Behavior in Stress and Deformation Sands for Asphalt Mixes

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Abstract. For asphalt mixes, the visco-elastic model proposed initially by Huet and Sayegh predicts very accurately laboratory complex modulus test results. It gives by semi-analytical calculations relatively good stress and strain fields for heavy traffic pavements specially for base course made with classical materials. On the contrary, for flexible pavement with low traffic, for high temperature gradients and for the analysis of damages under slow heavy loads, it is necessary to take into account the visco-elastic behaviour of asphalt materials. In this paper, a model for a semi-infinite multilayer structure taking into account the visco- elastic constitutive relation is presented. The main interest of this method compared to the two others is the very short computational time for a multilayer structure. The importance of developing an analysis of the flexural behavior of beams is related firstly to the use of beams as a basic element in the realization of structures, and also to characterize the mechanical properties of laminates and sandwich materials from bending test performed on specimens shaped beams. This study examines the mechanical behavior of a bituminous material quasi-compact. It aims to develop a pattern of behavior meets the requirements for industrial exploitation. Experimental responses show a behavior similar to that of concrete, namely the asymmetry (difference between tension and compression). Only the normal stress is taken into account. Although for different layers, the normal distribution of the stress is linear and is based only on the depth of the beam.

Keywords: Mechanical Behavior, Distribution of Stress and Strain, Bituminous Material.

1. Introduction

In recent years, road transport, both for passengers and freight, have grown significantly with the result that traffic is much faster and more secure. Much research has been conducted to improve the speed with which tracks are designed to withstand speeds of vehicles that are increasingly high variability of the mechanical properties of pavement materials is considered to be maintained within relatively narrow for materials developed and implemented in accordance with the standards and guidelines, the only factors taken into consideration to account for the variability in the appearance and development of pavement deterioration are the dispersions on: the results of fatigue tests The thickness of the layers at runtime [1,2,3,4]. The use of modified bitumen or upgraded for this project is justified by the three main reasons: The modified bitumens are recommended for highways with high traffic and axle load important because they increase the resistance to rutting and cracking while providing a better resistance to aggressive weather conditions (precipitation and temperatures).

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- The use of modified bitumen reduces the cost of maintaining and operating the long term because of the high performance and extending the life they give the pavements.
- This last point is extremely important for the administration as the management costs of the highway in the long run can be very exorbitant if proper techniques are not used.

To design bituminous mixes for high resistance to rutting it is necessary to have good tests to characterize this property,
i.e. capable of realistically reproducing the effects of all composition parameters that are relevant to rutting resistance:
binder consistency at high temperatures, possibly binder elasticity, and the shear resistance of the mineral skeleton. This is not true of all the tests that have been used up to now, and even of some which have been used almost universally. The characteristics of a good test for resistance to permanent deformation appear to be as follows: it should be a repeated loading test (to reproduce the effect of elastic binders); it should involve interparticle friction (if it is a compressive test it should be triaxial, if it is a shear test the effect of dilatancy should be mobilized); it should be performed under realistic conditions of temperature and loading time; and the load applied should result in realistic permanent deformation. The test should be made on a bituminous material in a condition as close as possible to that which it will have in a pavement when it is prone to rutting.
This means that not only its voids content, but also the structure of the mineral skeleton, the distribution of the binder on a microscopic scale and the properties of the binder must be representative. The choice of laboratory mixing and compaction methods is, therefore, very important.
Because of the nature of road materials and the impossibility to accurately predict the values of the external parameters determining their performance (particularly traffic and climatic conditions (Table 1) over a long period (pavement service life), models to predict the various types of distress are bound to be statistical [5,6,7,8].
From a practical point of view, this means that the solutions to which these models may lead involve a certain risk factor.

Table 1: Main external factors determining the resistance of bituminous pavements to permanent deformation.

TRAFFIC	- Frequency of occurrence of loads - Number of repetitions - Distribution across the carriageway
LOADS	- Vertical action (weight, contact pressure, radius of tyre-road contact area) - Tangential action - Time of application (depends on speed)
CLIMATE	- Temperature pattern - Moisture content regime

2. Characterization of mechanical properties of asphalt

In the literature the bituminous mixtures are considered homogeneous materials, isotropic, visco-elastic, linear and thermo-sensitive. Note that the characterization of the rut, not addressed here, asked to characterize the asphalt concrete by elasto-visco-plastic in the case of large deformation, high temperature and slow loading. However the introduction of layers of asphalt creates a density gradient with depth after compaction, and directs a privileged aggregate. In addition the work of show that testing tension / compression on cylindrical specimens cored using different orientations, show differences of 20% on the results of a sample orientation to another [9,10,11,12].
Therefore the samples are taken to heart the material to be found in conditions close to the assumption of isotropy. Laboratory and is therefore trying to make withdrawals and impose stresses that correspond to the direction of greater distortion.
The modulus measurements of bituminous materials are used in their linear range. This therefore requires application of "small deformations". The assumption of a linear viscoelastic behavior of asphalt is accompanied by the validity of the Boltzmann superposition principle [13,14,15,16]: the response of a material to a request made by a number of requests is the sum of elementary responses to each of these basic demands [17,18,19,20].

3. The structure of roadway

The ground-layer unit of form represents the platform support of the roadway. The sub grade has a double function. During work, it ensures the protection of the ground-support, allows the quality of leveling as well as the circulation of the machines. Then the base course and the sub-base come thus forming the underlay's, which bring to the roadway the mechanical resistance to the vertical loads induced by the traffic and distribute the pressures on the platform support in order to maintain the deformations on an acceptable level.

Lastly, the surfacing is composed of the binder course and wearing course possibly between the latter and the underlays. It has two functions, on the one hand, it ensures the protection of the underlays with respect to the water infiltrations and on the other hand it confers to the users an all the more satisfactory comfort of control as the characteristics of surface are good.
The use of the modified bituminous mix is effective means to increase rigidity and makes it possible to have a better fatigue strength and to rutting. This novel method makes it possible to minimize the thickness of the layers (used in the EAST-WEST Highway schematized in figure 1).

Fig. 1: Roadways with multi-layer structure used for the EAST-WEST highway

4. Stresses and strains within the structure

The mechanical strength of asphalt concrete in tension is less than one tenth of that in compression. Different types of tests can be done to characterize the behavior of the material under tension. However, this technique is to set up a heavy and requires specific equipment both in the press of the specimen. The level of normal strain in the layer can be different depending on the location of the neutral surface of the beam. The neutral surface is the surface where strain/stress is zero. The strains/stresses below the neutral surface and above the neutral surface have opposite signs (Figure 2). In pure bending, the normal strain (ε_x) and normal stress x (σ_x) in x-direction can be expressed as, respectively,

