

Sensitivity Analysis of Flexible Pavement Dimensioning with AASHTO (1993) Method. Part I: Effect of Subjective Variables

Hugo Alexander Rondón-Quintana¹, Juan Gabriel Bastidas-Martínez² and Saieth Baudilio Chaves-Pabón³

¹ Ph.D., Full Professor, Facultad Medio Ambiente y Rec. Naturales, Universidad Distrital Francisco José de Caldas, Colombia.
ORCID: 0000-0003-2946-9411

² Ph.D., Facultad de Ingeniería, Universidad Piloto de Colombia, Colombia.

³ Ph.D., Facultad de Estudios a Distancia, Universidad Militar Nueva Granada, Colombia.

Abstract

One of the most currently used methods in the world for dimensioning flexible pavement structures is the one from American Association of State Highway and Transportation Officials – AASHTO, 1993. The authors of this article have found that design variables for this method can be classified in two groups named “objective” and “subjective”. The first are established by conducting field and/or laboratory studies, while the second are defined by the designer, based on recommendations established by AASHTO. Both types of variables may present a high variability, which may impact the dimensioning of flexible pavements. This study carried out a sensitivity analysis of design variables in order to evaluate the aforementioned. The influence of subjective and objective variables will be presented in two separate papers. This paper (Part I) presents the influence of variables of a subjective nature. Additionally, a brief review of the AASHTO method is shown, where subjective and objective variables are described. A general conclusion reports that the subjective character variables evaluated have a significant impact on design thicknesses and the cost of structures, especially when subgrade resistance is lowered, and when there is an increase of traffic and temperature.

Keywords: sensibility analysis, AASHTO, design, flexible pavement, subjective variables.

I. INTRODUCTION

One of the most currently used methods in Colombia and in a large portion of the world for dimensioning flexible pavement structures is the one from American Association of State Highway and Transportation Officials – AASHTO [1]. This method, developed in United States on the 60s decade, had preceding versions such as those named “provisional” AASHO [2] and AASHTO [3], and the design version of ASHTO [4]. All these versions were based on empirical results obtained in real scale tests made in testing sections during two years in AASHO Road Test (Ottawa, state of Illinois). The construction of said sections began in 1956, where the first results were registered between 1958 and 1960. A great part of them can be accessed on the Highway Research Board – HRB [5].

On the other hand, AASHTO [1] version has also been modified, introducing certain mechanistic concepts (National Cooperative Highway Research Program – NCHRP [6]), under Project NCHRP 1-37a. This new AASHTO [7] version, titled Design Guide for Mechanistic-Empirical Pavements (M-E),

has, among others, the following use limitations: i) There is little existing knowledge about the method and a limited access to the Mechanistic–Empirical Pavement Design software - MEPDG 2008; ii) the methodology uses load spectrums as traffic variable (magnitude and frequency of expected loads during pavement design period); iii) the guide requires climatic information such as hourly temperature, precipitation, wind speed and relative humidity, among others, in order to be added to a climatic model named EICM (Enhanced Integrated Climatic Model); iv) also, it needs more detailed information in comparison to previous versions, information such as the variation of the subgrade modulus based on the soil changes in moisture and the variation of asphalt mix modulus in function of temperature, load frequencies, asphalt aging, etc. during the pavement’s useful life; v) additionally, in some countries, there is neither the availability of the necessary laboratory equipment for obtaining the mechanical parameters of materials, nor the sufficient support of government institutions for the methodology’s implementation. Given the aforementioned, this new version has not been extensively used at a world-wide level, and because of such reason, AASHTO [1] is still being used in countries such as Colombia.

