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Analytical Investigation for Determining Ressilient Modulus for Interface Layer of Aggregates

Swapan Kumar Bagui

PhD Student, Bengal Engineering Science University, Howrah, India, Sr. Pavement Specialist, ICT (I) Pvt., Ltd., A- 8 Green Park, New Delhi, India

ABSTRACT

Quality road aggregates have become rare and costly in many places in India due to massive construction activities required for the development of new infrastructure facilities. The pavement industry seeks ways of improving lower quality materials that are readily available for use in roadway construction. Cement treatment has become an accepted method for increasing the strength and durability of soils and marginal aggregates. However, cement-stabilized bases can also be the source of shrinkage cracks in the stabilized base layer, which can reflect through the asphalt surface. Introducing “Interface Layer of Aggregates (ILA)” between stabilized base and bituminous concrete (BC) layer appears to be very favorable. The present paper presents the determination of resilient modulus of ILA using FPAVE software. Figures have been developed to determine resilient modulus, varying the CBR value, the thickness of cement treated base and the thickness of interface layer of aggregates which is useful to determine the pavement thickness for cement treated base.

KEYWORDS: Pavement design, CBR, Resilient Modulus, Interface Layer of Aggregates, CTB.

INTRODUCTION

Quality road aggregates have become scarce and costly in many places in India due to massive construction activities required for the development of new infrastructure facilities. The pavement industry seeks ways of improving lower quality materials that are readily available for use in roadway construction. Cement treatment has become an accepted method for increasing the strength and durability of soils and marginal aggregates.

The two major problems that arise with the use of stabilized materials in road pavement layers are: cracking and the long -term durability of the material. The extent to which either of these is a problem is intimately related to the purpose of the stabilized layer in the road pavement as a whole and it is, therefore, difficult to divorce the two factors.

Many factors contribute to the cracking and crack - spacing of stabilized pavement layers. Some of them are listed below:

1) Tensile strength of the stabilized material;
2) Shrinkage characteristics;
3) Volume changes resulting from temperature or moisture variations;
4) The subgrade restraint;
5) Stiffness and creep of the stabilized material; and
6) External loadings such as those caused by traffic.

As in the case of compressive strength, the tensile strength of stabilized materials takes time to develop. On the other hand, a stabilized material in a road pavement layer will be subject to volume changes resulting from at least one of the factors listed above as soon as it is compacted. Cracking in stabilized layers

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due to changes in temperature or moisture content cannot, therefore, be avoided and must be accepted as inevitable although steps can be taken to reduce the effect. Cracking may also occur as a result of fatigue failure due to trafficking and is an entirely separate phenomenon from the initial cracking due to environmental changes.

Cracks in stabilized layers used at capping and sub-base level are unlikely to cause significant problems, but at base level the cracks may be reflected through the surfacing. The existence of cracks in a road surface may be assumed to indicate the need for remedial action . The consequences of not doing so, may range from no problems at all to loss of interlock or to eventual failure when the stabilized layer has reduced to unconnected blocks. Cracks may also permit ingress of water leading to weathering of materials at crack faces, de-bonding between pavement layers or deterioration of moisture-susceptible layers beneath the stabilized layer.

Introducing "Interface Layer of Aggregates (ILA)" between stabilized base and bituminous concrete (BC) layer, appears to be very promising. It is also known as "Inverted Pavement". Only limited research has taken place in this area till now in India. It is, therefore, necessary to conduct some more studies on fatigue lives of BC layer on cracked CTB with and without crack relief layers.

However, cement-stabilized bases can also be the source of shrinkage cracks in the stabilized base layer, which can reflect through the asphalt surface. The cracks that develop are not the result of a structural deficiency, but rather a natural characteristic of cement-stabilized bases. The surface cracks tend to follow the same pattern as the cracks in the base, and are referred to as "reflection" cracks. This paper examines the reason for the crack reflection and discusses different design and construction procedures that can control the occurrence of these cracks.

Three conditions must occur in order for reflection cracking to happen:

1. Cracks in the base layer must be wide enough to generate stress concentrations in the asphalt surface.
2. There is no method available to relieve the stress concentrations.
3. The asphalt is brittle enough to crack due to the upward propagation of the stress concentration.

Research work has taken us in India and other places to minimize the reflection cracking. Methods of controlling reflective cracking fall into the categories of:

1) Reducing the width of cracks in the stabilized base.
2) Providing stress relief at the base-surface interface such as:
   a) A bituminous surface treatment (chip seal) between the stabilized base and the asphalt surface.
   b) A geotextile between the stabilized base and surface.
   c) Providing a layer of aggregates between the stabilized base and the aggregate layer.
   d) Providing a 75 mm BM layer.
3) Increasing the BC layer thickness.

Methods of controlling reflective cracking basically fall into the categories of: 1) reducing the width of cracks in the stabilized base and 2) providing stress relief at the base-surface interface.

