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Revisiting the Drainage Coefficient of the AASHTO93: A Method for Improving Local Flexible Pavements Design

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Abstract: The AASHTO93 guide is widely used in pavement design and is considered the primary reference in road construction and management for many countries worldwide. One important parameter used in the AASHTO93 design method is the drainage coefficient, which considers the impact of saturation on the stiffness of granular materials. However, selecting this parameter can be associated with high uncertainty in practice. With the knowledge gained from applying this methodology and advancements in current research on granular materials, it is possible to revise this parameter to achieve optimal results. This work aims to review the basics of the drainage coefficient and introduce a more logical approach to define the proper parameter for local conditions. The goal is to enhance pavement design practices in administrations that utilize the AASHTO93 method, particularly those with no intention of transitioning to advanced techniques like the mechanistic-empirical approach or where alternative approaches are still minimal.
Keywords: Granular Materials; Pavements; Performance; Optimization; Drainage.

1. Introduction

The pavement is a multi-layered linear structure whose purpose is to allow vehicles to travel comfortably, economically, and safely while adequately protecting the subgrade and the layers of the different materials that constitute the structure and interact with each other as part of the system. Pavements act as an interface between traffic, climate and the underlying soil, and have a dual purpose: from top to bottom, it distribute the load, and from bottom to top, it mitigates various geotechnical effects [1]. The typical layer system in a flexible pavement consists of an asphalt concrete (AC) surface course, a base course of untreated aggregates, and a sub-base course generally of local untreated materials. This layer system is supported by moderately compacted natural soil (subgrade). The thicknesses required for each layer of the structure vary greatly depending on variables such as material properties, the number and configuration of vehicle loads, environmental conditions, and the expected service life of the structure [1,2].

Pavement design and analysis have traditionally been performed using empirical or semi-empirical methods, with the AASHTO93 method being the most widely used approach. However, as it is primarily an empirical method, any consideration of new materials or loading conditions requires calibrations that can be both complicated and costly. Mechanistic-empirical (M-E) approaches are increasingly being adopted for pavement analysis. M-E methods use structural response models to calculate stresses, strains, and displacements resulting from traffic loading and environmental factors. These responses are then used in performance models to assess damage accumulation for specific failures, such as fatigue cracking or structural rutting [3,4].

Although M-E methods offer several advantages, such as improved accuracy, flexibility, and cost-effectiveness, they require detailed and comprehensive input data, which can sometimes be challenging. Adopting the M-E design approach by many administrations is still a developing process. In some cases, the M-E principle is only partially applied in the verification stage, where pavement layer thicknesses are determined using AASHTO93.

The AASHTO93 method has been updated through the years to reflect new materials, traffic loads, and design concepts. Despite its empirical nature, this method has proven effective in designing and analyzing pavements for many years. The AASHTO93 method represents the bearing capacity of a pavement using a hypothetical parameter called structural number (SN), which is calculated with the following Equation (1):

$$SN = \frac{1}{2.5}\sum_{i=1}^{n} h_i \times a_i \times m_i \tag{1}$$

Where SN is the pavement structural number, a_i is the structural coefficient of layer i, m_i is the drainage coefficient of layer i, and h_i represents the thickness of layer i in cm. The design objective is to match the pavement's SN (Equation (1)) with a required structural number, calculated as a function of soil type, road type, and expected traffic during the pavement's service life [5].

Equation (1) reveals that a representation of pavement capacity (SN) requires three variables: (i) layer thickness, h_i, which is determined as part of the design; (ii) material stiffness, a_i, which is correlated with the material's modulus; and (iii) the drainage characteristics, m_i, which vary as a function of saturation conditions and the material itself. Among these variables, the least researched is the coefficient representing the drainage capacity of the materials. With the experience accumulated over the years, it is becoming evident that some parameters in the AASHTO93 methodology must be reevaluated to optimize pavement designs in local conditions. This re-evaluation can also support the transition to modern methods, such as those based on M-E criteria.