Now, when flexible pavement structures are dimensioned through the AASHTO [1] methodology, some variables are of a “subjective”, as others are of an “objective”. Objective variables are those derived from studies carried out on the field and/or laboratories such as subgrade and traffic. In order to determine these, prior soil and traffic studies must be carried out, respectively. Subjective variables are those defined by the designer, based on the recommendations established by AASHTO [1] methodology, which may present magnitudes in a broad range of values. This may lead to the fact that on multiple occasions, flexible pavement structure thicknesses tend to present broad ranges of values that could be allowed in order to resist loads imposed by traffic and environmental conditions. With the purpose of evidencing the aforementioned, this study conducted a sensitivity analysis for each of the variables of the [1] method, with the goal of evaluating their influence on the dimensioning of flexible pavements, as well as for their initial cost. The influence of objective and subjective variables will be presented in two separate papers. This paper (Part I) presents the influence of subjective variables, which are the present serviceability index (PSI), reliability, layer drainage coefficients and mechanical properties of the asphalt layer.

II. METHODOLOGY

Design variables will initially be described, and the values used for conducting the sensitivity analysis will be presented. First, subjective design variables will be presented, which will vary during simulations, and lastly, objective variables will be described, which must remain constant. A more detailed description of design variables may be reviewed in [8].

II.I. Subjective variables

II.I.I Mechanical properties of materials

The mechanical parameter with which the materials are characterized is resilient modulus in psi. However, when it comes to the design and dimensioning of the thickness in flexible pavement layers, these parameters are translated to structural coefficients of layers a_i , which are determined through figures presented in the AASHTO [1] design manual or in equations (1-3). When sub-index $i=1$, the structural coefficient corresponds to the asphalt surface course; $i=2$ for the unbound granular base course and $i=3$ for the unbound granular subbase course.

$$a_1 = 0.184 \times \ln(E_1) - 1.9547 \quad (1)$$

$$a_2 = 0.249 \times \log(E_2) - 0.977 \quad (2)$$

$$a_3 = 0.227 \times \log(E_3) - 0.839 \quad (3)$$

E_1 , E_2 and E_3 are resilient modulus in psi of the asphaltic concrete, the granular base and the subbase, respectively. In the process of designing flexible pavements, choosing the modulus for the asphaltic concrete is generally subjective. For its definition, the method recommends using Fig. 1, which presents a broad range of values that can be chosen by pavement designers for a reference temperature (T_{ref}) of 20°C. Based on the experience of authors and the reviewing of diverse pavement design consultantship studies, in E_1 at 20°C may vary between 3×10^5 and 4.5×10^5 psi, being a typical value 4×10^5 psi ($a_1=0.42$). In other words, in pavement design, a_1 may vary between 0.37 and 0.44, impacting the pavement structure's thickness.

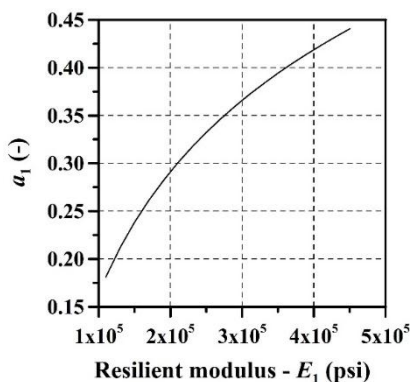


Fig. 1. a_1 and E_1 for mixing asphaltic concrete at 20°C [1]

On the other hand, in order to consider that E_1 changes with temperature (T , in °C), designers use diverse equations for

obtaining this value. Some of the most used are presented in equations (4-7) [9-13]. E_{ref} is the modulus obtained at T_{ref} .

$$E_1 = \frac{E_{ref}}{e^{-0.072(T_{ref}-T)}} \quad (4)$$

$$E_1 = E_{ref} \times \left[1 - 2.2 \times \log\left(\frac{T}{T_{ref}}\right) \right] \quad (5)$$

$$E_1 = \frac{E_{ref}}{10^{-0.0275(T_{ref}-T)}} \quad (6)$$

$$E_1 = \frac{E_{ref}}{e^{-0.06(T_{ref}-T)}} \quad (7)$$

The process of choosing E_2 and E_3 is simpler. Designers in Colombia use the criterion of considering the minimum values of CBR (Californian Bearing Ratio) demanded by the material specifications for roads of the Instituto Nacional de Vías - INVIAS [14], for its determining. For the case of roadways with low (NT1) and medium (NT2) traffic volumes, the minimum CBR for bases and subbases is 80% and 30%, respectively, while for high traffic volumes (NT3), said values are 95% and 40%. With these magnitudes, and considering the figures recommended by [1], designers generally use $E_2=28000$ psi and $E_3=15000$ psi for NT1 and NT2, and $E_2=30000$ psi and $E_3=17000$ for NT3.

