STUDY OF ALTERNATIVES OF FLEXIBLE PAVEMENT STRUCTURES IN HIGHWAYS

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ABSTRACT

This paper focuses on alternative flexible pavements each having a 30-year life cycle. Preferred alternatives are those that present the lowest Annual Uniform Equivalent Cost of Maintenance (C.A.U.E.M.) based on the conditions of study. These alternatives have the ability to adapt to extreme overloading conditions defined structurally by the DNIT and AASHTO pavement design method. The FLAPS software is used for the analysis of the stresses and strains on the pavement layers. This study considered the traffic count obtained by the DNIT (USACE) and AASHTO methods, incorporating traditional flexible pavement, semi-rigid pavement, inverted pavement and rigid pavement

KEY WORDS: Highways, Pavements, Alternatives, Annual Uniform Equivalent Cost of Maintenance (C.A.U.E.M)

1 INTRODUCTION

Pavements have been designed over the years with the aim of having a structure that have service lives (Vs) commensurate with their respective design periods (Pp). The design is done by considering functional and structural aspects as well as the cost of implementation (CI) which is linked to the budget resource (RO). Under the structural aspects, civil engineers consider rutting (ATR) and fatigue cracking (TR). The functional aspect of design deals with roughness (IRI) and friction (μ) . These studies are based on the design methods of DNIT and the AASHTO/2002 guide, having using FLAPS (Finite Layer Analysis of Pavement Structures) software as an analysis tool for stresses and strains (Rodrigues, 1998).

2 TRAFFIC ESTIMATION

Studies conducted showed that the ratio of AASHTO to USACE traffic load factors is approximately 1/3.

2.1 Traffic according to USACE - Flexible Pavement

Table 1 - Traffic projection USACE (Np), Flexible Pavement

Urban Crossing Piumhi- MG (Track 2,DNIT)			
Reference	Factors		
Period (years)	10	20	30
increase %/year	3%	3%	3%
Np	8,92E+06	2,09E+07	3,70E+07

2.2 Load Factor according to AASHTO - Flexible Pavement

The load Factor as a function of axis, ΔPSR,SN is obtained in the AASHTO Guide, using the equation:

$$(PSRo - PSR_N / PSRo\text{-}1{,}5) = (N/\rho)^{\beta} \qquad (\text{eq.2})$$

Where:

$$\rho = 5{,}93 + 9{,}36 \text{ x } \log(SN+1) - 4{,}79 \text{ x } \log(L1+L2) + 4{,}33 \text{ x } \log L2$$
$$\beta = 0{,}40 + (0{,}081 \text{ x } (L1 + L2)^{3,23}) / (SN + 1)^{5,19} + L2^{3,23}$$

L1 is the load in Kips and L2 is 1 (ESRS,ESRD), 2 (ETD) and 3(ETT), and SN is the structural number of the pavement.

Applying equation 2 above, to the same traffic, with the aim of estimating the traffic of the project, for flexible pavement the AASHTO Load factor (Fc) is 1,78.

2.3 Load Factor according to AASHTO - Rigid Pavement

The load factor (Fc), is obtained in AASHTO Guide, adopting the equation:

$$(PSRo - PSR_N / PSRo\text{-}1{,}5) = (N/\rho)^{\beta} \qquad (\text{eq.3})$$

Where:

$$\log \rho = 5{,}85 + 7{,}35 \text{ x } \log(D+1) - 4{,}62 \text{ x } \log(L1+L2) + 3{,}28 \text{ x } \log(L2)$$
$$\beta = 1 + (3{,}63 \text{ x } (L1 + L2)^{5,20}) / (D + 1)^{8,46} \; L2^{3,52}$$

L1 is the load in Kips and L2 in which: 1 (ESRS, ESRD), 2 (ETD), 3 (TTE), and D = concrete plate thickness in inches.

Applying equation 3 above, to the same traffic, with the aim of estimating the traffic for a rigid pavement with concrete slab thickness, D = 12 ", the AASHTO Load factor (Fc) is 5,37.