This work aims to introduce a methodology for selecting the drainage coefficient (m_i) of the AASHTO93 that accurately reflects the specific conditions of the base and subbase layers at the local level. The methodology was developed by complementing the field test considered during the formulation of the AASHTO93 with recent research results on granular materials and the Venezuelan experience in pavement performance. The proposed methodology can contribute to developing more efficient and sustainable pavement design practices in developing countries.

2. Background

The AASHTO93 method applies a service life approach based on resistance to equivalent load repetitions using Equation (2).

$$\log W_{80} = Z_R S_0 + 9.36 \log(SN+1) - 0.2 + \frac{\log\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.3 \log\left(\frac{M_r}{0.007}\right) - 8.07 \quad (2)$$

Where, W_{80} is the number of 80kN axle repetitions that the structure can withstand before failure, Z_R is the standard deviation, S_0 is the combined standard error of traffic prediction and pavement performance prediction, ΔPSI is the difference between the initial level of service index (P_0) and the terminal level of service index (P_t), and M_r is the effective soil resilient modulus in MPa.

All the parameters used in Equation (2) are typically known during the pavement design phase, except for the pavement's structural number (SN). Equation (2) must be solved to determine the SN value required for the estimated traffic (W_{80}). This can be done using an iterative method or with the help of existing nomograms provided in the AASHTO93 guide [5]. Once the required SN value has been found, it is possible to calculate the thickness of each pavement layer by converting SN into thickness using Equation (1). The pavement structure is satisfactory when the SN value calculated by equations (1) and (2) are equal.

The pavement's structural capacity, as represented by Equation (1), depends on three variables, of which two are known (a_i and m_i) and one variable is to be determined (h_i). While the analysis of the a_i coefficient is not covered in this paper, the literature does provide information on how to define and correlate this parameter [1,2,4–6]. The layer structural coefficient (a_i) has a significant limitation, as it does not consider the seasonal variation of the layer stiffness, i.e., the effect of water saturation on the material's performance in different climatic conditions or during the pavement's service life. To account for this, the second variable of Equation (1), the drainage coefficient (m_i), is used to apply this consideration to the pavement's structural capacity. It is essential to note that the m_i coefficient only considers the effects of drainage on the unbound granular base and subbase, respectively, and does not address the effects of moisture in asphalt or other stabilized layers. In the same way, the effects of changes in saturation in the subgrade are considered in the effective subgrade modulus, which is an input to Equation (2).

The AASHTO93 offers a framework for selecting the mi coefficient through the use of Table 1, which requires two pieces of information: (i) the climatic condition of the project site and (ii) the drainage quality of the structure, i.e., the degree to which water entering the pavement is drained. Table 2 determines the pavement drainage quality by considering the drainage quality of the granular layer (base or subbase) and the subgrade. Table 1 and Table 2 are used to determine appropriate m_i values for different climatic conditions and levels of drainage quality for the granular and subbase materials.

According to the AASHTO93 guide, the values of the m_i coefficient depends on the expected annual precipitation and the prevailing drainage conditions in the area. In contrast, the m_i value from the experimental tests used as a reference for developing the AASHTO93 method remains constant at 1, regardless of the type of material used [5]. It is important to note that these parameters specifically apply to unbound base and subbase materials.

Table 1. Recommended m_i values for unbound bases and subbases of flexible pavements [5]

Drainage quality (pavement)	% of time with the structure near saturation			
	< 1%	1% to 5 %	5% to 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

Table 2. Pavement drainage quality [5,7]

Drainage quality (subgrade)	Drainage quality of the granular layer (Base or Subbase)				
	Excellent < 2 h[a]	Good 2 h to 24 h	Fair 24 h to 168 h	Poor 168 h to 720 h	Very poor > 720 h
Good	Excellent	Good	Fair	Fair	Fair
Fair	Good	Fair	Fair	Fair	Poor
Poor	Fair	Fair	Fair	Poor	Very poor
Very poor	Poor	Very poor	Very poor	Very poor	Very poor

[a] Drainage time to 85% saturation

The AASHO Test Track, also known as the AASHO Road Test, was built in Ottawa, Illinois, during the 1950s as a research facility. Its purpose was to conduct a series of experiments to study the performance of various types of pavements and road materials under controlled conditions [8]. This was done to collect data on the performance and durability of pavements under different climatic and traffic conditions. The results obtained in this research have been used to establish the different parameters considered in the AASHTO93 guide, including the drainage coefficient [7]. In addition, these results have been used to calibrate performance models used in a mechanistic-empirical pavement design approach [2].