As previously mentioned E_1 at 20°C may vary between 3×10^5 psi ($a_1=0.37$) and 4.5×10^5 psi ($a_1=0.44$), being a typical value of 4×10^5 psi ($a_1=0.42$). These should be the modulus used for conducting the sensitivity analysis. E_1 and the thickness of the asphaltic concrete surface course are inversely proportional. Given the aforementioned, when using these three magnitudes, pavements will be dimensioned within a range where thickness will be lowest – B ($E_1=4.5 \times 10^5$ psi), a middle value – P ($E_1=4 \times 10^5$ psi) and a higher value – A ($E_1=3 \times 10^5$ psi). Sensitivity analysis will be conducted for pavements subjected to average annual middle temperatures (TMAP) of 10, 20 and 30°C. In order to consider the effect of temperature on mix stiffness, values presented on Table 1 will be used for TMAP=10°C and 30°C, which are the averages obtained when using equations (4-7).

Table 1. E_1 values in simulations

TMAP (°C)	E_1 (psi)		
	B	P	A
10	8.35×10^5	7.42×10^5	5.57×10^5
20	4.5×10^5	4.0×10^5	3.0×10^5
30	2.45×10^5	2.18×10^5	1.63×10^5

II.I.II Loss of serviceability index, reliability and drainage coefficients

Total loss of serviceability ($\Delta PSI = P_o - P_f$), is a subjective index that considers the initial state of serviceability (P_o) and final serviceability (P_f) of pavement that will be designed and used in construction. For its determining, each state serviceability state will be evaluated in a quantitative manner by assigning a value between 0 and 5. Once the value of 5 is assigned, it is assumed that the pavement is in perfect state of serviceability. Generally, the process begins with an initial value for

pavements of P_o between 4.0 and 4.5 (good state for a roadway) and the lowest value that can be tolerated by roadway users is determined before it is necessary to repair or rebuild pavement P_f . This last one varies with the roadway's level of importance (between 1.5 and 3.0, see Table 2). This means that ΔPSI may vary during the useful life of a pavement, between 1.0 and 3.0, and on that range, flexible pavement structures may greatly vary in design thicknesses. The recommended value of P_o in the AASHTO roadway experiment for flexible pavements is of 4.2, and this will be the one used for conducting sensitivity analysis. Generally, the use of P_f below 2.0 is not recommended, even if the traffic is low.

Table 2. Final serviceability, P_f [1]

Type	P_f
Highway	2.5 – 3.0
Important road	2.0 – 2.5
Industrial zones	
Urban pavement (important)	1.5 – 2.0
Urban pavement (secondary)	1.5 – 2.0

Reliability (R) considers the degree of uncertainty presented during the process of estimating design variables, but mainly, it considers the traffic. Thus, it can be seen as a security factor. Table 3 presents R values recommended by the design method. It is possible to observe that on this table, the designer has a broad range of choosing R values in which pavement structures can significantly vary their thicknesses.

Table 3. Recommended reliability levels R

Classification	R [%]	
	Urban	Rural
Interstate and other	85.0 –	80.0 – 99.9
Principal arterials	80.0 –	75.0 – 95.0
Collectors	80.0 –	75.0 – 95.0
Local	50.0 –	50.0 – 80.0

Considering tables 2-3, the values of P_f and R show on Table 4 were chosen for this study. These values were established based on the author's experience and in reviewing several consultancy studies on pavement design. Additionally, these were divided, according to the manner in which they will impact design thickness, just as was done for the case of E_1 (B, P, A are low, average and the highest thickness that will be obtained, respectively). W_{18} is the traffic variable (described later).