$$\sigma_x = E_i\,\varepsilon_x = -E_i\kappa z \qquad Eq.(1)$$

$$\varepsilon_x = -\frac{z}{R} = \kappa z \qquad Eq.(2)$$

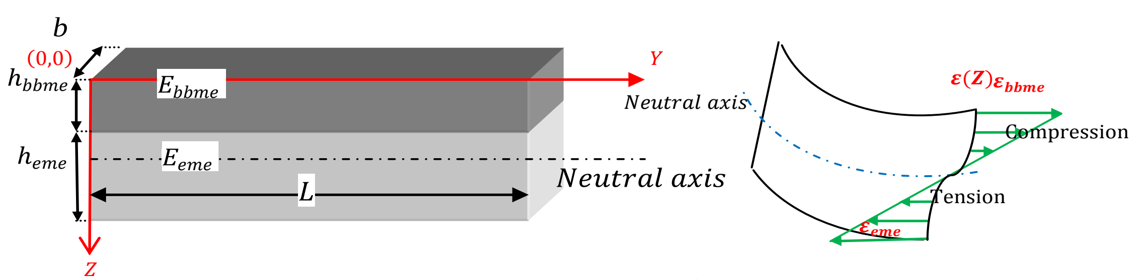

Fig.2: Section and normal distribution of constraint

Where

$\kappa = \frac{1}{R}$ is the radius of curvature of the beam due to bending, R is the radius of curvature, E_i is the Young's modulus, and z is the distance on z-coordinate from the neutral surface. For small deflections of the beam compared to the length of the beam, the radius of curvature can be determined from the following equation (assume the beam is under pure bending, that is the x-direction displacement.

$$D_x(x,t) = z\frac{\partial w(x,t)}{\partial x} \qquad Eq.(3)$$

y direction déplacement

$$D_y(x,t) = 0 \quad \text{and} \quad D_z(x,z,t) = w(x,t)) \qquad Eq.\ (4)$$

$$\frac{1}{R} = \frac{\frac{\partial^2 w(x,t)}{\partial x^2}}{\left[1+\left(\frac{\partial^2 w(x,t)}{\partial x^2}\right)\right]^{3/2}} \qquad Eq.\ (5)$$

If the beam is slightly bent, then $\frac{\partial^2 w(x,t)}{\partial x^2} \ll 1$, Equation (5) can be approximated as:

$$\kappa = \frac{1}{R} = -\frac{\partial^2 w(x,t)}{\partial x^2} = -\frac{\partial w(x,y,t)}{\partial y} = -\frac{M}{E_i I} \qquad Eq.\ (6)$$

where

$$M = -\iint_S z\sigma_x dS = -\kappa E \iint_S z^2\, dS = -\kappa E_i I \qquad Eq.\ (7)$$

$$et\ I = \iint_S z^2\, dS \qquad Eq.\ (8)$$

5. Numerical studies of the behavior of the structure

- Regarding this work, the following materials were chosen for our beam: The High Modulus Asphalt Concrete (BBME 1) in surface layer;
- The High Modulus Coated Material (EME 2) for the base layers.

The use of BBME 1 and EME2 respectively for surface layers and base layers aims to improve the rutting resistance of the binder with respect to the BBME and resistance to fatigue the base and foundation regarding the EME. According to French standards and with the experience known in France and elsewhere, or that kind of material is used, an asphalt of a module has high rigidity can be achieved by using a hard bitumen, the use of a modified bitumen, or by the addition of additive and with the aim of increasing the module. The use of modified asphalt or bitumen with pure additives, and although it increases the initial cost of construction of the road, will be used to significantly reduce maintenance costs and maintenance of the road is long term, because the duration of service of the roadway is increased. The disk is coated with bitumen is usually 10/30 with proven performance in Europe. However the supply of bitumen 10/30 is deficient in any project area, therefore, it is unrealistic to use the bitumen 10/30 for some paid. Nevertheless, we can achieve the performance of bitumen 10/30 with 35/50 by adding a suitable additive [21,22,23,24].

The technique of high modulus asphalt is already old and the first job on the road dating back some thirty years. But over the last ten years the job is developing rapidly in some countries, which use them for a significant proportion of maintenance pavement surface with heavy traffic, sometimes to impose them on the asphalt highway in- beyond a certain level of traffic as the case of the Algerian East-West Highway. Other countries are on the contrary very reserved.

6. Rutting versus thermal fatigue

Improving the granular structure of bituminous mixes has been the basic principle in taking measures against rutting. All provisions tending to reduce bitumen or filler content have turned out to be more harmful than useful. We are trying to increase mastic (i.e. bitumen + mineral filler) (OECD) content and to improve the properties of binders, in order to optimize the durability of bituminous materials. Thermal fatigue is part of our concern (Figure 3).

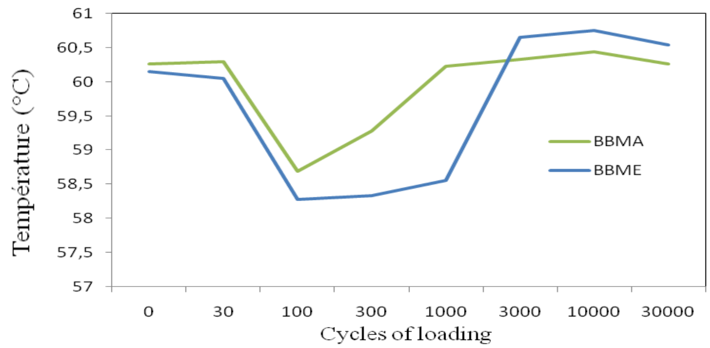

Fig. 3: Effect of the loading cyclic on the temperature

For heavily trafficked roads, modified binders are used and/or fibre additives are incorporated. These layers also have good resistance to skidding at high speed, and resistance to flow rutting is not required to the same extent with these very thin layers as with thick layers.

Now for rut prediction, the sensitivity of rut depth to the relevant external parameters is much higher. Thus S. Huschek [25,26,27,28] has shown that in the summer of 1976 the average air temperature in Switzerland was for sixteen consecutive days 6°C above the long-term average (1901 to 1960) [29,30,37,32] , and that this difference affected rut depth prediction by a factor which could range from 2.3 to 3. This tallies completely with the results presented in figures 3, 4 and 5 showing the sensitivity of rut depth to the stiffness of a bituminous mix, which is very susceptible to temperature especially in the high temperature range. This has led S. Huschek to draw the following conclusion: “In the light of these difficulties, the accuracy of prediction of the absolute value of rut depth is rather limited. However, in the practical situation a relative viewpoint is of primary importance. With a relative viewpoint one can choose the best of the available possibilities, independently of the largely unknown “future” parameters such as traffic loading and temperature conditions. I think we should not always speak about satisfying or even good correlation between predicted and measured values. We should speak more often about the risk of every real prediction and express the accuracy or, better, the lack of accuracy, in statistical values”.

The practical use of models to predict the absolute value of rut depth is, consequently, rather limited. The best way to mitigate flow rutting in bituminous mixes is to minimize the risk by using bituminous mixes that are little sensitive to permanent deformation.

This can be achieved by an appropriate choice of compositions based on laboratory tests, and by a correct design of the underlying courses.

The long-term strategy for bituminous courses is to alternate a very thin and a thick layer in time:

- After eight to ten years, a very thin course is laid to improve skid resistance and imperviousness. This layer has no structural effect;
- After another eight to ten years, 6 to 8 cm of dense mix with a high content of coarse aggregate is laid to improve the structural capacity of the pavement.