2.4 Traffic according to AASHTO- Flexible and Rigid Pavement

Table 2- AASHTO Traffic projection (Np), Flexible and Rigid Pavement

Urban Crossing Piumhi- MG (Track 2,AASHTO)						
Reference	Factors - Flexible Pavement			Factors - Rigid Pavement		
Period (years)	10	20	30	10	20	30
increase %/year	3%	3%	3%	3%	3%	3%
Np	2,61E+06	6,11E+06	1,08E+07	7,87E+06	1,84E+07	3,26E+07

3 DEFINITION OF PAVEMENT STRUCTURES

3.1 Equalization of subgrade

We can consider portions to be equalized in terms of suggestion, for example, with 1 km length of 1.2 km, assuming that this range is possible to have a material capable of being supported such as to allow this to be used as material loan. Table 3 below shows the values of thickness of equalization required for this overlay obtained by the difference between the total thickness of the pavement defined according to the CBR "in situ" and the total thickness defined according to the CBR material loan (CBR = 18%), based on the CBR method equation (eq.4).

$$Ht(cm) = 9{,}02 + (0{,}23 \times \log Np + 0{,}05) \times [7011/CBR-234{,}33]^{1/2} \quad \text{(eq.4)}$$

Thus, the assumed thickness of the overlays is shown below in Table 3, considering the USCAE design of traffic (Np) to 30 years, since we limit our design period up to 30 years.

Table 3 - Values of subgrade overlay thickness required for this equalization

Track	Soil expansion	CBR (%)	Acumulated Distance(m)	H(cm) local	H(cm) Calc.overlay	Field procedure
263+800	0,71	18	0	31,33	0,00	
264,1	7,36	3	300	91,13	59,81	4 layers
264,2	5,61	3	400	91,13	59,81	4 layers
264,3	4,3	6	560	63,75	32,42	3 layers
264,46	1,42	12	660	42,52	11,19	1 layer
264,64	3,36	11	840	44,97	13,64	1 layer
264,74	1,99	8	940	54,39	23,07	2 layers
264,84	2,5	16	1040	34,59	3,26	1 layer

3.2 Design of pavement structures

3.2.1 CBR Method - Flexible Pavement

The pavement structure, defined in Table 4, below, takes into account the CBR of 18% for the equalized subgrade in and active traffic within the project horizon of 10, 20 and 30 years and the concept of rupture of subgrade provided by the empirical method of DNER (current DNIT).

Table 4 - Flexible pavement structures

USACE Traffic	Design period (years)	Pavement total thickness (cm)	Hot Mix Asphalt Concrete (cm)	Calculated Unbound base layer (cm)	Adopetd Unbound base layer (cm)(*)	subgrade overlay(cm) (CBR=18%)
8,92E+06	10	29,56	7,5	11,76	15	15 to 60 (see Table 3)
2,09E+07	20	30,62	10	7,71		
3,70E+07	30	31,33	10	8,32		
NOTE(*): The minimum Thickness recommended by DNIT is 15 cm.						

3.2.2 AASHTO Method of Flexible Pavement

The equation from the AASHTO Guide 2002/2004, presented below (eq.5), is used in an iterative process to obtain the structural number of pavement layers, considering the value of ZR = -1, So = 0.4, Level of confidence 85%. ΔPSR = 3, the resilient modulus and active traffic within project periods of 10, 20 and 30 years.

$\text{Log } W_{18} = Z_R \times S_o + 9{,}36 \times \log(SN+1) - 0{,}20 + \log(\Delta PSI/4{,}2 - 1{,}5) / 0{,}40 + 1094/(SN+1)^{5{,}19} + 2{,}32 \times \log M_R - 8{,}07$ (eq.5)

where h1, coating thickness ≥ SN1/a1 ; h2, thickness of the base ≥ (SN2 - (a1 x h1))/a2 ; h3, thickness of the sub-base ≥ (SN3-(a1xh1-a2xh2))/a3 ; a1 = 0.44 (HMA); a2 = 0.19 (unbound layer - with the energy of the Modified Proctor); a3 = 0.11 (loan material with CBR ≥ 18%, according to the previously equalized CBR). Table 5 shows the structure of flexible pavement according to the AASHTO standards.

Table 5 - Structure of flexible pavement

AASHTO Traffic	Design period (years)	Hot Mix Asphalt (HMA) (cm) Calcul./Adopetd	Structural Number (SN)	Unbound base layer(cm)(*) Calcul. / Adopetd	Structural Number (SN)
2,61E+06	10	7,49 / 7,5	1,30	5,46 / 15	1,71
6,11E+06	20	8,75 / 9	1,52	5,97 / 15	1,96
1,08E+07	30	9,66 / 10	1,67	6,31 / 15	2,15
NOTE(*): The minimum thickness recommended by AASHTO Guide is 15 cm					

4 ANALYSIS OF USEFUL LIFE

4.1 The FLAPS software

The FLAPS software makes use of the standard truck configuration with 8,2 tf (80KN) and a coordinate system which combines the features of application of the wheel loads with multiple non-linearity of the materials of the layers.