Table 4. Recommended values for P_f and R to be used in sensitivity analysis

W_{18}	P_f (-)			R (%)		
	B	P	A	B	P	A
5×10^5	2.0	2.5	2.5	80	90	95
5×10^6	2.0	2.5	2.5	80	90	95
2×10^7	2.5	2.5	3.0	85	90	95

Drainage coefficients (m_i) consider that in high precipitation climates, untreated granular layers of the base and subbase can

be exposed to increase their water content in flexible pavements, which generates a reduction of their structural resistance. The drainage coefficient m_1 (asphaltic layer) is considered equal to 1.0, given that it assumed to be waterproof. In order to determine these coefficients in granular layers, it is initially necessary to subjectively establish, the drainage quality of granular material (see Table 5). Generally, in the process of dimensioning flexible pavements, designers tend to use drainage qualities that are "Fair" to "Very poor" in Colombia. This takes place because granular layers of the base and subbase use well-graded aggregates and are densified in such a point that air voids are scarce, which does not allow for easy evacuation of water in case it reaches into the material. In this condition, if water comes into contact with the untreated granular layer, it will tend to retain water, experiencing its worst behavior under cyclic load. When using any of the two criteria ("Fair" to "Very poor"), pavement thickness, and mainly layers of base and subbase, can vary significantly. After defining drainage quality, it is necessary to calculate time percentage (t) in the pavement structure that is exposed to moisture levels close to saturation. t is calculated as a percentage relationship between the number of average rainy days in the area where the pavement structure will be built and 365 days of the year. With both parameters (drainage quality and t), drainage coefficients are calculated m_2 (base) and m_3 (subbase), using Table 6. Generally, the coefficients of both layers are assumed to be equal ($m_2 = m_3$).

Table 5. Drainage Quality [1]

Drainage Quality	Water removed within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	Water will not drain

Table 6. Recommended values for m_i to be used in granular bases and subbases [1]

Drainage Quality	t			
	Less than 1%	1 – 5%	5 – 25%	> 25%
Excellent	1.40 – 1.35	1.35 – 1.30	1.30 – 1.20	1.20
Good	1.35 – 1.25	1.25 – 1.15	1.15 – 1.00	1.00
Fair	1.25 – 1.15	1.15 – 1.05	1.00 – 0.80	0.80
Poor	1.15 – 1.05	1.05 – 0.80	0.80 – 0.60	0.60
Very poor	1.05 – 0.95	0.95 – 0.75	0.75 – 0.40	0.40

The $m_2 = m_3$ values chosen for the analysis were 0.9 and 0.7. The first value is typical for granular layers with a fair drainage quality and $t=15\%$ (see Table 6). The second value is chosen by the designer for the same t , however, assuming a poor drainage quality. In general, very poor drainage quality is not chosen in pavement design. Just as for P_f and R , m_i values were divided according to the way in which they will impact pavement thickness ($m_2 = m_3 = 0.9$ for conditions B, P and 0.7 for A).

II.II Objective variables

II.II.I Traffic

The traffic (W_{18}) is characterized through the number of equivalent single axle loads number (ESALs) of 18 kips (8.2 tons). The following values were chosen for conducting simulation: $W_{18}=5 \times 10^5$, 5×10^6 and 2×10^7 . The first two are limits established by INVIAS [14] for NT1 ($W_{18} < 5 \times 10^5$) and NT3 ($W_{18} > 5 \times 10^6$).

II.II.II Subgrade

Subgrade is characterized through its resilient modulus (E_{subr}). Given that in practice, this variable's estimation is experimentally difficult to attain, be it because of costs, lack of equipment or finding the correct definition of stress trajectories, what is traditionally done with the variable, is to correlate it to the CBR value through equation (8).

$$E_{subr} = 1500 \times CBR \text{ (psi)} \quad (8)$$

The values taken from CBR subgrade for conducting simulations were 3% and 10%. Using equation (8) recommended by [1], the magnitudes of E_{subr} are 4500 psi and 15000 psi, respectively. Values lower than 3% were not chosen, given that there was a requirement for improving the subgrade.

