
   From Table 2-5, with the LER of 7056 and road classification of Arterial, the IP, is assumed to be 2.50.
9. Determine the regional factor (FR)
   From Table 2-7 , withthe rainfall of 838 mm/year, gradient less than 6.0 % and percentage of heavy vehicles (more than 13,000 kg) more than 30.0 %, the FR is assumed to be 1.00.
10. Determine the structural number (ITP)
    It is assumed that the IPo is between 3.5 and 3.9. From Figure B-1 of Appendix B, the ITP is given by 10.8.
11. Determine the layer thicknesses
    According to Table 2-9 with a consideration of the ITP value, the materials used are (see also Table 2-8 atd Appendix E for the characteristic and definition of materials) :
    - Surfacing = Laston/asphaltic concrete (MS 744)
    - Roadbase = Lapenlcoated macadam (mechanically mixed)
    - Subbase = Granular Class A (CBR 70 %)
    Then the layer coefficients according to Table 2-8 are given by :
    - Surfacing = al = 0.40
    - Roadbase = a2 = 0.23
    - Subbase = a3 = 0.13
    Essentially, three types of pavement thicknesses can be determined as the following :

If the thickness of roadbase and subbase use the required minimum thickness as shown in Table 2-9, and according to equation 2-18 :
10.8 = 0.40 (D1) + 0.23 (20) + 0.13 (10)
Dl = 13 cm
The layer thicknesses are given by :
- Surfacing = 130 mm
- Roadbase = 200 mm
- Subbase = 100 mm

If the thickness of surfacing and subbase use the required minimum thickness, then :
10.8 = 0.40 (10) + 0.23 (D2) + 0.13 (10)
D2 = 24 cm
The layer thicknesses are given by :
- Surfacing = 100 mm
- Roadbase = 240 mm
- Subbase = 100 mm

If the thickness of surfacing and roadbase use the required minimum thickness, then:
10.8 = 0.40 (10) + 0.23 (20) + 0.13 (D3)
D3 = l7 cm
The layer thicknesses are given by :
- Surfacing = 100 mm
- Roadbase = 200 mm
- Subbase = 170 mm

In this case, for economic reasons the third choice is chosen as the design, such that :
- Surfacing = 100 mm
- Roadbase = 200 mm
- Subbase = 170 mm
- Total = 470 mm

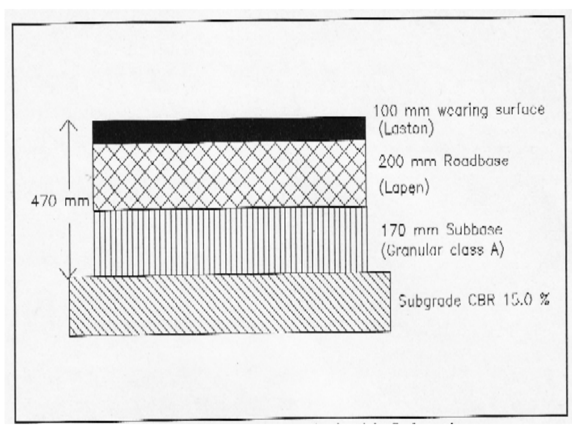

Figure 3.2 Thickness design layers obtained in Indonesia

METHOD

Comparison of Flexible Pavement Design for New Roads

In comparing design methods of flexible pavement for new roads, chapter 3 indicates that there is a difference in the pavement thicknesses obtained from the different methods in the two countries under consideration when the same parameters are used.

Due to the variability of the materials in each country, this comparison depends upon the assumption of the material used in designing a flexible pavement, and does, therefore, not provide a definitive answer.

Basically, the approaches in developing the design methods of flexible pavement in Great Britain and in Indonesia differ considerably. Great Britain uses the method developed by Transport and Road Research Laboratory (TRRL), which is based on the observation of the performance of the road pavement in term of cracking and deformation, and this has been related to the stress and strain in pavement layers. On the other hand, Indonesia uses the method developed by American Association of State Highway Officials (AASHO) from the AASHO Road Test, which is based on a road user definition of pavement failure (serviceability-performance concept) rather than based on structural failure concept (cracking or deformation). In this case, pavements may be designed for the level of serviceability expected at the end of the design period.

In assessing the traffic loading for the purpose of designing a flexible pavement for a new road, the traffic being considered in Great Britain is expressed in terms only of commercial vehicles over 1500 kg gross vehicle weight. This is due to the fact that loads imposed by private cars and light vans do not contribute significantly to the structural damage caused to pavements by traffic. By contrast, Indonesian methods requires all traffic loads to be considered, even though the damage factor to pavements by private cars and light vehicles are relatively small. This allows a pavement to be designed economically on the road where the percentage of heavy vehicles is very small. It would normally be accepted for designing a pavement in town centre or residential road where there is a restriction on heavy vehicles.

Great Britain uses the Annual Average Daily Flow (AADF) of commercial vehicles at opening in one direction, while Indonesia requires the estimation of the cumulative total number of standard axles over the projected life of the road.