7. Mechanical tests

The test for resistance to rutting is carried out with the LCPC wheel tracking tester (Figure 4).

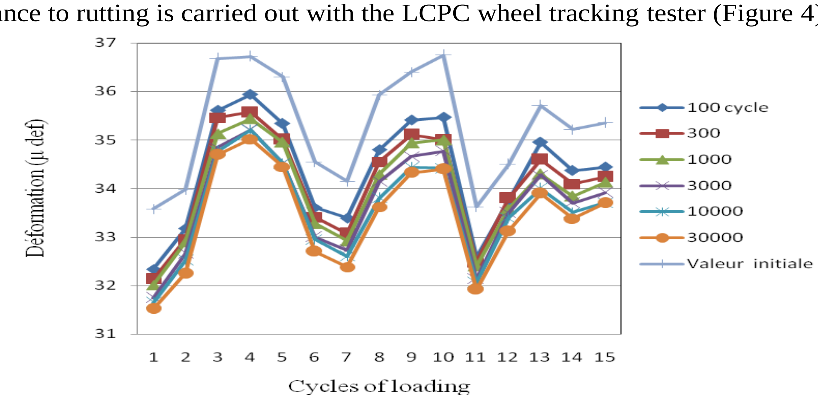

Fig. 4: Influence of the cyclic load on the percentage of the vacuums

This standard test (NFP 98-253-1) consists of subjecting a slab of bituminous material 500 mm long, 180 mm wide and 100 or 50 mm thick to the action of a wheel (with a smooth tread tyre inflated to a pressure of 0.6 MPa and loaded to 5000 N) rolling to and fro at the rate of one cycle per second. The test is performed in a thermostatically controlled enclosure, at the temperature specified in the relevant standard for bituminous mixes (generally 60°C). The log/log graphic representation of percentage of rutting against number of cycles is generally a straight line for a material that performs well for rutting [33,34,35,36].

This test is an integral part of the French methodology for bituminous mix design, and is the subject of specifications for each category of standardized bituminous mixes to ensure their resistance to rutting. For example, rut depth in a bituminous concrete wearing course mix with medium-sized coarse aggregate must not [37,38,39,40] exceed 10 mm after 30,000 loading cycles in a test performed at 60 °C.
The test is sensitive to variations in characteristics and nature of mix constituents.
This is illustrated by figures 5 and 6.

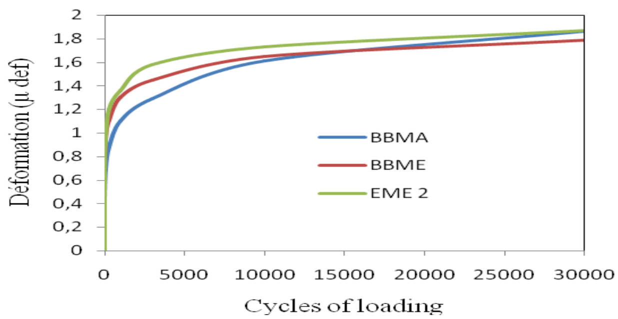

Fig. 5: Deformation with rutting according loading

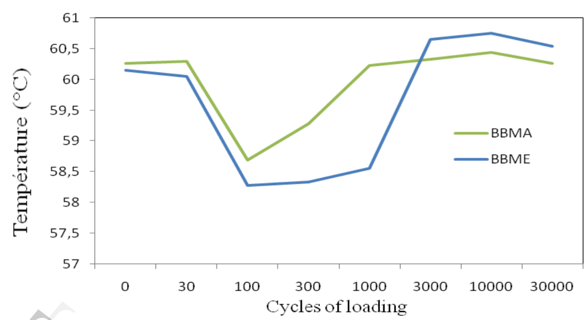

Fig. 6: Effect of the loading cyclic on the cyclic to the temperature

8. The importance of specimen preparation for predicting the field performance of mixes

Any mechanical test to assess how a bituminous mix will perform in the field when subjected to the combined action of loads and climatic factors involves the preparation of specimens. If the test results are to guarantee success, these specimens must correspond as largely as possible to the mix as it will actually be laid. For a given mix design this correspondence can only be achieved if the method of compaction in the laboratory (Figure 7), which differs from that used on site (except for accelerated full-scale tests), leads to the same arrangement of the mix constituents as obtained by field compaction.

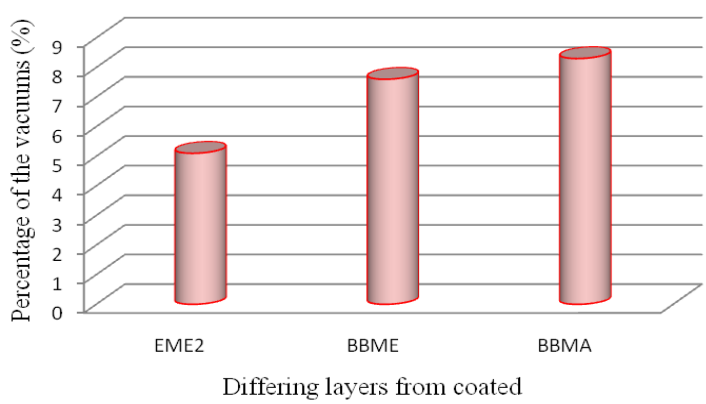

Fig. 7: Percentage of the vacuums for differing layers from coated

The law of permanent deformation versus number of loads is established by way of experiment for various stress conditions and these laws are assigned to various points in the structure of the pavement, where the stress condition has been calculated using a pavement model. One thus passes on directly from the calculated field of stresses in the structure to permanent deformation.

This means that a mechanical test cannot be dissociated from the way specimens are prepared and that, since the arrangement of mix constituents comes into play, mix designers should consider volumes and not masses (volume occupied by the aggregate, by the mastic, by the binder; volume of voids in the aggregate; volume of voids in the compacted mix; etc.) [41,42,43,44].
Because of the importance of the concept of composition by volume, the main relevant formulas may be reminded here:

$$V\ (\%) = 100\ \frac{Vb - Vm}{Vb} \quad Eq.\ (9)$$

where:
V (%) = percentage of voids in the specimen,
Vb = bulk volume of the specimen,
Vm = volume occupied by the same mass without voids.

On the other hand, a bituminous mix rich in aggregate and voids and poor in binder will have a high rutting resistance (good stability) but a low fatigue strength (poor durability) Figure 4.
The results of this test have the additional advantage of providing information on mix stability: if the percentage of voids is too low after ten revolutions, the mix is bound to be unstable. Thus it has been specified for dense bituminous mixes that voids content should be higher than 11% after ten revolutions.

