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DESIGN OF FLEXIBLE PAVEMENT USING CEMENT-STABILIZED CRUSHED ASPHALT

Gatot Rusbintardjo

Department of Civil and Environmental Engineering, Faculty of Engineering,

Sultan Aging Islamic University (UNISSULA) Semarang – Indonesia

Email: gatotrsb@gmail.com

Recycling asphalt concrete pavements investigated by means of utilizing the asphalt and aggregates they contain, and solving waste disposal problems. Discarded asphalt concrete pavement materials of crushed asphalt from existing pavement studied. Laboratory testing showed that this material could be recycled successfully into quality pavement materials. This successful recycling accomplish by mixing the crushed asphalt with cement. Four different of cement content were utilized to obtain a reasonable mixing for pavement materials. Further, those four mixing of cement and crushed asphalt tested at the three different temperatures 10°C,30°C, and 50°C to be able toapply in the tropical country like Indonesia. To get workable mixture, a water-cement ratio 1 has chosen. Specimen with cement content of 3.75% by mass of crushed asphalt has reasonable properties to use as a road base of pavement and comparable to the other specimens who have cement content 6.25%, 7.50%, and 8.75%. Pavement recycled materials with cement were also found still have linear elastic behaviour which enable to be analyzed as a flexible pavement. Mechanistic design procedure used to analyze the pavement life and the future performance of the pavement analyzed by using TRRL design procedure. Successful analysis has demonstrated that cement-stabilized crushed asphalt is reasonable to use as a road base of the pavement and can applied in tropical country like Indonesia.

Keywords: Crushed-Asphalt, Stabilized, Road-Base, Pavement, Successfully

Introduction

The pavement system for which the thickness layer design will described herein consists of either a surface dressing or an asphaltic concrete layer covering a cement-stabilized crushed asphalt layer, an unbound granular base course, eventually an unbound sub-base course, and the subgrade as shown in Figure 1.

The purpose of the surface dressing is to protect the cement-stabilized crushed asphalt layer from disintegration (erosion) by traffic, to seal against the ingress of water, and to improve the skid-resistance properties of a surfacing. In addition, the purpose of the asphaltic concrete is as a wearing course to reduce the stress and the strain level in the cement-stabilized crushed asphalt layer in case the number of load applications is relatively high.

Utilization of cement-stabilized crushed asphalt as a sub-surface layer is to take another advantage from recycling of asphaltic concrete pavement materials. As has broadly known, recycling of asphaltic concrete pavements has received considerable attention because of increase on conservation of energy and mineral supplies. Each ton of asphalt paving material contains approximately 50 kg of bitumen and 950 kg of graded aggregate.

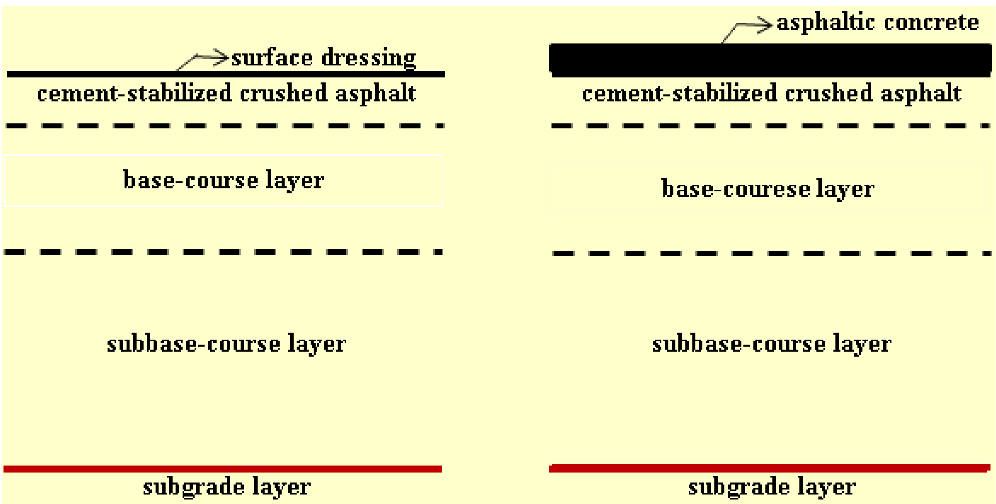


Figure 1. Layers of flexible pavement structure

Recycling of asphaltic pavement materials, which now discard as rubble and landfills, would not only conserve bitumen and aggregates materials, but would also solve a serious waste disposal problem.

One of the major processing problems associated with the recycling of asphaltic concrete encountered when reheating the material to temperatures necessary to remix and place the mixture without further oxidizing the bitumen. The second problem is that of rejuvenating of the old bitumen has generally been handled using soften agents. During mixing, the viscosity tends to reduce as well as the softening point. This together with an increased the penetration of the bitumen component needed to meet design specifications of new asphaltic concretes. One question that needs to be resolved concerning the use of rejuvenating materials is whether their softening effect has sufficient permanence to produce pavements whose service life properties are equivalent to those of origin bitumen or whether their effect will be of a shorter duration as the more volatile components within these additives are lost from the pavement.

In this paper, the pavement design for cold processing in-plant recycling will described, and in the process, Portland cement will used as a binder of the crushed-asphalt. The possibilities of such a cement-stabilized crushed asphalt to serve as a sub-surface layer in a road pavement structure will study. Since the old bitumen will processed in cold condition, Portland cement will used as a binder, the problems of reheating of crushed asphalt, and rejuvenating the bitumen will not encountered.