4.2 Elastic parameters

4.2.1 Definition of the Dynamic Modulus of asphalt

The dynamic modulus of asphalt equation, developed by "The Asphalt Institute's Thickness Design Manual" (MS-1 /1982), below, has its purpose in this context. It expresses the reaction of the coating layer to the action of transient loads by general correlations which have direct influence on the performance of the pavement.

$\text{Log } |E^*| = 5{,}553833 + 0{,}028829 \times (P_{200}/f^{\,0{,}17033}) - 0{,}03476 \times V_v + 0{,}070377 \times \eta 70^oF{,}_{10}{}^{6} + 0{,}000005 \times t_p{}^oF^{(1{,}3+0{,}49825 \times \log f)} \times P_{ac}{}^{0{,}5} / f^{1{,}1} - 0{,}00189 \times t_p{}^oF^{(1{,}3+0{,}49825 \times \log f)} \times P_{ac}{}^{0{,}5} / f^{1{,}1} + 0{,}931757 \times f^{-0{,}02774}$ (eq.6)

Where: $|E^*|$ = Dynamic Modulus of the asphalt (PSI - Pound Square Inch) ; P_{200} = #200 (%), adopted = 7; $\eta 70^oF{,}_{10}{}^{6}$ (Poises)= 29508,2 x $Pen_{77}F^{-2{,}1939}$; f (Hz) = 1/Tc ; Tc =(2 x r + 3 x z) V; r (m), V(m/s) ;$Pen_{77}F$ = f(CAP:50/70) = 60mm ; t_p = f(z) ;If Tar^oF ≥ 45,4 + 1,32 x z(inches) , t_p = -10+1,39Tar -0,52 x z ; If not, t_p =7,7 + 1 x Tar - 0,004 x z

4.3 Stress and strains acting on the pavement structures

Using the FLAPS under linear conditions, the allowable maximum surface deflection, stress at the bottom of the asphalt and the vertical deformation in the subgrade are presented in

Table 6. It was based on the CBR empirical method and the AASHTO mechanistic- empirical method.

Table 6 - Strains and Stresses acting on the pavement structures - FLAPS

	Nf (USCAE)		Nf (AASHTO)		
Reference	h (7,5 cm)	h (10 cm)	h (7,5 cm)	h (9 cm)	h (10 cm)
|E*|(kgf/cm2)	46195,05	42588,29	46195,05	43844,46	42588,29
E,base(kgf/cm2)	3500,00				
E,subgrade(kgf/cm2)	1800,00				
δ0	0,05948	0,05346	0,05948	0,05571	0,05346
εt	0,0002794	0,0002684	0,0002794	0,0002749	0,0002684
εv	0,0006714	0,0005596	0,0006714	0,0006056	0,0005596
NOTE:Adopted unbound granular material with 15 cm to all the base layer					

4.4 Strains and Stress Analysis with remaining life of the pavement

4.4.1 Maximum allowable deflection

This based on the PRO-11/79 DNIT method in the equation below.

$$\delta adm = 3{,}01 - 0{,}176 \log Nf \quad \text{(eq.7)}$$

Table 7, shows the maximum deflection values arising from the surface (PRO11/79) in hundredths of mm, and the maximum deflection values (FLAPS) in pavement structures designed by CBR method for project periods.

Table 7 -Deflections allowable and acting functions of the project period.

Reference	h=7,5 cm	h=10 cm	h=7,5 cm	h=10 cm	h=7,5 cm	h=10 cm
	Nf (USACE) - 10 years		Nf (USACE) - 20 years		Nf (USACE) - 30 years	
δadm(mm)	61,2		52,68		47,64	
δproj(mm)	59,48	53,46	59,48	53,46	59,48	53,46

4.4.2 Tensile stress at the bottom of the asphalt

Studies conducted by the Asphalt Institute of America (A.I.A.), with a confidence level is 87%, generated the equation presented in eq. 8 below that takes into account the characteristics of the asphalt layer, the dynamic modulus of asphalt layer and the tensile stresses at the bottom of the asphalt, obtained using the theory of elastic layers, in order to obtain a cracked area percentage of at least 20%.

$$Nf = C \times 18{,}4 \times (4{,}32 \times 10^{-3}) \times (1/\epsilon t)^{3{,}29} \times (1/|E^*|)^{0{,}854} \quad \text{(eq.8)}$$

where: $C = 10^M$ and $M = 4{,}84 \times (V_b / V_v+V_b) - 0{,}69)$; V_b = volume of asphalt (%) e V_V = volume of air voids (%); |E*|= Dynamic modulus of asphalt (PSI); Calibration Factor = 18,4 .