Pre-cracking

Originally reported in Austria in 1995, the method has been successfully tried on several projects in the United States. The procedure involves several passes of a large vibratory roller over the cement-stabilized base one to two days after final compaction. This introduces a network of closely spaced hairline cracks into the cement-stabilized material, which acts to relieve the shrinkage stresses in the early stages of curing and provides a crack pattern that will minimize the development of wide shrinkage cracks. In addition, since the pre-cracking is performed shortly after placement, the "micro- cracking" will not impact the pavement's overall structural capacity as the cracks will heal and the cement-stabilized material will continue to gain strength with time.

Scullion (2002) reported on a demonstration project involving several streets in a new residential subdivision in Texas. Three separate street sections were constructed using the precracking technique with an adjoining fourth street built in conventional fashion and used as a control section. The pavement design of all four streets consisted of 150 mm (6 in.) of limestabilized subgrade, 150 mm (6 in.) of cement-stabilized base and 50 mm (2 in.) of hot-mixed asphalt surfacing. The specified minimum design strength of the cement-stabilized layer was 3.4 MPa (500 psi) at 7 days.

In the study of Wayne and David (2004), the three sections were pre-cracked either one or two days after construction using a 10.9 tonne (12 ton) vibratory roller with the vibrator set on the maximum amplitude and traveling at a slow walking speed of about 3.2 kph (2 mph). Changes in the base stiffness were monitored before and after rolling. The average base stiffness decreased by approximately 30% after two passes of the roller and another 15% to 20% after two additional passes. Additional stiffness measurements with a Falling Weight Deflectometer (FWD) were made after 6 months for all three pre-cracked sections. The results showed that the stiffness measurements equaled or exceeded the initial stiffness measurements before cracking, indicating that the sections continued to gain strength with time. Visual observations after approximately one year indicated additional cracking in the pre-cracked sections, but still considerably less than in the control section.

Stress Relief Layer

Wayne and David (2004) reported in a conference paper that another method of reducing the potential for reflection cracking is to relieve the stress concentrations that result from cracks in the cement-stabilized base.

Pavement designs will reduce the stresses that cause reflection cracks (Wayne and David, 2004).

A bituminous surface treatment (chip seal) is introduced between the stabilized base and the asphalt surface. The additional flexibility of the surface treatment layer will help reduce stress concentrations. This surface treatment also provides an excellent temporary surface during construction for traffic control.

A geotextile can be introduced between the stabilized base and surface, or between the asphalt binder and wearing courses. Similar to the surface treatment, the geotextile provides flexibility and acts to intercept cracks without letting them pass through the material. A 50 mm to 100 mm (2 to 4 in.) layer of unbound granular material is introduced between the stabilized base layer and the asphalt surface. This use of a “sandwich” or “inverted” pavement design adds additional structure to the pavement and prevents the propagation of cracks through to the surface layer.

LITERATURE REVIEW

Cracks that reflect through the bituminous layer are usually prevented if the cemented material is confined to the sub-base layer and the base consists of untreated material. The initial cracks in the sub-base do not usually reflect through untreated base material like G1, G2 or G3. This is the most effective way of preventing cracks (TRH13, 1986).

The STRATA® Reflective Crack Relief System consists of a polymer-rich dense fine aggregate mixture layer that is placed on the top of the deteriorated pavement and is then overlaid with HMA. As indicated by the manufacturer and owner of this technology (SemMaterials), the use of the STRATA® system delays the appearance of reflective cracking for two years and extends the overlay service life against reflective cracking by five years. The manufacturer recommends using this system on structurally-sound concrete pavement in which any severe distresses should be repaired prior to application. Since its first application in 2001, at least 28 states including Louisiana have tested the STRATA® system with mixed performance (Bischoff, 2007).

Carey presented one of the first evaluations of paving fabrics in Louisiana (Carey, 1975). Two paving fabrics (a nonwoven polypropylene fabric and a nylon fabric) were applied to highly distressed concrete pavements prior to the placement of HMA overlays to act as strain energy absorbers. A visual survey was conducted periodically for each test section to evaluate the effectiveness of the interlayer system in delaying reflective cracks. A comparison of test sections to control sections indicated that paving fabrics were not effective in delaying or preventing reflective cracking. However, a long-term evaluation of the test sections was recommended to evaluate the potential of the fabrics to provide waterproofing benefits after reflective cracks have appeared.

Marks (1990) presented the performance of fiber-glass grid in delaying reflective cracking in four test sections in Iowa . The fiber-glass grid was installed on I- 35 in which two 1.5-in. lifts of binder course were placed followed by a 1.5-in. wearing surface. Performance was monitored annually for five years by determining the number of cracks that reflected through the layer and by comparing the reinforced sections to the control segments. In one section, the fiber-glass grid was placed directly on the top of the concrete pavement, while in the three other sections it was placed between lifts of asphalt mixture. Results of this monitoring showed that the best performer was the section in which the fiberglass grid was placed directly on the top of the concrete pavement, with 43 percent of the joints reflecting after five years. The poorest performer was one section with the fiber-glass grid placed between lifts of asphalt concrete with 80 percent of the joints reflecting after five years. Conclusion of this study indicated that the use of fiber-glass grid yields a small reduction in reflective cracking but does not justify the cost of this interlayer system.