9. The complexes module E*

9.1. Deflection test 2 points

The deflection test is standardized according to standard NF P 98-261-1. It is used for the pavement design in France (LCLC and SETRA 1998) (Young f & al. 1998). Like all the deflection tests, it is about a no homogeneous test.
The trapezoidal test-tube is embedded at its great base and is requested at its top (figure 3). The requests can be exerted in force or displacement.
The trapezoidal shape of the test-tube is selected to obtain a maximum deformation apart from the zone of embedding of the sample requested in beam comforts. The rupture is generally carried out in the vicinity of the 1/5 the total height (H) of the test-tube. In experiments a great disparity exists on the values this height. According to Boding [45,46,47,48] they can be included/understood between h/10 and h/2 account held of the heterogeneity of material.
The standard of the complex module, |E*|, is an indicator of the rigidity of the material, whose knowledge makes it possible to better include/understand the burden-sharing in the structure of the roadway. In addition, the phase gives an idea on the behavior of the bituminous mix and varies 0° for a purely elastic material with 90° for a purely viscous material.
The bitumen binder confers on the bitumen binder its viscoelastic behavior. Consequently, the frequency of request and the temperature condition its behavior. The mechanical behavior is regarded as viscoelastic linear when the deformations are weak. The viscoelastic character of the bitumen binder is expressed by the module complexes E*(ω) where the real part (Er) represents the elastic component of the relation stress-strain whereas the imaginary part (Ei) internal excitation reflects the viscous component of this relation. Mathematically one can write:

$$e^{i} = \cos\varphi + i \sin\varphi. \qquad \text{Eq. (10)}$$

Consequently:

$$|E^*|\, e^{i} = |E^*| \cos\varphi + i\, |E^*| \sin\varphi \qquad \text{Eq. (11)}$$

The complex module is then:

$$E^*(\omega) = E_r + iE_i = |E^*(\omega)|e^{i\varphi} \quad \text{ou} \quad E* = E_1 + i\,E_2 \qquad \text{Eq. (12)}$$

$$\text{With: } E_r = E_1 = |E^*| \cos\varphi \qquad \text{and } E_i = E_2 = |E^*| \sin\varphi \qquad \text{Eq. (13)}$$

Where ω is the angular frequency;
i: the imaginary part of a complex number (i^2 = -1);
On the one hand, the dynamic module |E* (ω)| is defined as being the module or the standard of the complex module as expressed by the following relation:

$$|E^*(\omega)| = \sqrt{(E_r)^2 + (E_i)^2} \qquad \text{Eq. (14)}$$

In addition, the angle of dephasing φ is given by:

$$\varphi = tang^{-1}\left(\frac{E_i}{E_r}\right) \quad Avec\ tang^{-1} = Arctang \qquad \text{Eq. (15)}$$

9.2. Maîtresse curve

After having traced the isotherms of the standard of the module complex and established the factors of translation (Figure 8), the main curve at the temperature of reference of 15°C was worked out and is presented to the Figure 8-b.

We tested 8 test-tubes at temperatures of 15°C and 20°C (to see figures 8 (a and b)), and at a frequency of 10 Hz and subjected to a force throughout one 2 minute.

For EME 2 we have an average apparent bulk density of the four test-tubes of 2,384 g/cm3, corresponding to an average porosity of 3, 5% (MVRe declared = 2,471 g/cm3).

Standard NF P98-140-1999 indicates that the test-tubes intended to be tested for the EME 0/14 classifies 2 must have a porosity ranging between 3% and 6%.

In the same way the test-tubes of the BBMA were tested under the same conditions as those of EME2. The average apparent bulk density obtained of the four test-tubes is of 2,320 g/cm3, corresponding to an average porosity of 6, 5% (MVRe declared = 2,480 g/cm3).

Standard NF P98-141-1999 indicates that the test-tubes intended to be tested for the BBME 0/10 classifies 1 must have a porosity ranging between 5% and 8%.

The module of rigidity of the EME 0/14 classifies 2 made up of aggregates coming from Tlemcen- Tizi and of pure bitumen 40/50 of Alma Petroli Spa and additive AQ-1 was given using the deflection test two points on trapezoidal test-tubes.

The median values of the module of rigidity to 15°C and 20 °C, for a frequency of 10Hz are of 14503 MPa and 12207 MPA respectively.

This bitumen binder EME 0/14 classifies 2 presents a module of rigidity in conformity with standard NF P98-140-1999 (≥14 000 MPa with 15 °C) and satisfy the criterion with design (≥11 000 MPa with 20 °C).

As well as the module of rigidity of the BBME 0/10 aggregate classe1 made up coming from Tlemcen- Tizi and bitumen Alma Petroli Spa 40/50 was given using the deflection test in two points on trapezoidal test-tubes.

The median values of the module of rigidity to 15°C and 20°C, for a frequency of 10Hz are of 11424 MPa and 9118 MPa respectively.

This bitumen binder BBME 0/10 classifies 1 presents a module of rigidity in conformity with standard NF P98-141-1999 (≥9000 MPa with 15 °C) and satisfy the criterion with design (≥6000 MPa to 20 °C).

The E_1 component represents the real part of the module complexes and makes it possible to quantify stored elastic energy; E_1 is the reversible module associated with the elastic part of material. In addition, the E_2 component quantifies the energy dissipated by internal friction under the effect of a request. This dissipated energy is transformed into heat and will increase the temperature within a test-tube subjected to a cyclic loading. It is the viscous character of the material which is at the origin of this specific property (Salençon, J. 1983). E_2 is the irreversible module, the dissipated module.

This capacity to dissipate energy of viscoelastic materials will play a big role as far as the fatigue strength of the bitumen binders which can be given starting from test-tubes subjected to the repetition of many cycles of loading (Di benedetto h. 1998 & Di benedetto h, Soltani a, Chaverot p. 1996). Although the level of request is low, the cyclic effect induced an increase of the temperature within the test-tube and modifies the rheology of the bituminous mix in the course of tests. The standard of the complex module of a bituminous mix decreases with the increase in the temperature. The evolution of the rheological properties in the course of fatigue test thus requires to be taken into account in the analysis of the results.

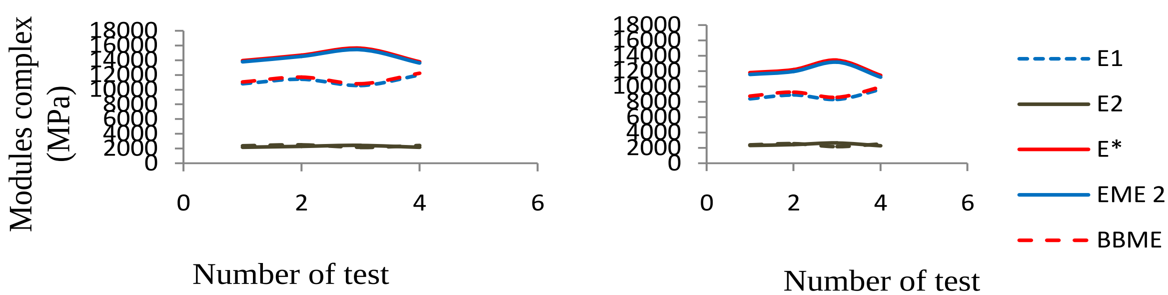

Fig. 8: a- Main curve of the bituminous mix tested (Temperature of reference 20°C)
b- Main curve of the bituminous mix tested (Temperature of reference 15°C)

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9.3. Fatigue cracking (variations in temperature)

These are produced by periodic - especially daily - variations in temperature, which are joined by load-induced stresses (or strains).
The phenomenon can be counteracted by carefully designing the mix, selecting the binder and designing the structure.
Hence it can be concluded that there is a relation between the capability of a mix to resist rutting and its | E* | modulus; the shape of the curve shows that the stability of the mix (Figure 9).
This explains why flow ruts in bituminous layers sooner appear in low traffic speed areas and why the phenomenon occurs mainly, and sometimes very rapidly, in hot weather periods, since these are the conditions under which the | E* | modulus is lowest (rising part of the curve in figure 9).