Several parameters that usually have used in pavement analysis will also used. These parameters fall into two categories:

a. Theoretically calculated or experimentally derived stresses, strains, or deflections, and
b. Pavement distress parameters such as cracking of bound materials, shear failure of unbound materials and permanent deformation or rutting.

Considering those pavement analysis parameters, mechanistic design procedures associated with stress-strain and deflection parameters and design procedures based on fatigue analysis will applied in this design of flexible pavements with a cement-stabilized crushed asphalt sub-surface layer.

To be able to use the results of this design in tropical area, some design inputs like air or pavement temperature and subgrade conditions will take according to the tropical area conditions. The pavement model and design criteria will discuss, and after that, experimental work and its results will describe and discussed.

Pavement Model and Design Criteria

Generally in design procedures the pavement structure is regarded as a three-layer system as illustrated in Figure 2. The lowest layer semi-infinite in the vertical direction represents the subgrade. The middle layer represents the base and/or sub-base layer. The upper layer represents the surface layer and very often the surface layer is divided into two layers where the top layer is called wearing course. These layers (subgrade, sub-base, base, and surface layer) generally considered to have complete friction between them and subjected to a standard dual wheel load.

The analytical solution to the state of stress or strain has several assumptions, they are:

1. The pavement structure regarded as a linear-elastic multi-layered system in which the stress solutions characterized by two material properties for each layer. They are Poisson's ratio μ, and the elastic modulus E.
2. Two material of each layer are homogeneous, that is, the property at point A, is the same at point B (Figure 2.).
3. Each layer is isotropic, that is, the property at a specific point such as A, is the same in every direction or orientation,
4. Each layer has a finite thickness except for the lower layer (subgrade), and all are infinite in the horizontal direction,
5. Full friction is developed between layers at each interface, as mentioned above,
6. Surface shearing forces are not present.

From the theory, can be seen (Figure 2.) that at a given point within any layer, nine stresses exist. These stresses are comprised of three normal stresses (σ_{zz},σ_{xx}, σ_{yy}) acting perpendicular to the element face and six shearing stresses ($\tau_{xy,}\tau_{yx}$, τ_{xz}, τ_{zx}, τ_{zy}, τ_{yz}) acting parallel to the face.

Static equilibrium in the element shows that the shear stresses acting on intersecting faces are equal. Thus, $\tau_{xz} = \tau_{zx}$; $\tau_{xy} = \tau_{yx}$; $\tau_{zy} = \tau_{yz}$. At each point in the system. There exists a certain orientation of the element such that the shear stresses acting on each are zero. The normal stresses under this condition are defined as principal stresses and are denoted by σ_1 (major stress), σ_2 (intermediate stress), σ_3 (minor stress).

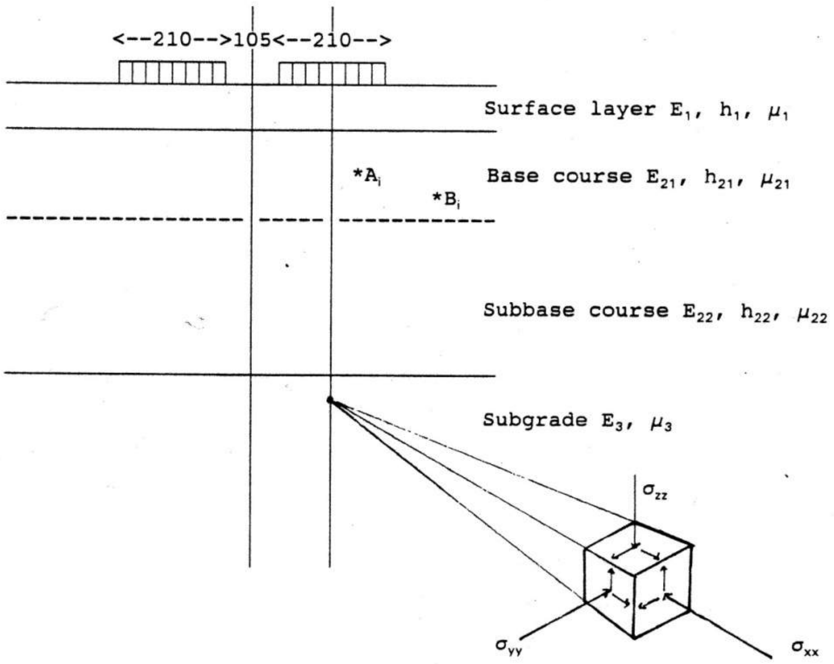


Figure 2. Generalized multi-layered elastic system

The bulk stress θ defined as the principal stresses at a point. Given the tri-axial state of stress of any element, the strains may compute by the following equations:

$$\epsilon_{zz} = \frac{1}{E}\left[\sigma_{zz} - \mu\left(\sigma_{xx} + \sigma_{yy}\right)\right] \quad (1)$$

$$\epsilon_{xx} = \frac{1}{E}\left[\sigma_{xx} - \mu\left(\sigma_{yy} + \sigma_{zz}\right)\right] \quad (2)$$

$$\epsilon_{yy} = \frac{1}{E}\left[\sigma_{yy} - \mu(\sigma_{xx} + \sigma_{zz})\right] \quad (3)$$

The points at the pavement structure at which strains used for design criteria are:

- The maximum horizontal tensile strain in the surface layer, generally at the bottom of the layer, and
- The vertical compressive strain at the top of the subgrade

Claessenet.al (1977), however has found that in some cases, with high modulus ratios between base and asphalt layer, the maximum horizontal asphalt strain is not at the bottom but higher within the asphalt layer. From extensive calculations, it has found for pavement structures with unbound base layers that the level at which the maximum horizontal asphalt strain occurs depend on the factor c:

$$c = h_1 \, x \, \frac{E_2}{E_1} \, mm \quad (4)$$

where: E_1 and E_2 are the elastic moduli (in MPa) of the asphalt and base layer respectively, and h_1 is the thickness of the asphalt layer in mm.