Dempsey (2002) developed a composite interlayer system, known as the Interlayer Stress Absorbing Composite (ISAC), which consists of a low stiffness geotextile at the bottom, a viscoelastic membrane at the center and a high stiffness geotextile at the top. A detailed analysis of the causes of reflective cracking indicated that neither a stress-absorbing membrane interlayer (SAMI) nor a geotextile can completely control this distress when used separately. Through the ISAC system, the low-stiffness geotextile fully adheres to the existing pavement and accommodates large deformation at the joint without breaking its bond with the slab.

Mechanistic analyses using available pavement analysis programs-KENPAVE, WINLEA, EVERSTRS, EVERFLEX and VESYS 5W were conducted for the Federal Highway Administration's (FHWA's) Accelerated Loading Facility (ALF) pavements that were constructed using different asphalt binders including highly modified and unmodified ones. The analyses were divided and presented in two parts: (1) the primary response under the ALF pavements and (2) the performance of the ALF pavements. In the present paper, the discussion is limited to the second part as the first part is available elsewhere. The analyses included pavement performance analysis using VESYS 5W program, performance predictions using existing rutting and fatigue models, and calibration and evaluation of these models. The mechanistic analyses showed that the VESYS-predicted HMA rut depths correlate well with the ALF-measured HMA rut depths for both the ALF 150-mm and 100-mm asphalt pavements, and that the VESYS 5W program seems to be capable of predicting the HMA layer rutting more effectively than the pavement total (surface) rutting. It was also found that the VESYS 5W program under-predicts the ALF-measured rut depths in the primary phase (at earlier stages) of rutting, and it favorably predicts the measured values in the secondary stage of rutting (Al-Khateeb et al., 2007).

OBJECTIVES

The primary objectives of the present study are:

- To determine relative fatigue lives of BC layer over cracked CTB with and without ILA .
- To carry out analytical investigation to develop

"design procedure" for inverted asphalt pavements over "Cement Treated Bases".

- To develop a model for predicting the resilient modulus of ILA.

PROPOSED METHODOLOGY

The proposed methodology consists of two steps as follows:

Step 1- Resilient Modulus of Granular Materials

Granular materials constitute the major part of the thickness of flexible pavement. The knowledge of elastic modulus of granular layer is necessary to analyze the flexible pavement. Since the primary objective of the study is the evaluation of failure criteria of the granular layer and correlating with resilient modulus of material, it is necessary to review the work conducted in the laboratory and field investigations on the granular layer and also the theoretical approach followed for analysis of flexible pavements with granular layers.

The following section reviews the relevant laboratory investigations carried out and various procedures adopted to measure the resilient modulus taking principle stress ratio failure into account. Different analytical approaches used by researchers for the analysis of flexible pavements having granular layers are also highlighted.

AASHTO design guide 2002 (NCHRP 1-37A) has proposed a generalized constitutive model for the characterization of unbound granular materials, which is as follows:

$$M_R = k_1\, P_a\, (\theta/P_a)^{k2}\, [(\tau_{oct}+1)/P_a)]^{k3} \quad (1)$$

where,

M_R = Resilient modulus of the material in psi,

P_a = Normalizing stresses (atmospheric pressure) in psi,

θ = Bulk stress in psi,

τ_{oct} = Octahedral shear stress in psi,

k_1, k_2 and k_3 are constants. These constants may be determined from repeated triaxial test as per AASHTO T309 1999.

Some tests were conducted to determine the regression coefficients. So, the final equation adopted in AUSTROADS '04 is:

$$M_R = 300(\sigma m/100)^{1.05}\, (\tau_{oct}/100)_a)^{-0.4} \quad (2)$$

where,

M_R = Modulus (MPa).

σm = Mean normal stress (kPa).

τ_{oct} = Octahedral shear stress (kPa).

This equation has been used to determine M_R value of CTB by trial and error method using different stress values as obtained from S.A. trial thicknesses of flexible pavement using Step 2 running FPAVE Software. CTB shall be considered initially with a trial M_R value of CTB, and mean normal stress and octagonal stress are determined using the following equations:

$$\tau_{oct} = (1/3)[\{ \sigma_{z-} \sigma_r\}^2+\{ \sigma_{z-} \sigma_\tau \}^2+\{\sigma\, \tau_{\_}\, \sigma_r\}^2]^{0.5} \quad (3)$$

$$\sigma m = (\, \sigma_{z+}\, \sigma_{r+}\, \sigma_\tau)_{/3} \quad (4)$$

M_R is determined using the values of mean normal stress and octagonal stress from the two equations (3 and 4). Trial shall be terminated when the assumed value is close to the achieved value.

Step 2- Analytical Determination of Resilient Modulus

A flexible pavement is modeled as an elastic multilayer structure and stresses and strains at critical locations (Figure 1) are computed using a linear layered elastic model. A software (FPAVE) developed under the MORTH Research Scheme R-56 'Analytical Design of Flexible Pavements' can be used for the analysis of stresses in flexible pavements. Any commercially available softwares such as CIRCLY, RUBICON, BISAR, KENPAVE… etc. can also be used for the analysis. Tensile strain, ϵ_t , at the bottom of the bituminous layer and vertical subgrade strain ,ϵ_v, on the top of the subgrade are conventionally considered as critical parameters for pavement design to limit cracking and rutting in the bituminous and non-bituminous layers, respectively. Computation in multilayer bituminous pavement indicates that tensile strain near the surface close to the edge of a wheel can be sufficiently large to initiate longitudinal surface cracking.