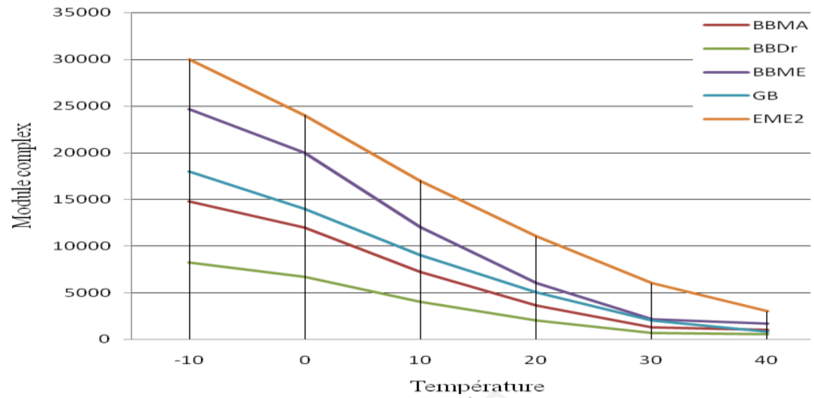

Fig. 9: Influence of the temperature on the modules complex

10. Calculation of the constraints and the deformations

In the course of tests, the constraint and the deformation are recorded according to a frequency of acquisition of 100 points per cycle of request. Although the signal generated by the generator of wave of the controller of the press is sinusoidal, it does not remain about it less than the answers in deformation and in constraint display a distortion due to the parameters of control of the press. In order to calculate the complex module, the parameters of calculation (maximum amplitudes: ε_o and σ_o) and the phase are established on the basis of curve of sinusoidal regression extrapolated starting from the experimental data by the method of least squares (Figure 10).

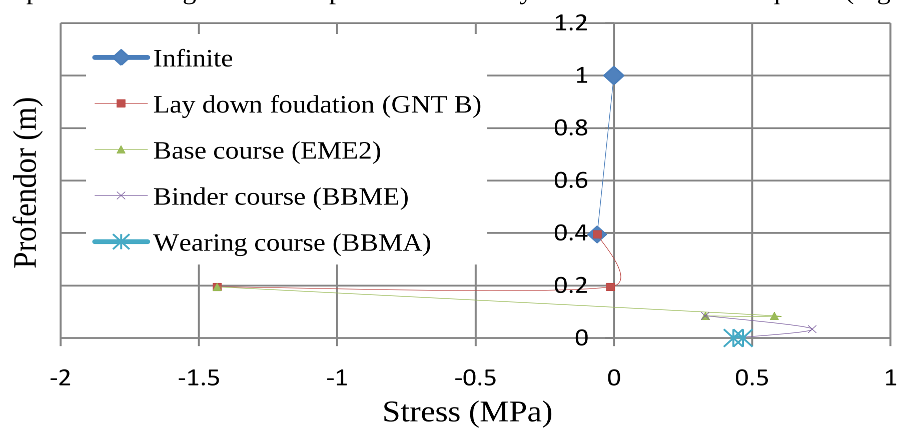

Fig. 10: The dissipation of shear stresses for the various layers

The calculation of the complex module starting from the maximum values recorded during a cycle rather than by those of the curve of regression can lead to an erroneous estimate of the complex module: error in estimation ranging between 20 and 50%.
There exists always an adhesive interface or a layer of penetration between wearing course/base course and base course/sub-base. Thus it is necessary to calculate the horizontal deformation of the base course as well as the vertical deformation of the sub-base and the sub grade.

According to the "Standard Catalog of the Structures of New Roadways (1998 SETRA/LCPC)" (Young f & al. 1998), the wearing course in BB is a 8.5 cm thickness, the sub-base in GNT is a thickness of 20cm, it remains the thickness of the base course to define. The computation result is summarized in the figures (11, 12 & 13).