II.III. Costs analysis

Once the sensitivity analysis has been carried out, the cost per m^2 for each of the dimensioned structures was estimated. This was only considered in cost analysis for materials (including transport, extension, compaction, labor force and equipment). Reference unit prices for calculations were the ones recommended by Instituto de Desarrollo Urbano - IDU [15] for the city of Bogotá D.C. Dollar price is for February of 2020.

III. RESULTS OF SIMULATIONS

Simulation results were presented in Figs. 2-6. h_1 refers to the thickness of the asphalt layer. With the purpose of evaluating the results of figures, base thicknesses (h_2) and subbase (h_3) were kept constant for each analyzed temperature, depending on the subgrade and the traffic (see Table 7). It is boldly observable in Fig. 2, that there is an increase in h_1 when there is an increase of $TMAP$ and W_{18} , and a reduction of CBR in the subgrade.

Table 7. Values of h_2 and h_3

CBR (%)	W_{18}					
	5×10^5		5×10^6		2×10^7	
	h_2 (cm)	h_3 (cm)	h_2 (cm)	h_3 (cm)	h_2 (cm)	h_3 (cm)
3	15	30	20	40	30	50
10	10	10	15	20	15	30

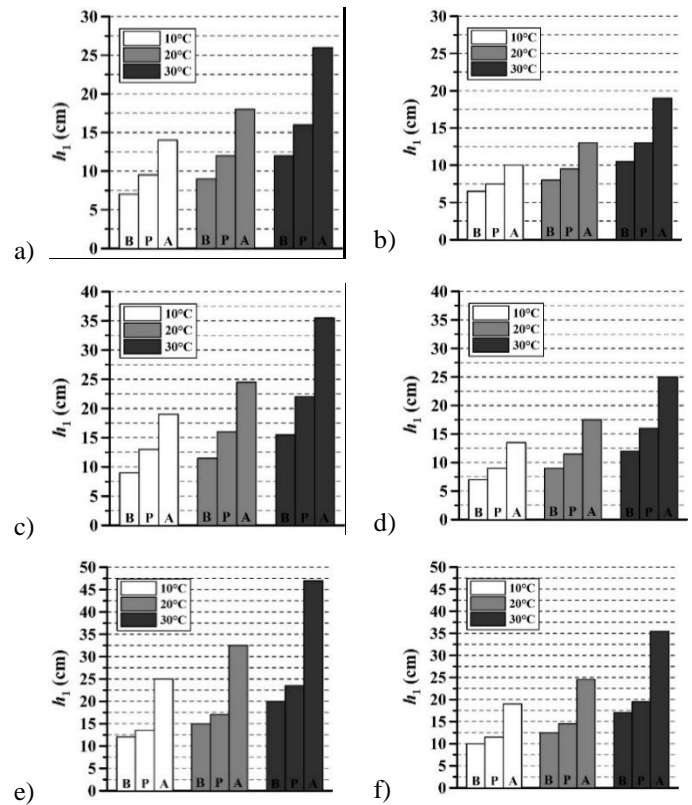
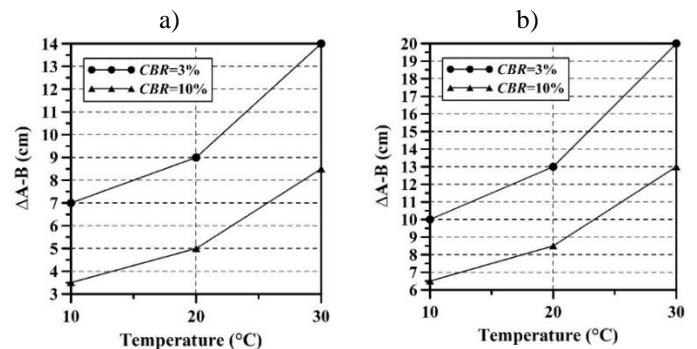


Fig. 2. a) $W_{18}=5 \times 10^5$, CBR=3%; b) $W_{18}=5 \times 10^5$, CBR=10%; c) $W_{18}=5 \times 10^6$, CBR=3%; d) $W_{18}=5 \times 10^6$, CBR=10%; e) $W_{18}=2 \times 10^7$, CBR=3%; f) $W_{18}=2 \times 10^7$, CBR=10%