Since Great Britain is a developed country, there is a less difficulty in predicting traffic growth rate of the commercial traffic, hence an annual growth rate of 2.0 pet cent is generally assumed without serious loss of accuracy for trunk roads and motorways.

Moreover, the assumption of 20 years design life provides the economic balance between a long design life and failure. The vehicle damage factors are estimated by preparing the design charts with the assumption of the year of the opening of road to be 2000. But the charts can also be relaxed to be used for any expected year of opening from present to the year 2010, due to a relatively small effect on design thickness of different years of opening. Therefore, these commercial vehicle flows represent the cumulative total number of standard axles, since the pavement design procedure, based on flows at opening, is linked to the assumed parameters of growth rate, design life and vehicle damage factor.

The advantage the use of cumulative total number of standard axles, in relation with the equivalent damage factor, in Indonesia is that it allows for the effects of unusual traffic loads to be allowed for a projected road. However, it requires the extensive survey of traffic in order to be able to identify types of vehicles and their axle loads for the purpose of determining the equivalent damage factor. Consequently, additional cost and time may be necessary to follow this procedure.

Since Indonesia is a developing country, there is a difficulty in predicting traffic growth, and thus the design life of a pavement, even for shorter design lives. With a limited pavement construction budget when a longer design life is proposed, this is a

serious problem. This has resulted in an alternative design technique for flexible pavements which is to build roads in stages rather than to provide a large initial cost.

As can be seen in nomograms for flexible pavement in Indonesia, the regional factor has been included to allow areas with different condition to be allowed for in pavement design.

The term regional factor in Indonesia is interpreted as the annual average rainfall, alignment (gradient), percentage of heavy vehicles and of stopping movements.

According to AASHO[1], many procedures have been used to estimate regional factors, and one or more of the followings are used in assigning a regional factor :

- Topography
- Rainfall
- Frost penetration
- Temperature
- Ground water table
- Subgrade type
- Engineering judgement
- Subsurface drainage
- Number of annual freeze-thaw cycles
- Steep gradient with large volume of heavy truck traffic
- Areas of concentrated turning and stopping movements

Essentially, Great Britain has also used one or more regional factors defined by AASHO. Unlike the Indonesian method, the standard does not consider areas of climatic condition that it should be according to annual rainfall data, based on Transport Statistics Great Britain 1992[12].

The regional factor used in Great Britain may be expressed in term of ground water table, which is used in assigning the subgrade type. Moreover, any pavement material within 450 mm of the road surface, including the subbase, must be chosen to be frost resistant in order to minimise the directly effect of climatic condition on the subgrade.

Additionally, the use of capping layer, instead of its function described previously, also lessens any reduction in the strength of the subgrade during wet whether[17].

It has been shown in chapter 3, for Indonesian design, that three types of alternative pavement thicknesses can be obtained once in designing a flexible pavement for a new road. This can be done by maximising the surfacing layer, or roadbase, or subbase.

Furthermore, the use of layer coefficients provides a choice in deciding the pavement thicknesses from the provided various pavement materials. These allow the engineering judgement to be used based on practical and economical consideration. Due to a limitation on pavement construction cost, economic judgement is usually applied.

CONCLUSIONS

Conclusions

1. These conclusions are a summary of the design procedures and the comparison section between flexible pavement design in the two countries, Great Britain and Indonesia.
2. The differences in designing flexible pavements for new roads can be summarised as the following :
   a. The basic approach underlying the design method of flexible pavement, in terms of structural failure and serviceability-perfonnance concepts.
   b. The assessment of traffic loading, in terms of the traffic growth rate, design life and damaging effect of vehicles.
   c. The term of regional factor being used in the two countries to allow the different environmental conditions to be assigned in pavement design.
3. The use of structural number as an index that may be converted to the thickness of various flexible pavement layers in Indonesia, and therefore, layer coefficients, provide a great deal of choice of various materials available to be used in the pavement design for new roads based on engineering judgement.
4. Indonesian design procedure allows an alternative design for flexible pavement for new roads, that is to build the road in stages, considering the traffic growth rate,

5. the design life and pavement construction cost.
6. The parameters used, moreover, design charts, in Great Britain provide a simple procedure of flexible pavement design for new roads. Additionally, the procedure has been prepared to allow the heavy trafficked road to be carried out.
7. Due to the variability of materials used and the conditions to build the road in flexible pavement designs for new roads Great Britain and Indonesia, it is therefore difficult to determine which of the methods considered is the most appropriate one.

Recommendations

1. Regarding areas of climatic conditions used in designing flexible pavements for new roads, the British design method should consider the inclusion of rainfall as a parameter, since Transport Statistic Great Britain shows that there is a significant different in rainfall data between England and Wales and Scotland.
2. Regarding the use of design charts in designing flexible pavement for new roads in Great Britain, that is valid only up to certain year, the British design procedure should consider the method in estimating the traffic as presented in the Advice Note HA 24/83, and also the use of the vehicle damage factor formula derived from dynamic weighbridge measurements, or more detail in TRRL LR 1132, in order to enable pavements to be designed for any years of construction.

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