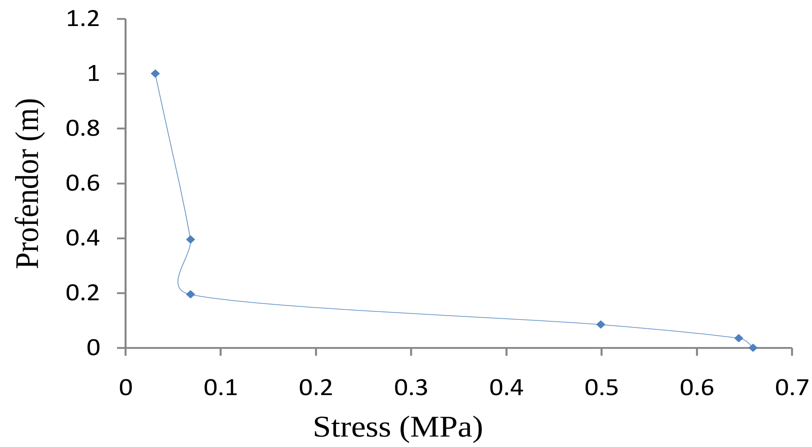

Fig. 11: The dissipation of the normal stress for the various layers

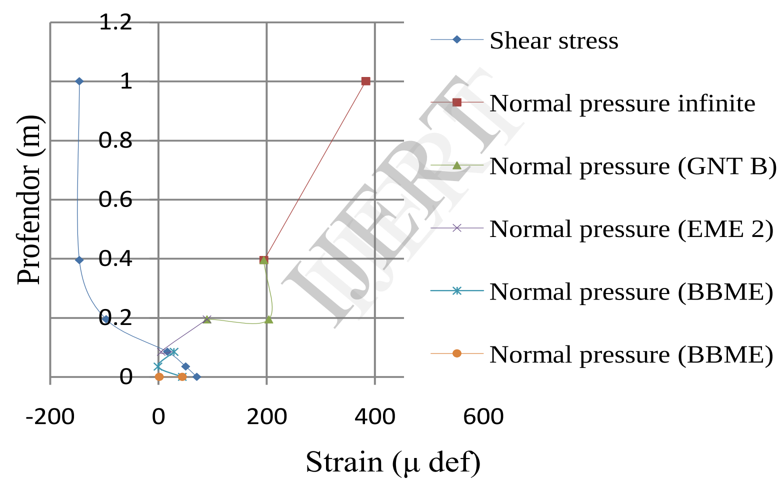

Fig. 12: Distribution of the normal deformations and sheerings

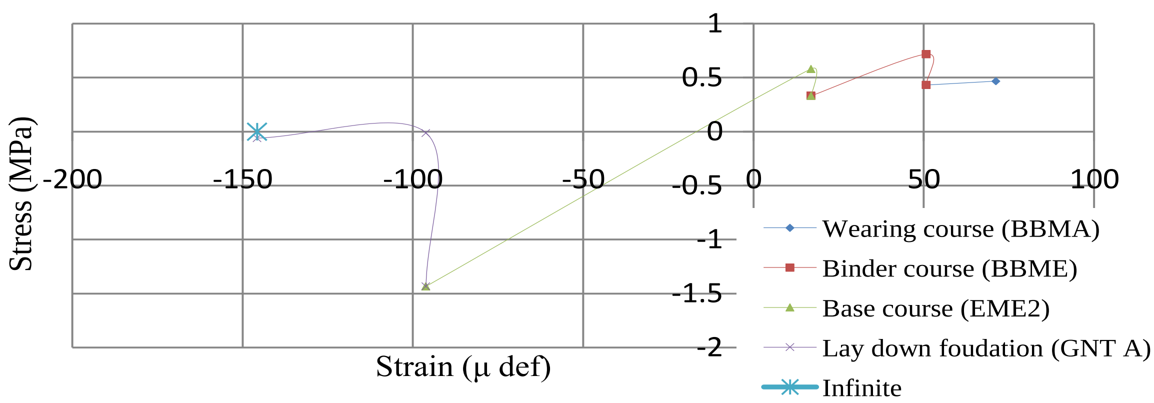

Fig. 13: Variation of shear stresses function of the deformations

The calculation of the efforts and the deformations which is carried out traditionally by considering multi-layer isotropic linear rubber bands, requires, initially, the knowledge of the Young modulus and possibly of the Poisson's ratio (to solve a problem considered as rubber band) (Elabd and al 2004 & Laveissiere, Laveissiere d 2001).
The results of the two parts (figures 11, 12 and 13) show that, compared with the reality of dimensioning, the taking into account of the behavior to the tiredness of the interfaces leads to a reduction in the lifespan (tiredness of the layers of bituminous mix). It is not sufficiently significant to modify the rational method of pavement design, because in all the cases one remains in the same class of road traffic. The level of joining to the interface remains constant during the lifespan of the structure for normal deformations.
The results of figures 12 and 13 showed that, if the tangential request is taken into account, a horizontal deformation (lengthening) would be of 212 µdef. This important value leads to the appearance of fatigue cracks on the road surface. Thus, it is imperative to check the behavior with the tiredness of the wearing course.
It is also shown that, if it took there into account at the same time tangential request and fatigue behavior of the interface, the lifespan of the sub-base is not very different from that obtained in the standard cases (current sections).

11. The average normal strain

From Equation (10) and Equation (15), the normal strain along the *x*-coordinate can be calculated as:

$$\varepsilon_x = -\frac{z}{R} = -\frac{Mz}{E_i I} \qquad Eq.(16)$$

For the BBME layer, the strain in the middle (centeroid) surface of the BBME is

$$(\varepsilon_{bbme})_{moy} = -\frac{M(z_{bbme} - \bar{z})}{E_{eme} I_{eme} + E_{bbme} I_{bbme}} = \frac{M(\bar{z} - z_{bbme})}{E_{eme}(I_{eme} + nI_{bbme})}$$

$$= \left(\frac{M}{E_{eme}}\right) \frac{\bar{z} - z_{bbme}}{\frac{bh_{eme}^3}{12} + (\bar{z} - z_{eme})^2 bh_{eme} + n\left(\frac{bh_{bbme}^3}{12} + (\bar{z} - z_{bbme})^2 bh_{bbme}\right)}$$

$$= \left(\frac{M}{bE_{eme}}\right) \frac{\bar{z} - z_{bbme}}{\frac{h_{eme}^3 + nh_{bbme}^3}{12} + h_{eme}(\bar{z} - z_{eme})^2 + nh_{bbme}(\bar{z} - z_{bbme})^2} \qquad Eq.(17)$$

With
z is the distance from neutral surface,
$n = \frac{E_{bbme}}{E_{eme}}$: Thickness ratio of BBME over EME2,
$\mu = \frac{h_{bbme}}{h_{eme}}$: Young's modulus ratio of BBME over EME2.
Since the following equations (16 and 17),

$$\bar{z} - z_{bbme} = \frac{h_{bbme} E_{bbme} z_{bbme} + h_{eme} E_{eme} z_{eme}}{h_{bbme} E_{bbme} + h_{eme} E_{eme}} - z_{bbme}$$

$$= \frac{h_{eme} E_{eme} (z_{eme} - z_{bbme})}{h_{bbme} E_{bbme} + h_{eme} E_{eme}} = \frac{h_{bbme} E_{bbme} (h_{bbme} + h_{eme})}{2(h_{bbme} E_{bbme} + h_{eme} E_{eme})} = \frac{h_{bbme}(1+\mu)}{2(1+\mu n)} \qquad Eq.(18)$$

$$\bar{z} - z_{eme} = \frac{h_{bbme} E_{bbme} z_{bbme} + h_{eme} E_{eme} z_{eme}}{h_{bbme} E_{bbme} + h_{eme} E_{eme}} - z_{eme}$$

$$= \frac{h_{eme} E_{eme} z_{eme} - h_{bbme} E_{bbme} z_{bbme}}{h_{bbme} E_{bbme} + h_{eme} E_{eme}} = \frac{h_{bbme} E_{bbme} (z_{bbme} - z_{eme})}{h_{bbme} E_{bbme} + h_{eme} E_{eme}}$$

$$= \frac{h_{bbme}(1+\mu)}{2(1+\mu n)} = \left(-\frac{h_{bbme} E_{bbme}}{h_{eme} E_{eme}}\right) \frac{h_{bbme} E_{bbme} (h_{bbme} + h_{eme})}{2(h_{bbme} E_{bbme} + h_{eme} E_{eme})}$$

$$= -\mu n \frac{h_{bbme}(1+\mu)}{2(1+\mu n)} \qquad Eq.(19)$$

The equation (19) becomes:

$$(\varepsilon_{bbme})_{moy} = \left(\frac{6M}{bh_{eme}^2 E_{eme}}\right)\frac{\mu+1}{\mu^4 n^2 + (4\mu^3 + 6\mu^2 + 4\mu)n + 1}$$

$$= \left(\frac{M}{bE_{eme}}\right)\frac{\frac{h_{eme}(1+\mu)}{2(1+\mu n)}}{\frac{h_{eme}^3 + nh_{bbme}^3}{12} + h_{eme}\left(-\mu n\frac{h_{eme}(1+\mu)}{2(1+\mu n)}\right)^2 + nh_{bbme}\left(\frac{h_{eme}(1+\mu)}{2(1+\mu n)}\right)^2}$$

$$= \left(\frac{M}{bh_{eme}^3 E_{eme}}\right)\frac{\frac{h_{eme}(1+\mu)}{2(1+\mu n)}}{\frac{1+n\mu^3}{12} + (\mu^2 n^2 + n\mu)\left(\frac{1+\mu}{2(1+\mu n)}\right)^2} \qquad Eq.\ (20)$$

The plot of the average strain in the BBME with four different thickness ratios is shown in Figure 14. The Young's modulus of the EME2 and bending moment used in the simulation are

$E_{eme} = 11$ (GPa) and $M = 100$ (N·m) respectively.