Richardson et al. [7] extensively reviewed the criteria used to develop the drainage coefficients in the AASHTO93 guide. Richardson et al. [7] reported some discrepancies with the description of drainage quality associated with the AASHO experimental tests and the considerations described as referential for m_i coefficients. Specifically, they argued that given the base and subbase materials and the type of subgrade in the experimental test sections, the drainage quality should have been classified as 'poor' drainage (as per Table 2). However, it was instead taken as 'fair' drainage, a discrepancy that would significantly impact the values recommended by the AASHTO method, as described in Table 1. In addition to the above, Richardson et al. [7] also argued that the drainage time was incorrectly referred to as the time to reach 50% saturation. According to the authors, the correct value is the drainage time to a drainage degree of 50%.

From the above discussion, it can be inferred that resources for conducting local calibrations of the AASHTO93 method are limited. This is especially true for tropical and subtropical countries with different climatic conditions and soil types compared to those encountered during the experimental tests that led to the recommendations for structural pavement design by AASHTO.

Jugo [9] proposed a set of recommended m_i values for different zones in Venezuela (see Table 3). They were developed based on each zone's prevailing soil and climate conditions to ensure the values represented the local conditions. However, Venezuela's climate conditions have changed with time, and more precise information is available. Malizia [10] updated the boundaries of each climatic zone to consider in the pavement design and analysis (see Figure 1), which has made it possible to maintain the use of the coefficients proposed by Jugo [9] with a little more precision.

Figure 1. Climate zones of Venezuela based on Thornthwaite indices [10].

Table 3. Recommended m_i values for Venezuela [9]

Drainage quality (base or subbase)	Climate zone[a]			
	XII	IX	II, VII, VIII, X, XI	I, III, IV, V, VI
Excellent	1.20	1.20	1.20	1.20
Good	1.20	1.20	1.10	1.00
Fair	1.20	1.10	0.90	0.80
Poor	1.10	0.90	0.80	0.80
Very poor	1.00	0.85	0.80	0.80

[a] According to Figure 1.

Although the approaches described above provide practical benchmarks for determining the m_i coefficient, it should be noted that they may not fully account for the impact of moisture changes over the seasons nor the effect of material-specific factors on saturation susceptibility and performance. As a result, it can be suggested that a wide range of conditions are being consolidated into a single factor, which can lead to uncertainty when using this approach. This limitation underscores the need to consider the factors influencing the m_i coefficient to achieve more accurate and efficient results in pavement design.

3. Quantifying the impact of drainage on the performance of granular material

Drainage coefficient (m_i) plays a crucial role in accounting for the drainage capacity and saturation levels in the performance of granular materials. Although the AASHTO93 guide provides reference m_i values, its formulation lacks clarity and has been criticized [7,11]. However, understanding the principles and analysis criteria underlying the mi coefficients in the AASHTO93 guide makes it possible to develop a methodology to define coefficients more suitable to represent local material and climate conditions. The following sections present a methodology to define m_i values according to the material granulometry and expected performance in different saturation conditions.

3.1 Considerations regarding particle size and gradation

The drainage quality, defined by a reference drainage time, is essential in characterizing pavement materials performance. Table 4 shows the references provided in the AASHTO93 guide to characterize the drainage quality of granular materials.

Table 4. Drainage quality classification based on drainage time and permeability [5].

Drainage quality	Drainage time, t (hours)	Permeability required, k (10^{-4} cm/s)
Excellent	≤ 2	≥ 2470
Good	2 - 24	140 - 2460
Fair	24 - 168	18 - 140
Poor	168 - 720	5 - 18
Very poor	≥ 720	≤ 5

As seen in Table 4, the permeability of the material, characterized by its permeability coefficient (k), is a binding indicator in defining the drainage quality of the granular material. The permeability of granular materials can be directly related to the grain size distribution [12,13]. Previous research has already correlated the permeability coefficient with the granulometry of soils and materials [14–17]. Multiple references argue that correlating permeability with grain size properties allows an adequate estimation parameter in the design phase where field measurements are unavailable [7,11,18–20]. This correlation is attributed mainly to larger particles in the material creating larger pores and voids, which allow for easier fluid flow and, hence, higher permeability.