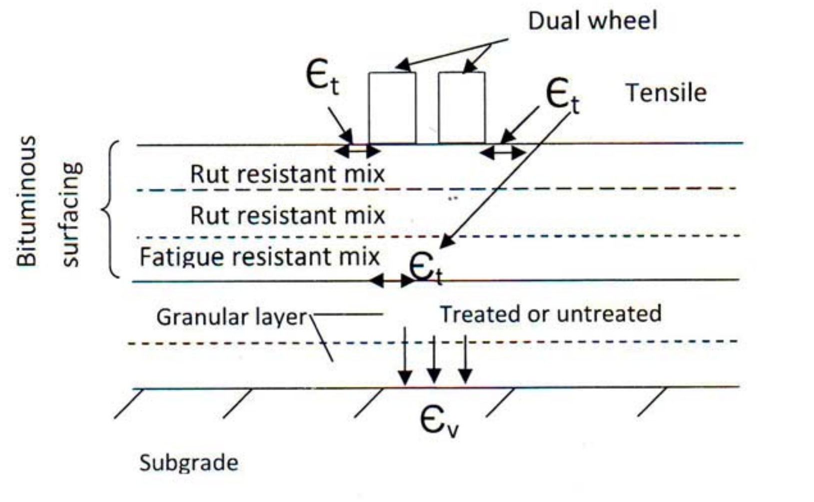

Figure 1: Different layers of a flexible pavement

Steps of Mechanistic Design Procedure of Inverted Asphalt Pavements Input Requirements:

Step 1: *Select a trial pavement thickness.*

Step 2: *Determine the following elastic parameters for the insight sub-grade and selected sub-grade materials* EV, EH =0.5 EV.

Step 3: *Determine the elastic parameters for sub-grade and sub-base.*

Resilient modulus for sub-grade and sub-base are determined using the following equations:

Sub-grade:

$$Mr(MPa) = 10 \times CBR \quad \text{for } CBR \leq 5 \text{ and}$$
$$= 17.6 \times CBR^{0.64} \quad \text{for } CBR > 5.$$

Granular sub-base and base:

$Mr_2 = Mr_3 \times 0.2 \times h^{0.45}$

Mr_2 = Composite elastic modulus of sub-base and base (MPa).

Mr_3 = Elastic modulus of sub-grade (MPa).

H = Layer thickness of the granular layer.

Step 4: *Determine the elastic parameters for CTB and bituminous layers.*

Assume resilient modulus of CTB for analysis and known value of resilient modulus of bituminous layer.

Step 5: *Program Analysis.*

Conduct analysis using FPAVE Software. Determine τ_{oct} stress from output results of FPAVE Software. Determine Mr for Interface Layer of Aggregates. Check assumed Mr and Mr obtained from equation (2) for ILA. If both values are close, adopt the value for ILA.

Step 6: *Comparison of pavement design life.*

Compare case studies with and without use of ILA for pavement design life.

Step 7: *Final pavement design.*

Assume pavement thickness and determine Mr of

ILA following the procedure mentioned above. Carry out stress and strain analysis. Finalize pavement life. If pavement life is close to design life, design is complete, otherwise go for next trial.

Following equations shall be used to determine fatigue life and rutting life of the pavement.

Fatigue life of a bituminous mixture for bottom up cracking at a reliability level of 80% is given as:

$$N_f = 2.21 * 10^{-04} \text{ x } [1/\varepsilon_t]^{3.89} * [1/E]^{0.854} \quad (5)$$

N_f = fatigue life, ε_t = Maximum tensile strain at the bottom of the bituminous layer, E= resilient modulus of the bituminous layer.

The equation for rutting (for 80% reliability) is given as:

$$N = 4.1656 \text{ x } 10^{-08} [1/\varepsilon_v]^{4.5337} \quad (6)$$

Indian Roads Congress (IRC) recommended the thickness of granular sub-base (GSB) of 200 mm for CBR of 5% and above, and 300 mm for CBR below 5%. In the suggested design, 300 mm (GSB) has been proposed. Thicknesses of bituminous layer and CTB have been varied from 100 mm to 200 mm, and from 150 mm to 250 mm, respectively, and the thickness of IGL has been varied from 50 mm to 150 mm.

Based on the above-mentioned procedure and adopting thicknesses as stated above, Mr of ILA has been determined varying different parameters like thickness of ILA and CBR of sub-grade. Final results are shown in Fig. 2 to Fig. 4.

Comparison of Fatigue Life with and without IGL

A simple example has been considered as stated below:

Example 1

CBR =10%

Asphalt concrete=100 mm, with E =3000MPa

IGL = 50 mm with E value = 675 MPa

GSB= 200 mm

CTB=150 mm with E =500 MPa

Maximum tensile strain at the bottom of asphalt is found to be 141 micro strains, and fatigue life is calculated (using equation 5) as 226 MSA, and rut life is found to be 239 MSA (using equation 6) for maximum sub-grade compressive strain of 334 micro strains.