When c > 133 mm, the maximum asphalt strain occurs in the asphalt layer. In this case, when $h_1 \leq 200$ mm, the maximum asphalt strain is found located in the upper half of h_1. In this study, the c-value of the pavement structures is lower than 133 mm, so the maximum strain will occur at the bottom of the asphalt surface layer.

Asphalt strain criterion

Generally, the fatigue criterion for asphalt layers is, based on the permissible strain as a function of the number of strain repetitions and stiffness modulus of the asphalt. As mentioned above, the strain criteria for the asphalt layer is the maximum horizontal tensile strain. If this is excessive, cracking of the asphalt layer will occur.

The following formula shows the fatigue relationship of the asphalt concrete layer:

$$\log N_{fat} = -4.05 - 2.7\, x \log \varepsilon \qquad (5)$$

where: N_{fat} = the allowable number of repetitions of tensile asphalt strain ε_t.

This formula was derivefor asphalt pavements in tropical areas by Asgari (1992) from theoretical analysis of a large numberof pavements and relating to the results there of equation 5.25of the Highway design and Maintenance Standard Series (HDM 3) (William D.O. Peterson, 1987). That equation is:

$$TE_{cr2} = 0.0362\, SNC^{2.65}\, X\, e^{-0.143SY} \qquad (6)$$

where :TE_{er2} = expected (mean) cumulative traffic loadings at initiation of narrow cracking.

SNC = modified structural number

SY = $SNC^4/(1{,}000YE_4)$, provided that SY ≤ 8.

Asgari(1992) derived the fatigue characteristics formula into five probabilities to achieve allowable number of load repetitions i.e. 10%, 25%, 50%, 75%, and 90%. In the study, the relation will used that related to a probability of survival of 50%.

Subgrade strain criterion

The second design criterion as mentioned above is the vertical compressive strain at the top of the subgrade layer. If this excessive, permanent deformation will occur at the top of the subgrade and this will cause deformation or rutting at the pavement surface. The permissible compressive strain in the subgrade is shown as a function of the number of load application associated with a Present Serviceability Index (PSI) = 2.50.

Further, the allowable N_{fat} – value of the asphalt layer must compared with the N_{fat} – value of the subgrade in order to determine the lowest N_{fat} – value that has to be chosen for the structural design of the asphalt pavement or overlay.

The physical phenomena related to the value of N_{fat} can described as follows:

- if $N_{traffic} > N_{fat}$ of the asphalt layer, cracking of the asphalt layer will occur
- if$N_{traffic} > N_{fat}$ of the subgrade layer, rutting will occur.

Other criteria

Decisive stress

Besides horizontal strain at the bottom of the asphalt layer and vertical compressive strain at the top of the subgrade, stresses within the crushed asphalt, base and sub-base layer also may cause pavement failure.

In the crushed asphalt layer, the minimum and maximum horizontal tensile stresses that occur at the bottom of the layer will calculated by the equation:

$$\sigma_{1,3} = \frac{1}{2}\left[(\sigma_{xx} + \sigma_{yy}) \pm \sqrt{\left\{(\sigma_{xx} - \sigma_{yy})^2 + (2\,x\,\tau_{xy})^2\right\}}\right] \quad (7)$$

where:

$\sigma_{1,3}$ = normal stress (MPa)

σ_{xx} = horizontal stress in x-x direction (MPa)

σ_{yy} = horizontal stress in y-y direction (MPa)

τ_{xy} = shear stress (MPa)

If the maximum horizontal tensile stress exceeds the tensile strength of the cement stabilized crushed asphalt, cracking will occur and the pavement will fail. In the study, a non-symmetrical case used to analyze the stress distribution that is why the stresses in x – x and y – y direction applied instead of stresses in radial and tangential direction.

In the base and sub-base layer, fatigue strength of the pavement determined by the shear stress failure at the top of the layer. The shear strength obtained from the Mohr-Coulomb failure curve, where the normal stresses to draw the Mohr's circle calculated by the equation 8:

$$\sigma_{1,3} = \frac{1}{2}\left[(\sigma_{zz} + \sigma_{xx}) \pm \sqrt{\{(\sigma_{zz} - \sigma_{xx})^2 + (2\,x\,\tau_{xz})^2\}}\right] \quad (8)$$

where:

$\sigma_{1,3}$ = normal stress (MPa)

σ_{zz} = vertical stress (MPa)

σ_{xx} = maximum horizontal stress (MPa)

τ_{xy} = shear stress (MPa)

This shear stress failure criterion is dependent on the cohesion c and the angle of internal friction ø of the material of the base and sub-base. In this study, the shear stresses at top of the unbound base and sub-base will calculate.