Table 8, below, shows, the admissible design traffic for each of pavement structures.

Table 8 - Traffic admissible according to the tensile stress at the bottom of the asphalt and to the project period

Reference	h = 7,5 cm	h = 9 cm	h = 10 cm
Ɛt (cm)	0,0002794	0,0002749	0,0002684
Nf(proj)	863289,39	19383343,8	21498118,3
Nf(AASHTO)	2606596,09	6109643,28	10817445,8
	0,33	3,17	1,99

4.4.3 Subgrade rupture

In studies carried out by SHELL using a confidence level of 85%, it was possible through the equation given below (eq.9) to obtain permissible traffic from the AASHTO method according to the vertical deflection limit in subgrade based on the theory of elastic layers.

$$N_{ad} = 1{,}94 \times 10^{-7} \times (1/\mathcal{E}v)^4 \qquad \text{(eq.9)}$$

Table 9, below, shows the design traffic permitted for each of the pavement structures for different project periods.

Table 9 - Admissible Traffic as a function of vertical deflection limit on the subgrade

Reference	h = 7,5 cm	h= 9 cm	h = 10 cm
Ɛv (cm)	0,0006714	0,0006056	0,0005596
Nf (Proj)	954720,8	1442308,84	1978294,33
Nf (AASHTO)	2606596,09	6109643,28	10817445,8
	0,37	0,24	0,18

5 PAVEMENT STRUCTURES ALTERNATIVES

5.1 The Semi - rigid Flexible Pavement

It is the substitution of the unbound granular base with an unbound granular base treated with cement. To minimize the reflection of cracks, it was admitted the insertion, between the Hot Mix Asphalt Concrete and the unbound granular base treated with cement, an intermediate layer of TSD (double superficial treatment with polymer), considered as having no structural function. On the other hand, the insertion of a layer of granular sub-base of relatively high capacity support (CBR ≥ 20) improved the performance of the pavement against Rutting.

The fatigue cracking of cemented base and its subsequent reflection through the asphalt layers will cause an increase in vertical stress in subgrade and its weakening by the entry of rainwater. It should be noted however, that this type of pavement structure should be applied to values of fatigue life > 20 years. Thus, Table 10, below, shows the pavement structures with respect to the concept of semi-rigid pavement.

Table 10 - Structure of semi-rigid pavement adopted

AASHTO Traffic	HMA (cm)	Design period (years)	Unbound granular base treated with cement (cm)	TDS-double superficial treatment with polymer(cm)	granular sub-base of relatively high capacity support (CBR ≥ 20) (cm)
2,61E+06	7,5	10	15	2,5	10
6,11E+06	9	20			
1,08E+07	10	30			
NOTE: Adopted to Unbound granular base treated with cement , E = 120.000 kgf.cm2 / RCS (σc) = 70 kgf/cm2 ; to granular sub-base,E = 2000 kgf/cm2.					

In order to check for the fatigue cracking of the cement base for traffic design, we adopted the model developed from the tests accelerated in situ with the Heavy Vehicle Simulator (HVS) of South Africa, with reference to the tension at the bottom of the base cement. The following equation is used to define what the Nf will be:

$Nf = F_c \times 10^{7,19*(1-SSR/8)} \times R_{pc}$, where, SSR(*strain*-strength *ratio*) = $\varepsilon_t / \varepsilon_b \times d$ (eq.10)

F_C = calibration factor, model adopted in this case = 0.43, to initial cracking at the base layer; d = adopted 1.25 , granular sub-base with high support capacity (CBR ≥ 20); R_{PC} = Pass-coverage ratio = 2.5 to highways ; ε_b = 145 (function of the simple compression Resistance of concrete). Table 11 below, shows the strains and stress of the structures.