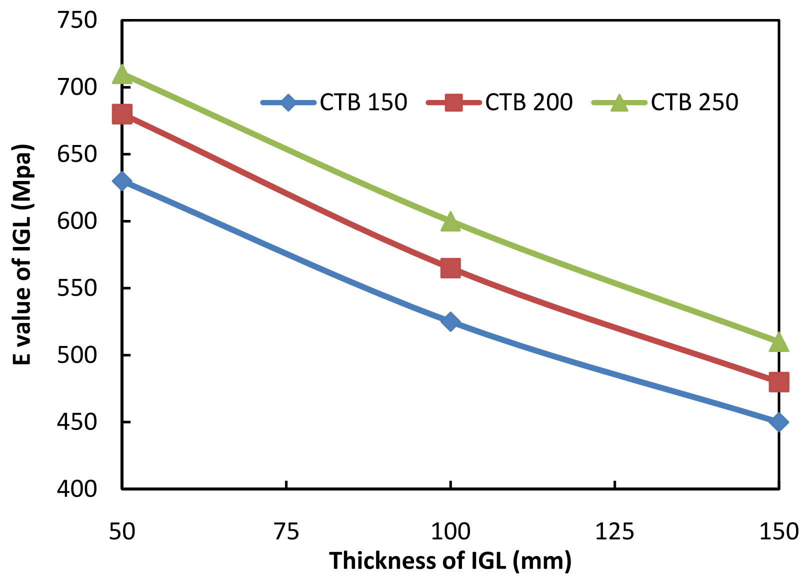

Figure 2A: E-value of IGL for bituminous thickness of 100 mm with sub-grade CBR of 5%

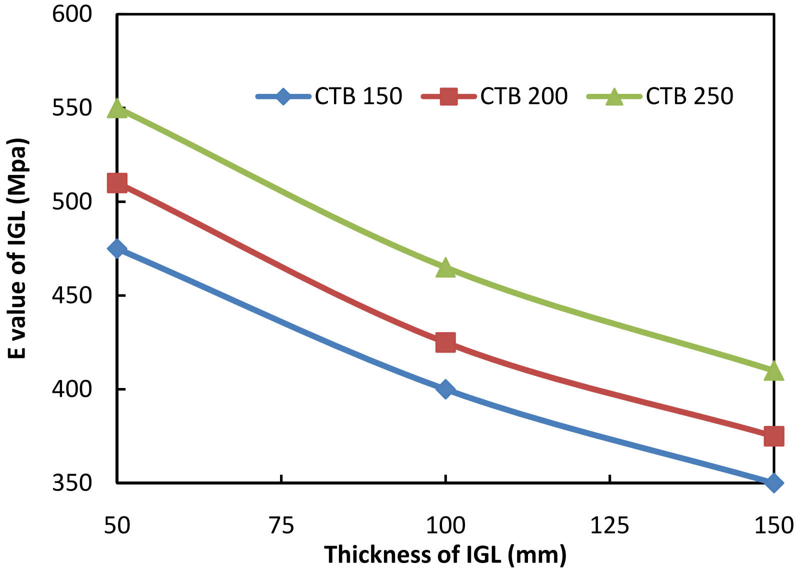

Figure 2B: E-value of IGL for bituminous thickness of 150 mm with sub-grade CBR of 5%

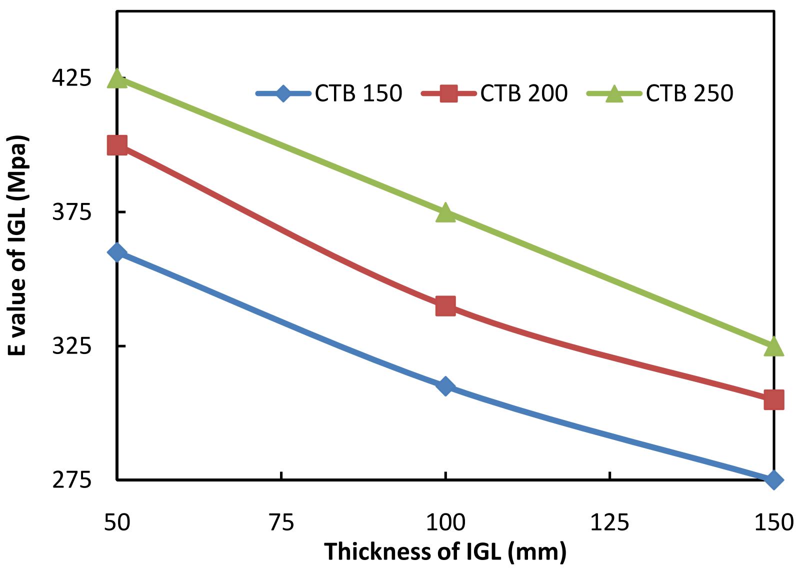

Figure 2C: E-value of IGL for bituminous thickness of 200 mm with sub-grade CBR of 5%