Fig. 3 shows the maximum variability range possible for h_1 (difference of thicknesses in the asphaltic layer between condition B and A, named $\Delta A-B$) for each W_{18} analyzed. Fig. 4 shows the influence of said variability in terms of costs. A notorious variability in $\Delta A-B$ can be observed (both in h_1 thickness as in the cost of dimensioned structures) when the values of $TMAP$ and W_{18} increase and the subgrade's CBR is reduced. In flexible pavement design, $\Delta A-B$ could vary between 3.5 cm (approximate cost variation of 5.4 US\$/ m^2) and 27 cm (approximate cost variation of 41.9 US\$/ m^2) for the case where variability range is lower ($TMAP=10^\circ C$, $W_{18}=5 \times 10^5$, CBR=10%) and higher ($TMAP=30^\circ C$, $W_{18}=2 \times 10^7$, CBR=3%), respectively. In other words, in the moment of dimensioning flexible pavements, the range of thicknesses and costs for structures is very broad.



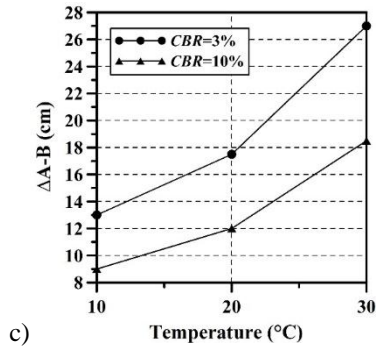


Fig. 3. Variability range of $\Delta A-B$ para a) $W_{18}=5 \times 10^5$; b) $W_{18}=5 \times 10^6$; c) $W_{18}=2 \times 10^7$

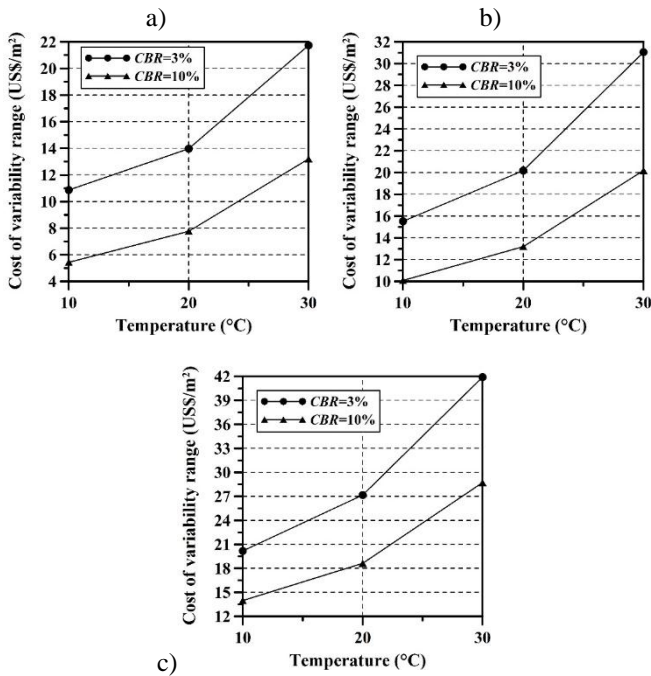


Fig. 4. Cost associated to variability range of $\Delta A-B$ for a) $W_{18}=5 \times 10^5$; b) $W_{18}=5 \times 10^6$; c) $W_{18}=2 \times 10^7$