The stress distribution in the unbound base and sub-base layers is dependent on the modular ratios of the individual layers and their thickness. On the other hand, the modulus of unbound materials is dependent on the state of stress. An estimate of the effect of stresses on the modulus of unbound layers can made using the rule of thumb developed by Shell researches that is:

$$E_2 = k \; x \; E_3 \tag{9}$$

where:

E_2 = elastic modulus of base or sub-base (MPa)

E_3 = elastic modulus of sub-base or sub-grade

k = $0.2\, h_2^{0.45}$ with limits 2 <k_2< 4

h_2 = base or sub-base thickness (mm)

Permanent deformation

Permanent or unrecovered deformation in pavements includes rutting, shoving, heave, small depressions, edge depressions, and large depressions. In this study will only consider rut depth progression or rutting, since rutting is historically a primary criterion of structural performance (related to traffic safety) in many pavement design methods.

Rutting caused by the traffic loading, post compaction, and plastic flow (of bituminous materials). Traffic loading causes deformation when the stresses induced in the pavement materials are sufficient to cause shear displacements within the materials.

There are many methods and equations to describe the development of rut depth. One that will use in this study is:

$$\log RD = \log a_c + b_c \, x \, llog \, n \qquad (10)$$

where:

RD = rut depth (mm)

n = equivalent number 80 kN standard axle load repetitions

a_c, b_c = rut depth coefficients

Rut depth coefficients a_c and b_c will be obtained by statistical calculations by using the least square line theory from some values of rut depth and number of load repetitions which are found from the calculations of pavement deformation in the subsequent pavement layers due to the traffic loading. This permanent deformation calculated by the equation:

$$u_p = u_{el} \, x \, a_m \, x \, n^{bm} \qquad (11)$$

where:

u_p = plastic deformation of the layer

u_{el} = elastic compression of a layer due to a 80 kN standard axle load

equal the difference between the deflection at the top of the

layer and the deflection at the bottom of the layer (under the

wheel load system).

a_m, b_m = permanent deformation coefficient

RD is the sum of plastic deformation, u_p, in all layers of the pavement structure, in this study, the maximum allowable rut depth taken as 18 mm and 35 mm (considered as maximum allowable rut depth in tropical area).

Experimental work

Materials

The crushed asphalt used in this study taken from an old asphalt pavement of Schiphol airport. The bitumen in this asphaltic material extracted with methylene chloride according to the ASTM test method D 1856-79. The percentage of the bitumen in the crushed asphalt appeared to be 4.4% mass. The particle size distribution of the crushed asphalt was also determined and the result shows that the grading found to be in accordance with the ASTM specifications for aggregate of asphalt mix. Only the percentage for the particles with diameter 19 mm and 25 mm are a little bit off from the ASTM grading limit. The other physical properties of crushed asphalt i.e. density and CBR values have also been examined and the results are given in Table 1.

The crushed asphalt stabilized with Portland cement class A. Since cement reacts with water, also, water used to mix cement with crushed asphalt, and the percentage of water or the water-cement ratio will explained in the next subsection.

Table 1: Properties of crushed-asphalt

Property	Value
CBR value	26%
Density	1993 kg/m^3
Bitumen content, % mass	4.4%
Bitumen data:	
Penetration at 25°C, 100gr	26

Softening Point Ring and Ball	58°C
Penetration Index (PI)	0.8

Tests, mix composition and preparation of test specimens

In order to obtain the pavement design parameters such as modulus of elasticity (E), tensile strength (σ_m), Poisson's ratio (μ), and visco-elastic characteristics of the cement stabilized crushed asphalt, two kind of tests were carried out into four types of specimens which have a different cement content. These two tests are:

- Indirect tensile test to determine the modulus of elasticity, the tensile strength, and Poisson's ratio
- Static creep test to determine the visco-elastic characteristics

In addition, a density test on each specimen also carried out.

The four types of specimens were called specimen type A, B, C, and D containing 3.75%, 6.25%, 7.5%, and 8.75% of cement respectively, which equals 75, 125, 150, and 175 kg cement per cubic meter respectively. In order to obtain workable mixes, a water/cement ratio 1 has chosen.

Since it decided to use indirect tensile test and the static creep test to obtain the design parameters, all specimens made according to the dimension of Marshall's specimen. Because of the relatively high water cement ratio, all specimens (except for specimen type A) were compacted by means of vibration compactor that usually used for concrete specimens. After compaction, the specimens cured in the room at room temperature (about 20°C) for 7-days before being tested. The 7-days strength generally used to obtain an initial indication of the strength of cement-stabilized materials.

Experimental results

The indirect tensile test is one type of tensile strength test used for stabilized materials. Most of the report of the test results has been for concrete or mortar, however, the test has conducted on cement-treated gravel, lime-soil mixtures, and asphaltic-stabilized materials. This test involves loading of a cylindrical specimen with a compressive load along two opposite generators. This results in a relatively uniform tensile stress acting perpendicular to and along the diameter plane of the applied load. This results in a splitting failure generally occurring along the diametral plane Yoder, E.J. and Witzczak, M.W. (1975).

In this study, the indirect tensile test was conducted at three different temperatures, 10°C, 30°C, and 50°C, while the static creep test was performed at standard temperature 40°C. The creep test is to determine the visco-elastic behaviour of an asphalt specimen. In the creep test, a constant force applied perpendicularly to the parallel end faces of a cylindrical or prismatic specimen. The specimen inserted between two load plates, one is fixed and the other, to which the load is applied, is moveable in the axial (vertical) direction. The deformation of the specimen in the axial direction, which occurs under influence of this load measured as a function of the loading time.

Discussion on the Indirect Tensile test results

Poisson's ratio

In this study, Poisson's ratio calculated with the formula (Van Gurp, 1990):

$$\mu = \frac{0.0673DR - 0.8954}{-0.249DR - 0.0156} \tag{12}$$

Where:DRis the ratio between horizontal and vertical displacement.