Table 11 - Actuating strains and stress - FLAPS software

	Nf (AASHTO)		
References	h = 7,5 cm	h = 9 cm	h = 10 cm
|E*|(kgf/cm2) / μ =0,33	46195,05	43844,46	42588,29
δ0 (cm)	0,05831	0,05469	0,05225
Asphalt Ɛt (cm)	0,0002694	0,0002659	0,0002628
base Ɛt (cm)	0,00008256	0,0007759	0,0000745
Subgrade Ɛv (cm)	0,0006773	0,0006161	0,0005791
E,Base = 3500 kgf/cm2) / μ=0,20 ; E,Sub-base = 2000 kgf/cm2 / μ=0,0,44; E, subgrade = 1800 kgf/cm2 / μ=0,44			

Table 12 below, shows the permissible design traffic considering the tensile stresses at the bottom of the cemented base for semi- rigid pavement structures with project periods of 10, 20 and 30 years.

Table 12 - Admissible Traffic based on the tensile stress at the bottom of the cemented base and the project period

Reference	h = 7,5 cm	h = 9 cm	h = 10 cm
Ɛt	0,00008256	0,00007759	0,0000745
Nf (proj)	16649754,13	16649548,19	16649756,52
Nf (AASHTO)	2606598,09	6109643,28	10817445,77
	6,39	2,73	1,54

5.2 Inverted Flexible Pavement

In the case of inverted Flexible Pavement made up of Hot mix asphalt concrete, unbound granular base, unbound granular sub-base treatment with cement over subgrade, there is no difference in performance when compared to conventional flexible pavements, since the bottom of the asphalt fiber is also subject to tensile strain. However, the sub-base treated with cement will also be subject to such deformations. Thus, there is a gradual process of fatigue cracking, with its modulus of elasticity being effectively reduced with repeated traffic loads, degrading the sub-base.

Table 13 - Structure of Inverted Flexible pavement adopted

AASHTO Traffic	Design period (years)	HMA (cm)	Unbound granular base (cm)	Granular sub-base weakly cemented, CBR≥20%)(cm)	subgrade overlay (CBR>18%) (cm)
2,61E+06	10	7,5	15	10	15 to 60 (see table 3)
6,11E+06	20	9			
1,08E+07	30	10			
NOTE: Adopted granular base, E = 3500 kgf/cm2 and sub-base weakly cemented, E=2000 kgf/cm2					

Table 14 shows the strains and stress of the structures of inverted Flexible pavements.

Table 14 - Strains and stress of the structures of inverted Flexible pavements

	Nf (AASHTO)		
References	h = 7,5 cm	h = 9 cm	h = 10 cm
|E*|(kgf/cm2) / μ =0,33	46195,05	43844,46	42588,29
δ0 (cm)	0,05831	0,05469	0,05225
Asphalt Ɛt (cm)	0,0002694	0,0002659	0,0002628
Sub-base Ɛt (cm)	0,0003068	0,0002760	0,0002578
Subgrade Ɛv (cm)	0,0006773	0,0006161	0,0005791
E,Base = 3500 kgf/cm2) / μ=0,35 ; E,Sub-base = 2000 kgf/cm2 / μ=0,20 ; E, subgrade = 1800 kgf/cm2 / μ=0,44			

Table 15 also shows the tensile stresses at the bottom of the asphalt the admissible traffic for each of the pavement structures, using inverted flexible pavement alternatives.

Table 15 - Admissible Traffic according to the tensile stress in the bottom of the asphalt

Reference	h = 7,5 cm	h=9 cm	h = 10 cm
Ɛt (cm)	0,0002694	0,0002659	0,0002628
Nf (Proj)	963203,32	21626700,37	23042387,37
Nf (AASHTO)	2606598,09	6109643,28	10817445,77
	0,37	3,54	2,13

Table 16, below, shows the tensile stresses at the bottom of the weak cemented sub-base and the admissible traffic based on the semi-rigid pavement design concept.

Table 16 - Admissible Traffic based on the tensile stresses at the bottom of the weak cemented sub-base and the project period.