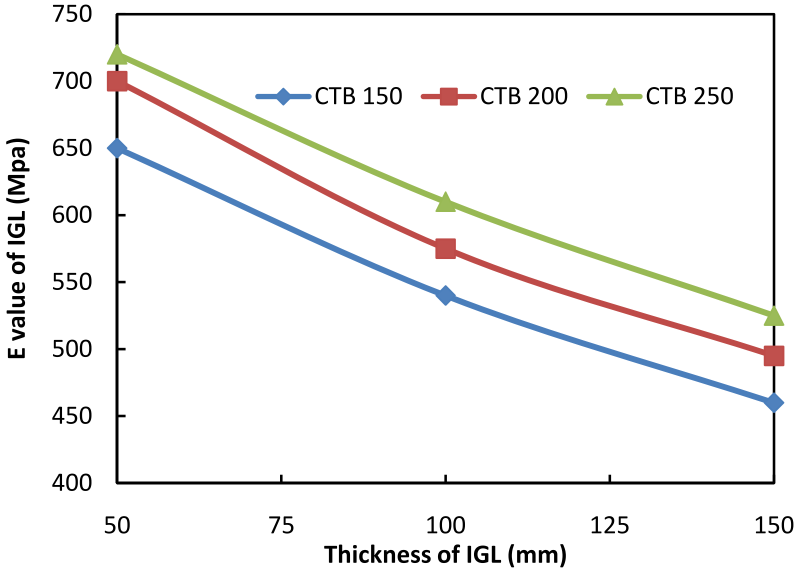

Figure 3A: E-value of IGL for bituminous thickness of 100 mm with sub-grade CBR of 7%

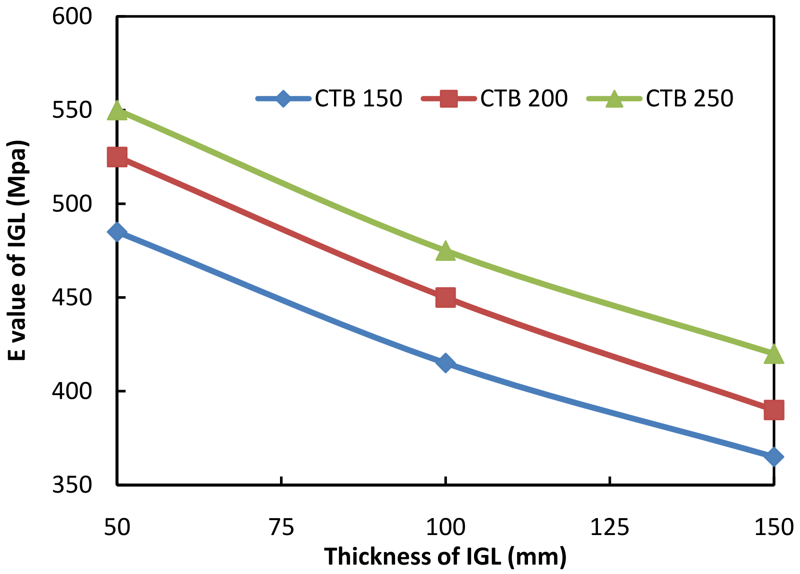

Figure 3B: E-value of IGL for bituminous thickness of 150 mm with sub-grade CBR of 7%

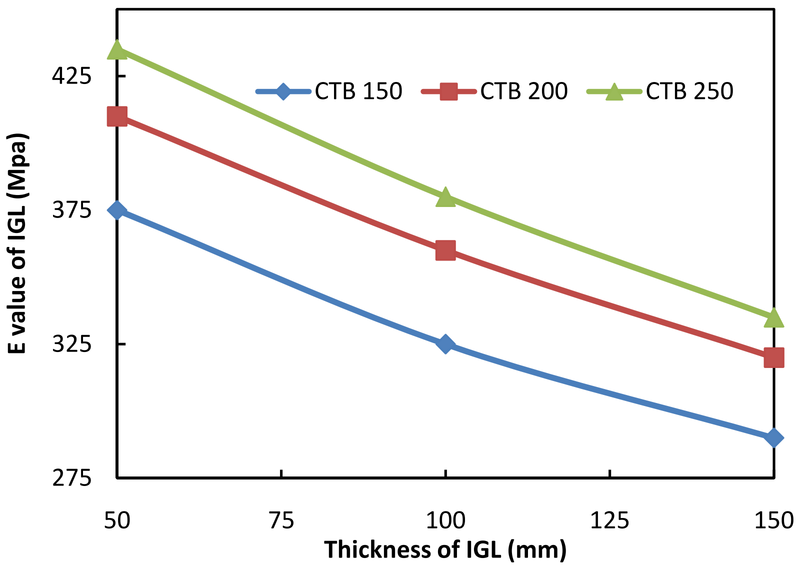

Figure 3C: E-value of IGL for bituminous thickness of 200 mm with sub-grade CBR of 7%