From the test, results it appears that if the ratio between vertical and horizontal displacements is higher than 10, very low values for the Poisson's ratio obtained. Poisson's ratio for cement-treated materials normally takes values between 0.10 and 0.25 (Yoder, E.J. and Witzczak, M.W., 1975). As shown in Table 2, from 36 specimens, 18 have very low Poisson's ratio, while the other 18 have a Poisson's ratio between 0.1 and 0.46. To calculate the modulus of elasticity the value of 0.25 used for all specimens. The influence of using fixed values for μ is limited, this means that taken the fixed values of μ will not to a very large extent influence the modulus of elasticity. As an example, specimen A2 test at temperature 10°C, the modulus of elasticity is 1926 MPa if it is calculated with μ = 0.6 (as measure) and is 1903 MPa if it is calculated with μ = 0.25, so the difference is only 1.2%.

Table 2: The values of Poisson's ratio calculated from the indirect tensile tests

Temperature	Specimens					
°C	A1	A2	A3	B1	B2	B3
10	0.25	0.06	0.31	0.15	-0.03	-0.04
30	0.34	0.33	0.32	0.15	0.21	0.10
50	0.46	0.12	0.37	0.28	-0.03	0.19
Temperature	Specimens					
°C	C1	C2	C3	D1	D2	D3
10	0.01	-0.50	0.15	0.03	0.03	-0.90
30	-0.02	0.03	0.06	0.03	-0.07	0.17
50	0.03	-0.70	0.40	0.25	0.04	0.04

Tensile strength (σ_m)

As can be seen in Figure 3, for all types of specimen, the tensile strength as obtained from the indirect tensile test, decrease with increasing temperature. A special comment has necessarily to given to the specimen type A. This specimen containing the lowest cement content shows the lowest dependency on the temperature, which means that the change in temperature only has a relatively small influence on the tensile strength. This is remarkable since this specimen contains the lowest cement content.

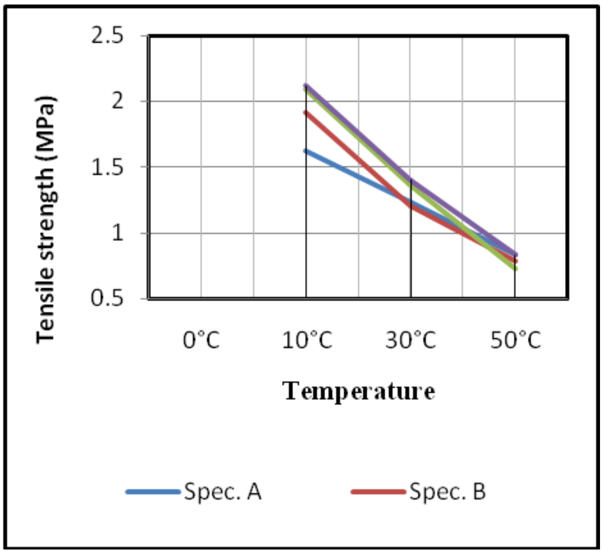


Figure 3: Relation between tensile strength and temperature

Modulus of elasticity

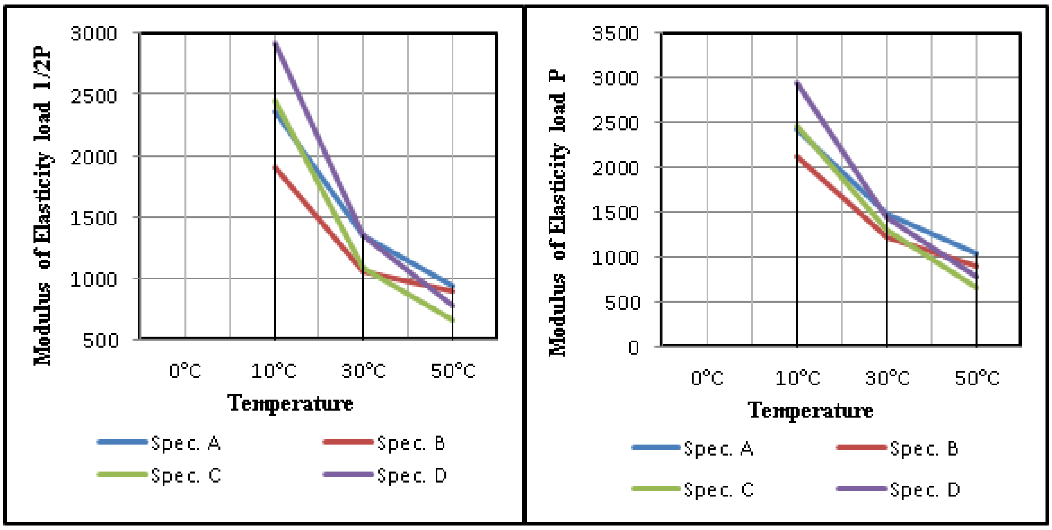
Similar to the tensile strength, the modulus elasticity also obtained from the indirect tensile test decreases with increasing temperature. The elasticity modulus calculated using the formula (Van Gurp, 1990):

$$E = [0.9974\mu + 0.2692]P/(\Delta h \; x \; t) \tag{13}$$

where:

μ	= Poisson's ratio
P	= maximum load until failure (kN)
t	= thickness of the specimen (mm)
Δ_h	= horizontal displacement (mm)

In this study, the modulus of elasticity also calculated with load ½ P because it expected that in phase materials still behave linear elastic. At failure load, materials are not any more in linear elastic phase. The test results for load P and ½ P given in Figure 4a and Figure 4b respectively.Again, the attention has to pay to the specimen A. The change in temperature does not cause the conspicuous change in the values of elasticity, especially for a temperature change 30°C to 50°C.