Heredia [19] proposed different characteristic grain size curves that can be associated with a probable m_i coefficient. This reference graph is based on literature compilation and the calculation of the permeability of the granulometric curves by analytical methods. The maximum fines content (smaller than 75 µm) was set to 10%, following the values considered in the AASHTO93 guide [5], and highly draining filter materials were included as an extreme reference curve. The particle sizes associated with different drainage qualities were discretized using the reference values in Table 4. The referential m_i coefficients for different drainage qualities are based on the guidelines in AASHTO93, considering between 5% and 25% probability of being close to saturation of the material (see Table 1). This range was chosen because it represented the average conditions expected in tropical and subtropical countries [6]. For materials with 'very poor' drainage quality, m_i coefficients of 0.7 or less were considered, while coefficients of 1.2 or greater were used for those with 'Excellent' drainage quality. For highly draining materials ($k \geq 0.71$ cm/s), the maximum m_i value of 1.4 was assigned, as specified in the original AASHTO work.

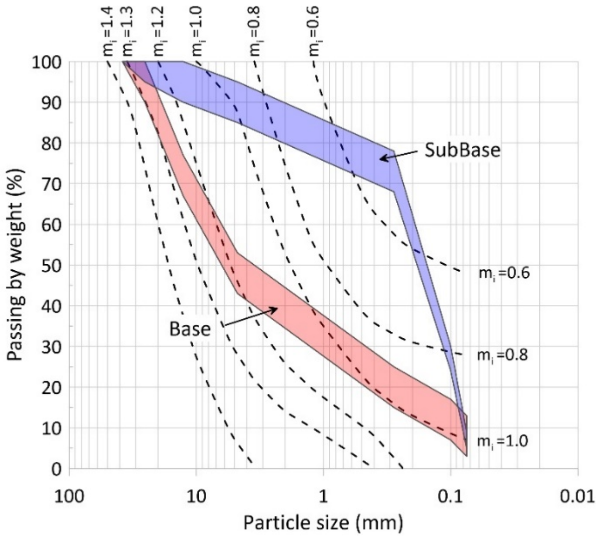

Figure 2. Permeability coefficients and m_i coefficients associated with different granulometric curves.

Figure 2 shows the curves proposed by Heredia [19], with shaded regions representing the particle size range for the AASHO test base and subbase materials [8]. The base course's granulometric range is within the "good" drainage region, predominantly below the line m_i=1.0 (ranges of m_i between 0.9 and 1.3). This can be considered consistent with that described in the AASHTO93 guide. In the case of the subbase layer, the grain size distribution is very different from the proposed reference curves, making it difficult to characterize the drainage using this tool alone. Determining m_i coefficients, relying solely on granulometric bands, may not be practical. For materials where the granulometric curve intersects two or more bands, the value of m_i to be adopted would be up to the engineer, leading to variability in the criterion.

To overcome the limitations of using particle size bands to associate a typical m_i coefficient to a material, an option is to adopt a ternary representation of the particle sizes. This type of representation categorizes granulometry based on its gravel content, sand content, and fines. Figure 3 is a ternary diagram that corresponds to the information illustrated in Figure 2. This diagram enables the identification of the appropriate m_i coefficient based on the granulometric composition. The gravel content is represented by the portion retained on sieve #4 (4.75mm), the sand content is represented by the proportion retained between sieve #4 (4.75mm) and sieve #200 (75µm) and the fines content is represented by the amount of material passing the sieve #200 (75µm). By utilizing the proportions of these three particle sizes, the ternary diagram indirectly accounts for the uniformity of the grain size curve. The equations to calculate the m_i coefficient for each zone presented in Figure 3 are shown in Table 5. The m_i coefficient calculated using these tools will be referred to as m_0 hereafter.