On the other hand, just as was previously mentioned, pavement designers generally use values assigned for condition P. With the purpose of comparing this condition with conditions B and A, Figs. 5-6 graph the difference of thicknesses h_1 between A and P ($\Delta A-P$) and between B and P ($\Delta B-P$), respectively. Fig. 5 shows a notable increase of $\Delta A-P$ as there is an increase of TMAP and W_{18} , and a reduction of CBR in the subgrade. In flexible pavement design, $\Delta A-P$ could vary between 2.5 cm (approximate cost variation of 3.9 US\$/m²; increase in cost for condition A with relation to a P of 20.5%) to 23.5 cm (approximate cost variation of 36.5 US\$/m²; increase in the cost of condition A with relation to a P of 55.7%) for the case where variability range is lower ($TMAP=10^\circ C$, $W_{18}=5 \times 10^5$, $CBR=10\%$) and higher ($TMAP=30^\circ C$, $W_{18}=2 \times 10^7$, $CBR=3\%$), respectively. For the case of $\Delta B-P$, the differences are lower and there is no clearly defined trend. The maximum difference reported between both conditions is of -6.5 cm (Approximate cost variation of 10.1 US\$/m²; cost reduction of condition B with relation to a p of 18.9%). These lower magnitudes of $\Delta B-P$ in comparison to $\Delta A-P$ are given because designers generally

assume magnitudes for subjective variables in P that are very similar to those of condition B, given that they tend to dimension using the main criterion of construction of less expensive structures.

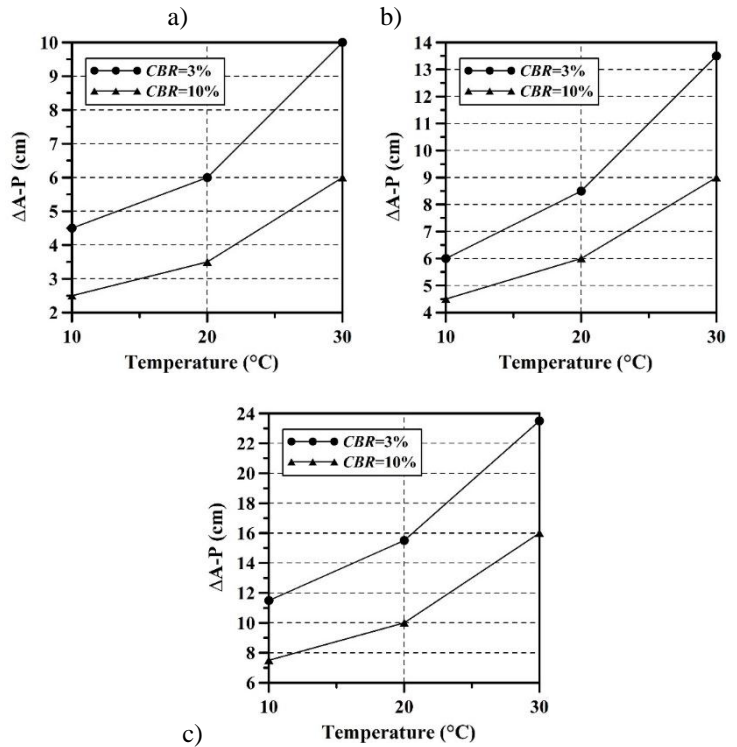


Fig. 5. $\Delta A-P$ for a) $W_{18}=5 \times 10^5$; b) $W_{18}=5 \times 10^6$; c) $W_{18}=2 \times 10^7$