(a) (b)

Figure 4: Relation between modulus of elasticity E and temperature

(a) Load ½ P, (b) LoadP

Tensile strain

Based on the values of tensile strength σ_m and modulus of elasticity E, the tensile strain can calculate by using the equation:

$$\epsilon_t = 1/E\,(\sigma_m - \mu\, x\, \sigma_c) \tag{14}$$

where:

ϵ_t = tensile strain

σ_c = compressive strength which is assumed ≈ $3\sigma_m$

The test results of the calculation given in Figure 5.

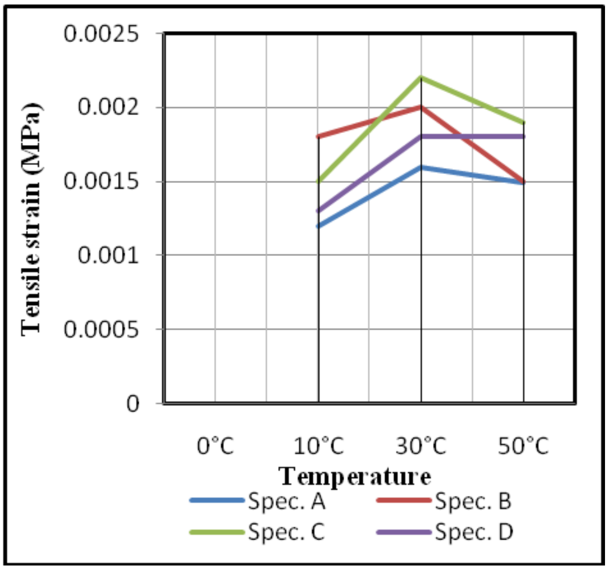


Figure 5: Relation between tensile strain and temperature

Explanation on the results of tensile strength and E-modulus

The tensile strength and the modulus of elasticity generally are dependent on the change in temperature, particularly in the temperature range from 10°C to 30°C. The values decrease with increasing temperature. This condition can explain as follows:

The water-cement ratio (wcr) of all specimens is 1 as have been reported before. This high wcr chosen in order to obtain workable mixes. As a result the strength of the mixture will become low even almost zero. The strength is decreasing if wcr is greater than

0.5. Nevertheless, the cement still has an ability to bind all the particles to become a relatively solid and stiff mixture.

Because of the low strength of cement, the values of σ_m and E merely influence by the bitumen and the ability of cement to bind aggregates. As the bitumen at high temperatures will become soft, the values of the tensile strength σ_m and the elasticity modulus E decrease with increasing temperature.

Discussion on the Static Creep Test results

The elastic, visco-elastic, and plastic strains obtained from the static creep test.The tests have executed at the standard temperature 40°C and static loading 0.1 MPa during 60 minutes.

The main conclusion that can draw from the test results is that the cement content does not influence to the elastic, visco-elastic, and plastic behaviour of the specimens. The averages values obtain from those parameters are almost the same for all specimens. The most important result obtained from this test is that all the materials still have elastic and visco-elastic characteristics, since in this study visco-elastic approach will used to solve the problem of pavement design against permanent deformation.

Design input

A very large number of parameters is involved in the structural design of parameters, these include environmental conditions (temperature, moisture), loading condition (axle load distribution, contact stress, contact area, loading time, dynamic effects) and material properties (modulus, permanent deformation, Poisson's ratio, failure criteria). Furthermore, most of parameters very considerably with time. To obtain a workable system for practical design, simplifications have to make.

1. Traffic

Traffic is representing in terms of the design number of equivalent standard axle loads to which the pavement will subjected during the design life. The pavement life in this design approach is the total number of equivalent standard axles 80 kN. Each standard axle is assumed to have two dual 20 kN wheels, each with a contact stress of 5.77 x 10^5 N/m^2 or 0.577 MPa, and radius of the contact area of 105 mm.

2. Temperature

Temperature generally has no significant effect on the elastic modulus of unbound materials but strongly influences the properties of bituminous materials as also has found in the tests on cement-stabilized crushed asphalt. For pavement design purposes, usually a 'weighted' Mean Annual Air Temperature (w-MAAT) is used which is derived from the Mean Monthly Air Temperature (MMAT). The w-MAAT related to an effective asphalt temperature and thus an effective asphalt modulus. The term "weighted Mean Annual Air Temperature," means that the effects on design of daily and monthly variations in the temperatures in the pavement have taken into account.

3. Subgrade

The performance of pavement structure is affect by the characteristics of the subgrade. Desirable properties of the subgrade include strength, drainage, ease of compaction, permanency of compaction and permanency of strength should fulfill.Four different types of subgrade will applied in the study ranging from a very poor subgrade with have CBR 2% and modulus of elasticity 20 MPA to a good subgrade with CBR 20% and modulus of elasticity 200 MPa.

4. Base and sub-base course

Some materials are considered to suitable as unbound base and sub-base material. They are coarse crushed masonry/concrete, coarse lava, crushed rock, crushed gravel, crushed stone, crushed slag or natural gravel.