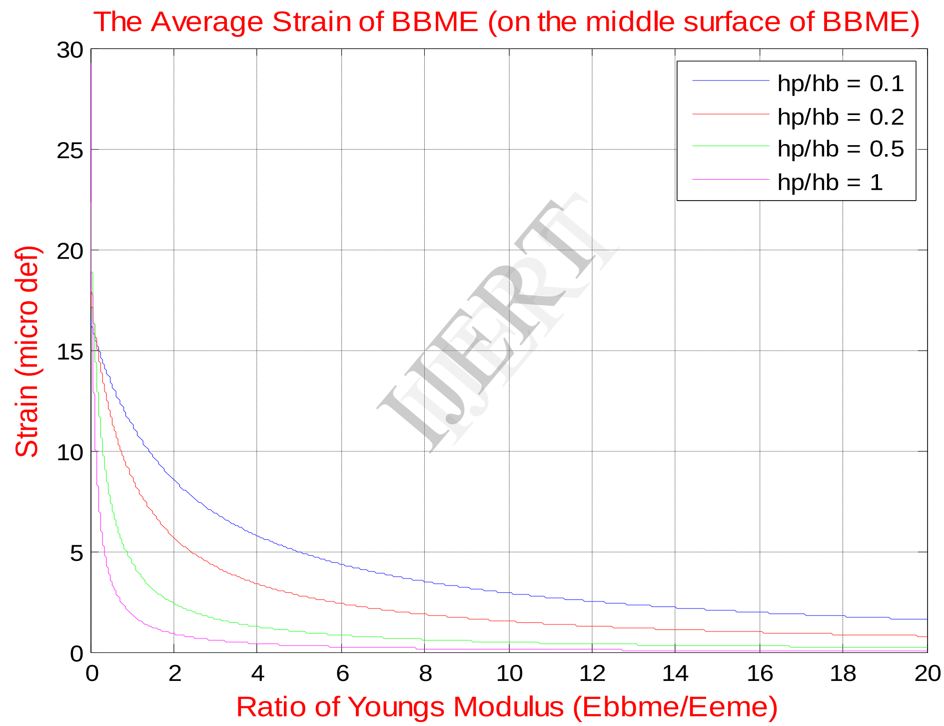

Fig.14: Average strain of the BBME layer (on the middle surface of the BBME)

For the surface layer, we have the same configuration (deformation, modulus of rigidity) for different thickness ratio between the surface layer and the base layer, we note that the higher the rigidity n and most important distortions becomes weaker, which means that the higher the modulus of rigidity of the surface layer is larger by the addition to the base layer we have a mesh distribution of the load due to traffic worm's body floor so a slight deformation of asphalt layers. Therefore, the more the thickness of the base layer is larger by a contribution to the surface layer deformation at the last (BBME) becomes more important which confirms that the thickness ratio between the base layer and the surface layer has an influence on the deformation behavior of the pavement. Similarly, for the beam layer, the strain in the middle (centeroid) surface of the EME is

$$(\varepsilon_{eme})_{moy} = -\frac{M(z_{eme} - \bar{z})}{E_{eme} I_{eme} + E_{bbme} I_{bbme}} = \frac{M(\bar{z} - z_{eme})}{E_{eme}(I_{eme} + nI_{bbme})}$$

$$= \left(\frac{M}{E_{eme}}\right)\frac{\bar{z} - z_{eme}}{\frac{bh_{eme}^3}{12} + (\bar{z} - z_{eme})^2 bh_{eme} + n\left(\frac{bh_{bbme}^3}{12} + (\bar{z} - z_{bbme})^2 bh_{bbme}\right)}$$

$$= \left(\frac{M}{bE_{eme}}\right)\frac{-\mu n\frac{h_{eme}(1+\mu)}{2(1+\mu n)}}{\frac{h_{eme}^3 + nh_{bbme}^3}{12} + h_{eme}\left(-\mu n\frac{h_{eme}(1+\mu)}{2(1+\mu n)}\right)^2 + \mu h_{bbme}\left(\frac{h_{eme}(1+\mu)}{2(1+\mu n)}\right)^2}$$

$$= \left(\frac{-M}{bh_{eme}^3 E_{eme}}\right)\frac{\mu n\frac{h_{eme}(1+\mu)}{2(1+\mu n)}}{\frac{1+n\mu^3}{12} + (\mu^2 n^2 + n\mu)\left(\frac{1+\mu}{2(1+\mu n)}\right)^2}$$

$$= \left(\frac{-6M}{bh_{eme}^2 E_{eme}}\right)\frac{(\mu^2+\mu)n}{\mu^4 n^2 + (4\mu^3 + 6\mu^2 + 4\mu)n + 1} \qquad Eq.(21)$$

The average deformation in the base course (EME2) with four reports/ratios different of thickness is represented in figure 15.

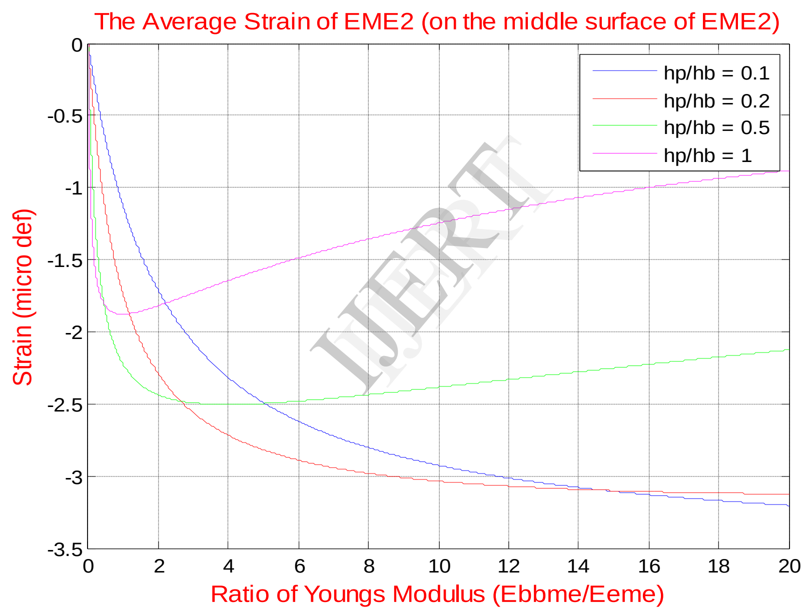

Fig.15: Average strain of the EME2 layer (on the middle surface of the EME2)

For the base layer, we have a distribution with opposite sign by input to the surface layer which signified the deformation transmitted from the surface layer are balanced at the base layer is the objective is to seek the base layer [50,51,52,53]. In addition, if the thickness of the surface layer is between 0.5 and 1 by contribution to the base layer we have a homogeneous and uniform distribution for different stiffness ratio by more against the thickness of the surface layer is lower by contribution to the base layer we have a change of sign of deformation at the layer EME2, which means that over the thickness of the surface layer is lower the more we have a very large distribution of effort between the surface layer and the base layer. Figures 14 and 15shows that the deformation in the base layer and the deformation in the surface layer have opposite signs, and when n is not too large, the average strain at the surface layer is much greater than that of the base layer [54,55,56,57,58,59].
The interface layer of surface-base layer is a sensitive area because it is often subjected to high normal stresses and shear horizontal and the upper centimeters of the seat are often treated as a low resistance (Bezazi 2009) represents the following figure 16.