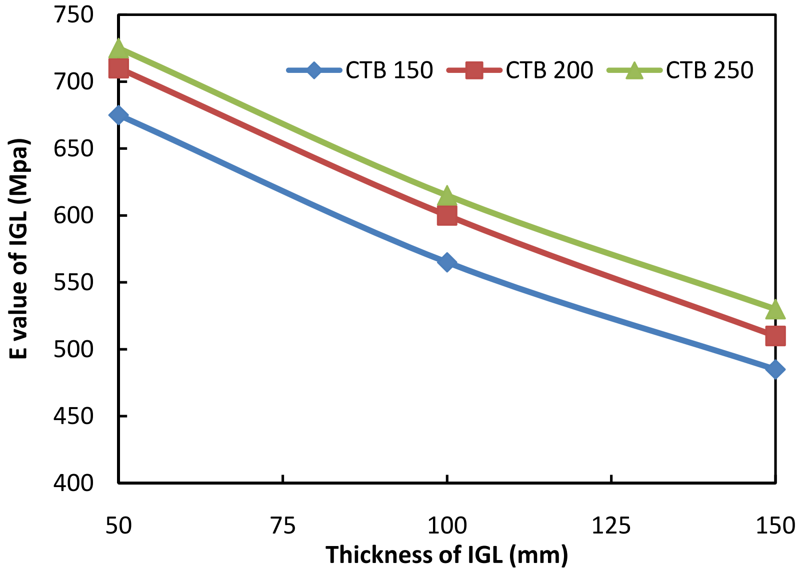

Figure 4A: E-value of IGL for bituminous thickness of 100 mm with sub-grade CBR of 10%

A simple example has been considered as follows:

Example 2

CBR =10%

Asphalt concrete=100 mm, with E =3000MPa

GSB= 200 mm

CTB=150 mm with E =500 MPa

Maximum tensile strain at the bottom of asphalt is found to be 175.5 micro strains, and fatigue life is calculated as 96 MSA, and rut life is found to be 102 MSA for maximum sub-grade compressive strain of 403 micro strains.

Ratio of fatigue life with and without IGL is found to be 226/96 = 2.35.This value shall be increased with the consideration of pre-cracking and post-cracking life of CTB.

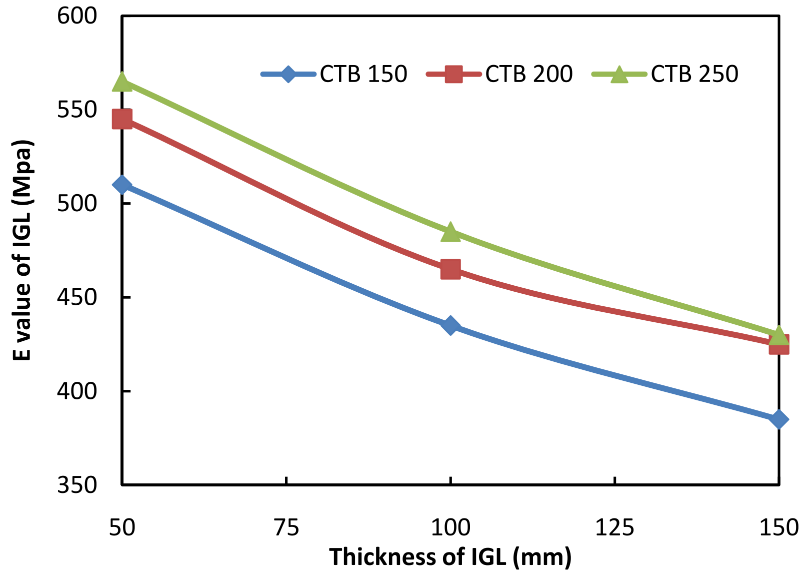

Figure 4B: E-value of IGL for bituminous thickness of 150 mm with sub-grade CBR of 10%

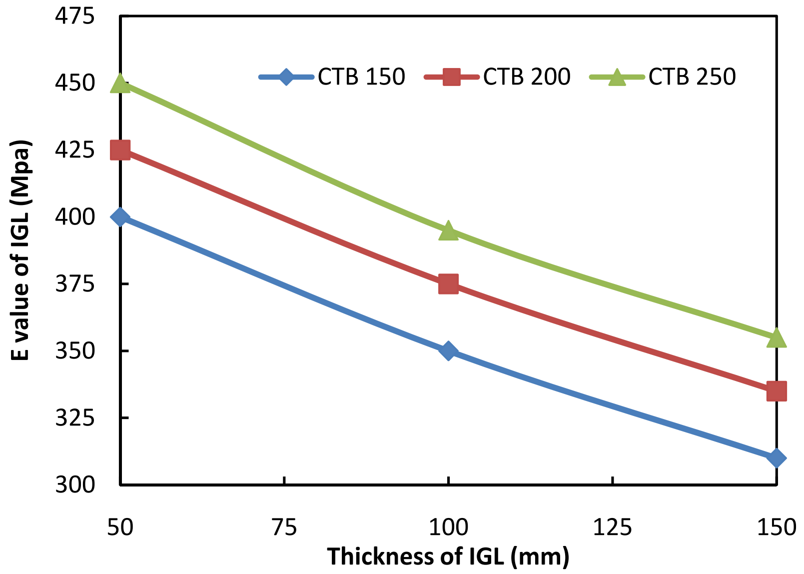

Figure 4C: E-value of IGL for bituminous thickness of 200 mm with sub-grade CBR of 10%