Table 5. Calculation of m_0 coefficient.

Zone	m_0
Zone 2	$m_0 = 1.163 \times e^{-0.026 \times F}$
Zone 3	$m_0 = 1.260 \times e^{-0.023 \times F}$
Zone 5	$m_0 = -0.008 \times F + 0.80$
Zone 1, 4 and 6	No information is available to determine m_0.

In Table 5, the variable F is the fines content. For zones 1, 4 and 6 there is not enough information to establish correlations with m_0 values. It must be noted that m_0 only considers the influence of granulometry and not other factors, such as regional climatic conditions or the type of subgrade. Thus, the applicability is limited to base and subbase materials exposed to the same climatic conditions as those used in the AASHO test. The following section will explore how environmental factors can modify the estimated m_0 coefficient.

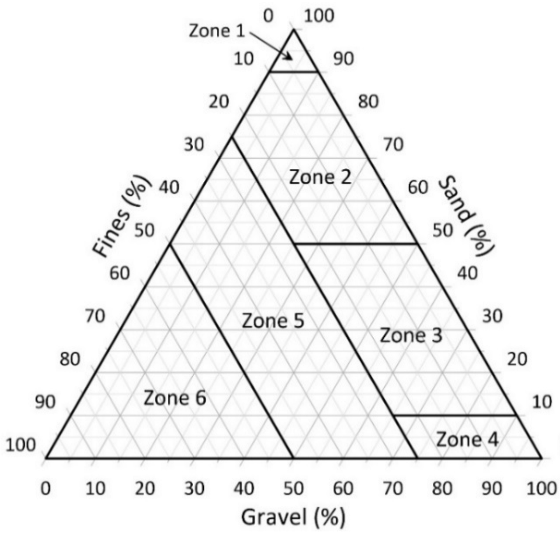

Figure 3. Ternary representation of different zones associated with the drainage coefficient m_i.

3.2 Considerations regarding saturation conditions

As described in previous sections, the m_i coefficient can be associated with the capacity of the material to drain water and with its environmental susceptibility, notably the saturation level. Both conditions are closely connected as they define how the pavement performance will be impacted by moisture affecting the granular layers. In other words, a sensitive material but rarely exposed to adverse saturation conditions may have a m_i coefficient that does not overly penalize the structural capacity of the layer by associating it with an unlikely scenario. Similarly, a material with low susceptibility but subjected to high saturation levels for extended periods should have a m_i coefficient that penalized the reference performance of the material. Since the conditions that lead to a saturated state are difficult to generalize, this study will focus on the influence of changing saturation levels on the mechanical behavior of granular materials.

The mechanical performance of granular materials is significantly influenced by saturation, which has been extensively studied in recent decades [15,16,21–23]. When a granular material is saturated, its strength and stiffness are reduced due to increased pore water pressure, which reduces the effective stress on the grains. This makes the material more prone to deformation and failure under loading. Moreover, the presence of water in the pore spaces can cause the particles to become lubricated, leading to a reduction in interlocking between particles and further contributing to the reduction of strength and stiffness. The degree of the saturation effect varies depending on factors such as the soil type, degree of saturation, loading and drainage conditions. Pérez-González et al. [24,25] proposed a normalized model that considers the impact of dry density and saturation level on the permanent deformation performance of materials subject to a high number of loading repetitions, described by equation (3).

$$N_c = \frac{\dot{\varepsilon_p}}{\dot{\varepsilon}_p^{ref}} = 0.0155\, S_w - 0.0047\, S_w \left(\frac{\rho_d}{\rho_w}\right) \tag{3}$$

Where, N_c is ratio between the permanent strain rate and the permanent strain rate at a reference condition in the shakedown state ($\dot{\varepsilon_p}/\dot{\varepsilon}_p^{ref}$). The reference condition is set at the maximum density and optimum moisture content defined by the Modified Proctor test ASTM D-1557 [26]. The shakedown state is defined as the equilibrium state of deformation upon application of a high number of load repetitions [24,27], S_w is the water saturation level in percentage, ρ_d is the dry density of the granular material in kg/m^3, and ρ_w is the density of water at 21 °C, in kg/m^3. This model was developed from repeated load triaxial tests and has been shown to be consistent with other performance models associated with permanent deformation [27].