Design of pavements

Pavement geometry

The pavement geometry shows in the Figure 6.The stiffness modulus of the cement-stabilized crushed asphalt layer was set at 1200 MPa being the mean test result of crushed asphalt with 3.75% of cement at a temperature of 35°C. The stiffness modulus unbound base and sub-base layers determined using the Shell relationship E_2 = k E_3 or the resilient modulus relationship $M_r = k\ \theta^{k_2}$. Using the Shell relationship where $k = 0.2\ h_2^{0.45}$, the results give in Table 3, together with modulus of elasticity of subgrade.

The stiffness modulus of the subgrade was set at four different subgrade conditions i.e. 20 MPa for very poor, 50 MPa for poor, 100 MPa for a moderate, and 200 MPa for a good subgrade. Based on those four different subgrade moduli, the pavement systems divided into four systems that are pavement system A, B, C, and D with subgrade modulus 20, 50, 100, and 200 MPA respectively

Loading conditions

As shown in Figure 16, the vertical contact pressure (which assumed to be uniformly distributed) is 0.577 MPa while the radius of the contact area is 105 mm. The stresses and strains will calculated at the top and the bottom of the asphalt layer, at the top and bottom of the crushed-asphalt, base and sub-base layers, and at the top of subgrade as shown by dot-black spot in the Figure 16.

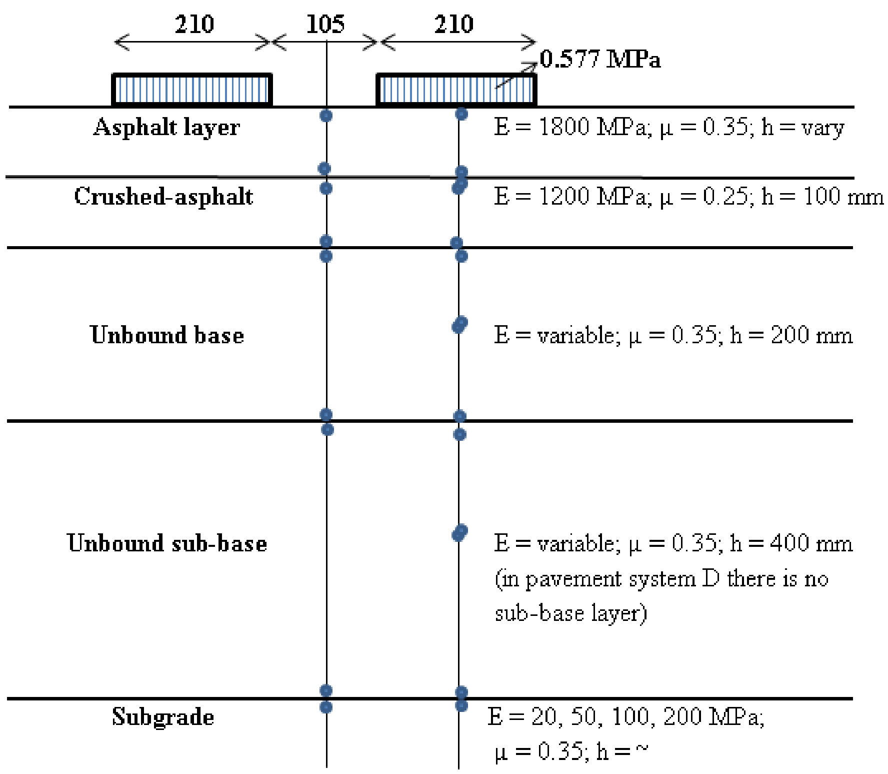


Figure 6: Characteristic of the pavement structure for 80 kN axle load

Table 3: Interim stiffness modulus values for base and sub-base

E subgrade (MPa)	E sub-base (MPa)	E base (MPa)
20	60	230
50	150	330
100	300	650
200	600	1300

Calculation Results

Based on the modulus elasticity of subgrade, there are four design pavement systems:

a. Pavement system A, B, and C:

Asphalt layer	E = 1800 MPa; μ = 0.35; h = 0, 50, 100, 150, 200 mm
Crushed asphalt	E = 1200 MPa; μ = 0.25; h = 100 mm
Base layer	E = 150, 375, 400 MPa; μ = 0.35; h = 200 mm
Sub-base layer	E = 60, 150, 300 MPa; μ = 0.35; h = 400 mm
Subgrade	E = 20, 50, 100 MPa; μ = 0.35; h = ~

b. Pavement system D:

Asphalt layer	E = 1800 MPa; μ = 0.35; h = 0, 50, 100, 150, 200 mm
Crushed asphalt	E = 1200 MPa; μ = 0.25; h = 100 mm
Base layer	E = 400 MPa; μ = 0.35; h = 200 mm
Subgrade	E = 200 MPa; μ = 0.35; h = ~

Those four pavement systems calculate based on three design parameters, cracking of asphaltic concrete layer, cracking of the crushed-asphalt layer, and subgrade deformation.

1. Cracking of the asphaltic concrete layer criterion

For all pavement systems, having an asphaltic concrete surface layer the horizontal tensile strain was the lowest for thin asphalt layer (50 mm). In the pavement system A there are no horizontal tensile strain in such a thin asphaltic concrete layer.

For pavement systems having an asphaltic concrete layer of 50 mm, the lowest value of the horizontal tensile strain was 0.102 x 10^4 at pavement system B and the highest 0.254 x 10^4 at pavement system D.