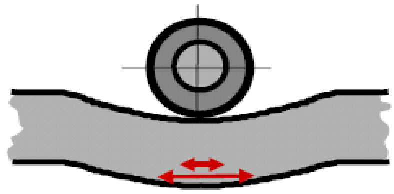

Fig.16: Cracking in inflection with the interface of the two layers

12. Average normal constraints

The stress σ can be obtained from equation $= E_i \varepsilon$, where E_i is the Young's modulus (unit: $Newton/Meter^2$ or Pascal).

The average stress of the BBME layer is

$$(\sigma_{bbme})_{moy} = E_{bbme}\,(\varepsilon_{bbme})_{moy} = nE_{eme}\,(\varepsilon_{bbme})_{moy}$$

$$= \left(\frac{6M}{bh_{eme}^2}\right)\frac{(\mu+1)n}{\mu^4 n^2 + (4\mu^3 + 6\mu^2 + 4\mu)n + 1} \qquad Eq.(22)$$

The average stress of the EME layer

$$(\sigma_{eme})_{moy} = E_{eme}\,(\varepsilon_{eme})_{moy} = \left(\frac{-6M}{bh_{eme}^2}\right)\frac{\mu(\mu+1)n}{\mu^4 n^2 + (4\mu^3 + 6\mu^2 + 4\mu)n + 1} \qquad Eq.(23)$$

One can notice that the ratio of average stress in the beam over average stress in the BBME is equal to the thickness ratio μ but in opposite sign (-μ), i.e. $\frac{(\sigma_{eme})_{moy}}{(\sigma_{bbme})_{moy}} = -\mu$.

When μ =1, this means the average stress in the EME2 and the BBME are the same but with negative sign.
Negative sign means that one is in compression and the other is in extension or vice versa.
This is consistent with beam theory.
Figure 17.a and Figure 17.b are the plots of the average stress in the BBME and the average stress in the EME2 respectively.

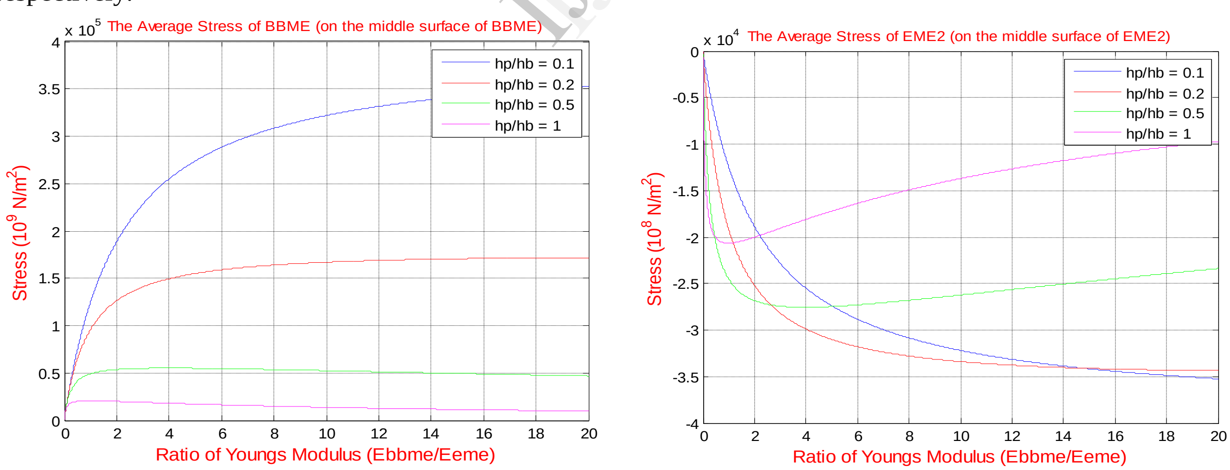

Fig.17: a- Average stress of the BBME layer (on the middle surface of the BBME)
b- Average stress of the EME2 layer (on the middle surface of the EME2)

Figure 17.a shows that the stress distribution at the surface layer is uniform for different thickness ratio and modulus between the surface layer and the base layer for low ratios of thickness, for if cons the thickness ratio between the layer exceeds a half we have no distribution constraints. This confirms the results in Figure 14 for the strain distribution in the base layer....

Figure 17.b shows the same distribution obtained in Figure 15, so we in a linear distribution of behavior between the stresses and strains at the base layer which means that the base layer to provide mechanical strength

to the floor vertical loads induced by traffic and distribute the pressure on the support platform to maintain the deformation to an acceptable level [60,61,62,63,64,65,66,67].
Consisting of aggregate bound by the asphalt binder, asphalt mix has certain mechanical properties similar to those of dry granular materials. The amplitude of the extension portion of the signal solicitation is approximately three to four times greater than that corresponding to the part in contraction. In addition, the resistance of the asphalt in tension is much lower than its compressive strength. The fatigue damage is therefore mainly in the phase of traction. Tensile bending being greater at the base of the road (when it is bonded layers), the beginning of the crack should theoretically trigger there.
The shape of the load and the number of parameters that define it (change in temperature, load, traffic, layer thickness, and the bearing capacity of foundation soil, climatic effects, and the modulus of rigidity ...) highlight the difficulties of reproducing the actual load.

13. Conclusion

Transmission constraints and travel between the base layer and the surface layer depends mainly on the mechanical behavior of two layers, and the behavior of the two layers at the interface. From the structural point of view, it is essential information for pavement design.
If used on layers support in good structural condition, the high modulus asphalt show no damage due to fatigue and the same is due to the balance between the strain distribution between the base layer and surface, the same is due to the distribution of stress at these two layers.
Problems with flow rutting in bituminous mixes had their source in the considerable advantages held out by these materials for road construction (impervious surfacing protecting the pavement structure from the damaging effects of water, relative ease of laying) and probably also in people thinking too soon that these advantages had been acquired for good. In doing so they did not reckon with a considerable gradual increase in traffic aggressivity or with the invariable difficulty of adapting to a rapidly changing situation.
However, experience has shown that even under severe loading conditions it is at present always possible to make bituminous mixes that perform well, provided that maximum use is made of the contributions of each of the constituents and of any beneficial interactions that may exist between them (friction between aggregate particles, voids in the aggregate, consistency of the bitumen-filler system, etc.).
In other words, constituents must be properly selected and judiciously proportioned, and mixes must be made as homogeneous as possible and laid with care. All this points to the necessity of making a mix design study and verifying its results through one or more mechanical tests in the laboratory.
The appearance of flow rutting in the eighties triggered a process of complete review of bituminous mix design in Quebec, based on performance requirements. The pursuit of given physical characteristics has been abandoned in favour of usage properties. With flow in bituminous materials relatively well under control, we are now concentrating on increasing low- temperature durability and flexibility in order to optimize the use of bituminous materials.

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