Reference	h = 7,5 cm	h=9 cm	h = 10 cm
Ɛt (cm)	0,0003068	0,0002760	0,0002578
Nf (Proj)	16649687,52	16649696,67	16649702,08
Nf (AASHTO)	2606598,09	6109643,28	10817445,77
	6,39	2,73	1,54

5.3 The Rigid Pavement

The rigid pavement design is based on the AASHTO Guide, where the definition of traffic is considered as the design load factor for a board thickness of 12". However, we know that this is an iterative process where, after the first evaluation of the thickness of the plate, the load factor is adjusted. Using equation 12, the slab thicknesses obtained from the AASHTO guide are shown in Table 17.

$\text{Log } W_{18} = Z_R * S_o + 7{,}35 * \log (D+1) - 0{,}06 + \log (\Delta PSI/4{,}5 - 1{,}5) / 1 + 1624 * 10^7/(D+1)^{8{,}46} + 4{,}22 - 0{,}32\ p_t * \log (Sc'*Cd\ [D^{0{,}75} - 1{,}132] / 215{,}63*J*[D^{0{,}75} - 18{,}42/(Ec/k)^{0{,}25}]$ (eq.12)

Table 17 - Structure of Rigid Pavement - AASHTO method

Design period (years)	Log(W18) f(traffic)	Rigid Pavement (cm)	
		Slab	Unbound granular Sub-base
10	6,90	24	15
20	7,27	27	
30	7,51	29	

where: So=0,29; ΔPSI = 1,7; Pt=2,5 ; Sc' = 650 PSI, Cd = 1; J=3,2; Ec = 5E+06; k = 7,2 PCI.

In a similar manner, performance analysis performed in flexible pavement design will also be done for the rigid pavement. Therefore, it is recommended that the following checks are made, which will not be shown in this work.

1. Percentage of cracked slabs (PTR≤10%)
2. Maximum Freeboard Tensile Stress
3. Gap between plates
4. Erosion at the top of the sub-base which is a function of pumping rate

6 DEFINITION OF THE ANNUAL UNIFORM EQUIVALENT COST OF MAINTENANCE (C.A.U.E.M)

This is the application of the relationship between the cost of implementation (CI) and the service life (Vs). The values for the C.A.U.E.M for each alternative considering the sizing, verification and analysis are shown below in Table 18.

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Table 18 - Comparison of costs of pavement structures - CAUEM

ALTERNATIVES	Reference table	CI (US$/m2)	Vs (years)	CAUEM (US$/m2)
1 Traditional Hot mix Asphalt	5	43,82		
Tensile stress at the bottom of the asphalt	8		4,2	10,43
Subgrade rupture	9		4,2	10,43
2 Semi - rigid flexible Pavement	10	163,67		
Tensile stress at the bottom of the asphalt	12		39	4,20
3 Inverted Flexible Pavement	13	109,41		
Tensile stress at the bottom of the asphalt	15		4,2	26,05
tensile stress at bottom of the Sub-base	16			
4 Rigid Pavement				
With Unbound granular sub-base	17	330,6	30	11,02
With Unbound granular sub-base treated with cement		357,31	30	11,91

7 CONCLUSION

Considering the *Traditional Flexible Pavement* we can conclude that in terms of maximum allowable deflection, all the structures analyzed meet the requirements. In terms of tensile stress at the bottom of the pavement, the HMA structure with a 7.5 cm thickness will not be enough. When the traffic is increased, a HMA thickness of 9cm is sufficient for a pavement life of 20 years. The same can be said for the expected traffic for 30 years, the pavement structure having a HMA thickness of 10 cm. However, through the use of the subgrade failure criterion, one can observe that such structures show a tendency of having in a short term , subgrade rupture or Rutting. This means that most pavement structures that fail before the end of the project period do not go through intervention procedures as well as being checked for the criteria above. In terms of *semi - rigid flexible pavement*, there will not be fatigue cracking of the cement base for traffic design, since the structure of the pavement is appropriately designed. The same can be said in the case of rupture of subgrade / Rutting. Therefore with this solution, the semi-rigid pavement can be a good option, even in terms of C.A.U.E.M. With the inverted pavement, a HMA of thickness 7.5cm will not be adequate considering the tensile stresses at the bottom of the pavement structure. On the other hand, a thickness of 9cm is more than adequate for an expected project life of 20 years. For a 30 year period, a thickness of 10cm will be adequate. In terms of C.A.U.E.M, the traditional flexible pavement is the best option due to its highly adaptable nature. It can also be enhanced with the appropriate overlay. However, the *semi-rigid flexible pavement* can be a good choice since its structure is properly dimensioned, as suggested in Table 10.

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