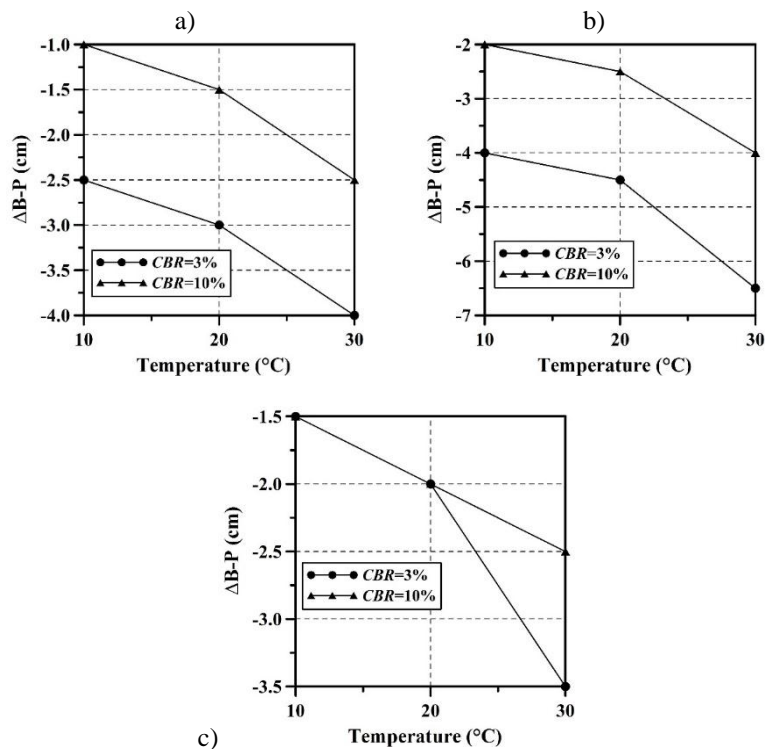


Fig. 6. $\Delta B-P$ for a) $W_{18}=5 \times 10^5$; b) $W_{18}=5 \times 10^6$; c) $W_{18}=2 \times 10^7$

IV. CONCLUSIONS

This study conducted a sensitivity analysis of each of the “subjective” variables in the AASHTO [1] method, with the purpose of evaluating its influence in dimensioning of flexible pavements and their initial cost. The following are the main conclusions.

- The range of possibilities the designer is exposed to at the moment of choosing the most appropriate values for E_1 , P_f , R y m_i have a significant impact in design thicknesses and costs for structures, especially when the subgrade’s CBR is reduced and when the temperature and the traffic increase.
- For the condition in which the impact of subjective variables is lower ($TMAP=10^\circ\text{C}$, $W_{18}=5 \times 10^5$, $CBR=10\%$), the thickness of the asphalt layer may vary in its designs in 3.5 cm. which entails an approximate variation in the cost of structures of 5.4 US\$/m². However, for the condition of higher impact ($TMAP=30^\circ\text{C}$, $W_{18}=2 \times 10^7$, $CBR=3\%$), said design thickness may vary in 27 cm, entailing an approximate cost variation of structures of 41.9 US\$/m². In other words, in the process of designing flexible pavements, thickness can increase between one or other condition in a measure of 7.71 times, while cost can increase 7.76 times.
- For the case of $TMAP=10^\circ\text{C}$, $W_{18}=5 \times 10^5$ and $CBR=10\%$, when a designer defines subjective variables, assuming high safety factors (where layer thicknesses will be the highest; named condition A in this study), the asphalt layer in designs can be 2.5 times superior in relation to the condition of generalized definition for said variables (where layer thicknesses are those typically obtained in pavement design; named condition P in this study), generating an approximate increase in structure cost of 20.5%. For $TMAP=30^\circ\text{C}$, $W_{18}=2 \times 10^7$ and $CBR=3\%$, the thickness in the asphalt layer can be 23.5 cm superior, entailing an approximate increase in the cost of structures of 55.7%.
- When a designer defines subjective variables assuming the condition where layer thicknesses will be the lowest possible (named condition B in this study), the asphaltic layer’s thickness does not vary with a clearly defined trend with relation to $TMAP$, W_{18} and CBR , when being compared to condition P. In the process of designing the variability of the asphaltic layer’s thickness in B with relation to P, it can vary between -1.0 and -6.5 cm, which represents a maximum cost reduction of condition B in relation to P of 18.9%.
- The lowest differences between P and B compared with those obtained between P and A are because designers tend to dimension flexible pavement structures using the main construction criterion of building less costly structures on a short term. However, designing these roadway structures has a high degree of uncertainty when variables are estimated (changing conditions on the soil, traffic, climate, mechanical properties of materials, among others) and in this regard, it is possible for them to be sub-dimensioned when: i) conditions P and B are being used; ii) when temperature increases and becomes high; iii) the magnitude and ESALs that circulate through the roadway being high; iv) resistance of the subgrade is lowered or is low. Results obtained in the sensitivity analysis of this study evidence this aspect.

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