For an asphaltic concrete thickness 100 mm, the horizontal tensile strain in pavement system A and B is lower than tensile strain for an asphaltic concrete thickness 150 mm, while for systems C and D the horizontal tensile strain for an asphaltic concrete thickness 100 mm is higher than for an asphaltic concrete thickness 150 mm. Meanwhile for all pavement systems with an asphalt layer of 200 mm, the horizontal tensile strain is lower than with an asphaltic concrete thickness 150 mm or 100 mm.

This has led to the conclusion that the horizontal tensile strain at the bottom of the asphaltic concrete surface layer is not only dependent on the thickness and its stiffness modulusbut, also on the stiffness modulus of the layers underneath. By using the characteristic fatigue equation for asphaltic concrete in tropical conditions:

$$\log N = -4.05 - 2.7\, x \log \epsilon_h \qquad (15)$$

The horizontal tensile strain also results in the highest number of allowable load application N in all pavement systems having a 50 mm thickness of asphaltic concrete surface layer.

2. Cracking of the crushed-asphalt layer criterion

The stresses life at the bottom of the crushed-asphalt layer used to determine the fatigue strength. From the radial and tangential stresses, calculations the greatest (principal) tensile stress can found by using the equation:

$$\sigma_{1,3} = \frac{1}{2}\left[\left(\sigma_{xx} + \sigma_{yy}\right) \pm \sqrt{\left\{\left(\sigma_{xx} - \sigma_{yy}\right)^2 + \left(2\,x\,\tau_{xy}\right)^2\right\}}\right] \tag{16}$$

where:

$\sigma_{1,3}$ = normal stress (MPa)

σ_{xx} = horizontal stress in radial direction (MPa)

σ_{yy} = horizontal stress in tangential direction (MPa)

τ_{xy} = shear stress (MPa)

3. Subgrade deformation criterion

There are no special comments to the results of the calculations based on subgrade criterion. For all pavement systems the thickset asphaltic concrete layer, the lowest it compressive strain. The lowest vertical compressive strain is 0.105 x 10^{-3} occurring at pavement system C having an asphaltic concrete layer of 200 mm, and the highest is 0.590 x 10^{-3} occurring at the pavement system A having an asphaltic concrete thickness 50 mm or Having crushed-asphalt as a surface layer. By using the fatigue formula for the subgrade deformation:

$$N = \left[{}^{0.028}/_{\epsilon_v}\right]^4 \tag{17}$$

these compressive strains result in the lowest value of the number of allowable load applications of 1.70 x 10^6 for pavement system A having crushed-asphalt as a surface layer, and the highest is 5.06×10^9 for pavement system C having 200 mm thick of asphalt concrete surface layer.

Pavement life

The final calculation result for those three design criteria, so the allowable number of equivalent 80 kN standard axle loas N is the lowest of the three. The following comments have to make:

1. The numbers of allowable load applications (N) found in the pavement systems B, C, and D for structure with a crushed asphalt surface layer and asphaltic concrete layer thickness of 50 mm show unrealistic values.
2. Except for pavement system D, all pavement systems having an asphaltic concrete thickness of 50 mm, cracking of the crushed-asphalt layer is the dominant design criterion.
3. For all pavement systems having an asphaltic concrete thickness of 100, 150, and 200 mm, except for pavement system A and B having an asphaltic concrete thickness 100 mm, the asphalt criterion is dominant. The lowest N-value is 7.02×10^6 and the highest is 5.06×10^9 for pavement system A in asphaltic concrete criterion and for pavement system C in subgrade criterion respectively.

Conclusions

The results of this study have led to the following conclusions:

1. With only 4% by mass of crushed-asphalt, cement can used to stabilize crushed-asphalt that is reasonable to use as a road base of the pavement.
2. Cement-stabilized crushed asphalt is not reasonable to use as a road base layer of a flexible pavement structure over a very poor and poor subgrade if only covered with seal coat or 50 mm of asphaltic concrete wearing course. For goo subgrade and thicker asphalt layer however, cement-stabilized is reasonable to use.
3. The rutting criterion seems to be controlling the pavement life if the allowable rut depth is 18 mm. If 35 mm rut depth allowed, only in the pavement systems B, C, and D having a surface dressing or 50 mm asphaltic concrete wearing course, rutting is the controlling factor on pavement life. In the other pavement structures, the pavement life determine by either the asphaltic concrete criterion or crushed-asphalt criterion.

References

Asgari (1992) “Comparison of Mechanistic and Empirical Methods to Predict the Deterioration of Asphalt Pavements”. M.Sc. Thesis of Transportation and Road Engineering in Developing Countries (TREND) Course – IHE – Delft University of Technology the Netherlands 1992.

Claessen, A.I.M., Edwards, J.M., Sommer, P., and Uge, P. (1977) Asphalt Pavement Design ‘The Shell Method”. The University of Michigan, Proceedings vol. 2 page 39 of the 4th Internationa Conference on Structural Design of Asphalt Pavement – August 1977

Shell Pavement Design Manual (1978). Asphalt Pavements and Overlays for Road Traffic. Shell Petroleum Company Limited – London 1978.

Van Gurp (1990) Test Materials for Road.Delft University Technology Publication.

William D.O. Peterson (1987). “Road Deterioration and Maintenance Effects”.Models for Planning and Management. The Highway Design and Maintenance Standard Series, A World Bank Publication. Published for the World Bank by the Johns Hopkins University Press, Baltimore and London.

Yoder, E.J., and Witczak, M.W. (1975).Principles of Pavement Design. Second Edition, A Wiley – Inter-science Publication. John Wiley & Sons, Inc. New York, London, Sydney, Toronto 1975.