Determination of MR of IGL

A trial composition has been considered

Asphalt thickness=100 mm, E= 3000 MPa

IGL = 100 mm, E=525 MPa(assumed)

CTB=150 mm, E =500 MPa

GSB=200 mm, E=166.7 MPa

Sub-grade= 500 mm (CBR = 5 %)

Using FPAVE Software, output results are shown below:

No. of layers 5

E values (MPa) 3000.00 525.00 500.00 108.50 50.00

Mu values 0.500.400.300.400.40

thicknesses (mm) 100.00 100.00 150.00 200.00

single wheel load (N) 20400.00

tyre pressure (MPa) 0.56

Dual Wheel

Z	R	SigmaZ	SigmaT	SigmaR	TaoRZ	DispZ	epZ	epT	epR
100.00	155.00	-0.1439E+00	0.3998E+00	0.2528E-01	-0.1234E+00	0.4895E+00	-0.1188E-03	0.1530E-03	-0.3423E-04
100.00L	155.00	-0.1439E+00	-0.8837E-02	-0.7906E-01	-0.1234E+00	0.4895E+00	-0.2071E-03	0.1530E-03	-0.3423E-04
200.00	155.00	- 0.9547E-01	0.3840E-01	0.1092E-01	-0.7908E-01	0.4675E+00	-0.2175E-03	0.1368E-03	0.6352E-04
200.00L	155.00	-0.9547E-01	0.4215E-01	0.1697E-01	-0.7908E-01	0.4675E+00	-0.2262E-03	0.1368E-03	0.6352E-04
200.00	0.00	-0.1026E+00	0.3503E-01	0.1871E-01	-0.2850E-01	0.4541E+00	-0.2365E-03	0.1307E-03	0.8715E-04
200.00L	0.00	-0.1026E+00	0.4218E-01	0.2543E-01	-0.2850E-01	0.4541E+00	-0.2459E-03	0.1307E-03	0.8715E-04
550.00	0.00	-0.1683E-01	0.1579E-01	0.1360E-01	-0.2594E-02	0.3665E+00	-0.2635E-03	0.1574E-03	0.1292E-03
550.00L	0.00	-0.1683E-01	0.1227E-02	0.2197E-03	-0.2595E-02	0.3665E+00	-0.3482E-03	0.1574E-03	0.1292E-03
550.00	155.00	-0.1787E-01	0.1684E-01	0.1532E-01	-0.3491E-02	0.3743E+00	-0.2832E-03	0.1646E-03	0.1450E-03
550.00L	155.00	-0.1787E-01	0.1337E-02	0.6416E-03	-0.3498E-02	0.3743E+00	-0.3731E-03	0.1645E-03	0.1451E-03

Average values of SigmaZ, SigmaT and SigmaR are-0.099,0.039 and 0.018.

Using average stress values, E-value of IGL has been found to be 521 MPa; i.e., close to the assumed value. Therefore, E-value of IGL may be adopted as 525 MPa.

Values of σm and τoc are found to be 0.14 and 0.0607, and E-value of IGL is found to be 521 MPa. Therefore, E-value of IGL may be adopted as 525 MPa.

Using this method, E-value of IGL has been determined and plotted in Fig. 2 to Fig. 4.

ANALYSIS OF RESULTS

Results are shown in Figs. 2 to 4. Following points are highlighted:

- It has been found that E-value of inter granular layer (IGL) decreases with the increase in thickness of IGL. Therefore, minimum IGL layer thickness may be recommended considering construction feasibility. Considering this aspect, thickness of IGL shall be 100 mm.
- Again, it is observed that E-value of IGL increases with increasing the thickness of cement treated base (CTB) for a known IGL thickness and bituminous thickness.
- For known IGL and bituminous thicknesses, E value of IGL increases with increasing the thickness of CTB.
- E-value of IGL depends also on the thickness of bituminous layer. E-value of IGL decreases with

increasing the thickness of bituminous layer.

- E-value of IGL increases with increasing CBR value for the case of same pavement compositions.
- Fatigue life of CTB with IGL is longer than that of CTB without IGL. It generally varies (2-5) times or more depending on thickness provision.

CONCLUSIONS

Cement treated base is very useful to fulfil the lesser quantity aggregate along with poor quality of marginal aggregate. It reduces the thickness of base and sub-base which yields lesser consumption of aggregate. The proposed methodology and reported Mr for Inter Layer Aggregate (ILA) may be useful for the computation of pavement thickness with cement treated base. Design may be finalized by trial and error method as shown in the various steps of the design. Reported Mr can be determined by trial and error method which may be suitable for pavement design. Following conclusions may be drawn from the present study:

1) Design charts developed for elastic modulus of various thicknesses of ILA over different pavement layer thicknesses may be used in pavement design.
2) Analysis indicated that the fatigue life of Inverted Asphalt Pavements was longer than that of flexible pavements with CTB.
3) The reflection crack propagation through aggregate interface was significantly different from that without aggregates. Because of ILA, the initiation of cracks is not right above the CTB. Finer cracks initiate from the bottom of BC layer. The deflection is also less in inverted sections.
4) A step-wise mechanistic design procedure was developed for the Inverted Asphalt Pavements.

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