Drainage coefficients modify the layer structural coefficients (as shown in Equation (1)) and are tailored to the specific drainage conditions of a project concerning the standard AASHO test drainage conditions [5]. This means that the drainage coefficients, described in the AASHTO93 guide, are calculated as a ratio between the structural coefficient of a granular material for a specific condition (a_{site}) and the structural coefficient of the material in the AASHO track under the test conditions (a_{aasho}) [11]. Since the structural coefficient (a_i) is a direct indicator of the structural capacity of the pavement, for the analysis of climatic conditions, the concept described previously will be extended to an analysis of expected performance using Equation (3). For this, it should be considered that a high N_c value implies an increase in the deformation rate (more damage), while a lower level of deformation is expected at low N_c values. Given the above, to estimate the m_i value, an inverse relationship to N_c must be considered as follows:

$$m = \frac{N_{aasho}}{N_{site}} \tag{4}$$

Where N_{aasho} and N_{site} are calculated using Equation (3). The reference value (N_{aasho}) will remain constant in the analysis, as the AASHO test conditions will be considered. However, saturation levels of the granular materials were not measured during the AASHO tests. Therefore, an average saturation value of 60% was adopted for this study based on the equilibrium saturation value reported in pavement bases and subbases [16]. By calculating the coefficient m using Equation (4), considering the conditions of the AASHO test track, and normalizing it to the average compaction and saturation conditions (95% maximum dry density, 60% saturation), a correlation of the variation of the coefficient m_i with the saturation level and the percentage of compaction can be established. This variation, hereafter called the Δm correction, is shown in Figure 4.

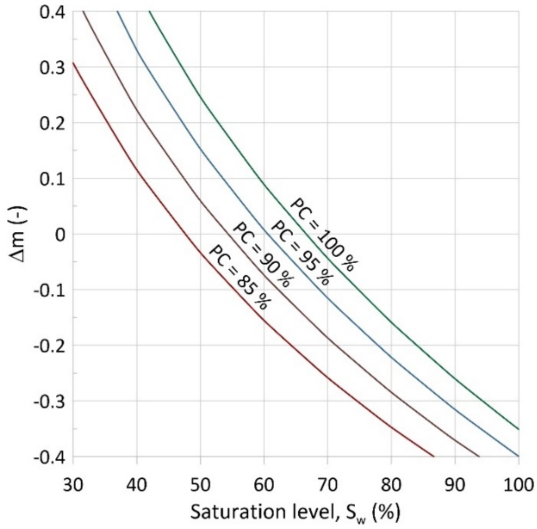

Figure 4. Normalized variation of the drainage coefficient as a function of the degree of saturation and percentage of compaction.

The Δm correction can also be described by the following Equation (5):

$$\Delta m = \log_e\left(\frac{S_w{}^{0.4291}}{S_w{}^{0.0129\times PC}}\right) + 0.0691 \times PC - 3.2964 \tag{5}$$

Where S_w is the degree of saturation in percent, and PC is the percent compaction relative to the Modified Proctor maximum dry density [26] in percent. To calculate a representative value of Δm, a consideration of the different saturation conditions in the region must be made, for which Equation (6) should be used.

$$\Delta m' = \frac{\sum_{i=1}^{n} T_i \times \Delta m_i}{\sum_{i=1}^{n} T_i} \tag{6}$$

Where $\Delta m'$ is the representative correction for the climatic region, n is the number of saturation conditions to consider in the analysis, T_i is the time in months when saturation condition i is expected, and Δm_i is the correction determined with Equation (5) for the saturation condition i.

A reference saturation level can be considered for each season to conduct the analysis. In Venezuela, the expected saturation level is $S_w \approx 40\%$ during dry months, whereas during saturated months, it is $S_w \approx 95\%$. The remaining months can be regarded as having an equilibrium saturation of $S_w \approx 60\%$. To associate the behaviour with a typical year, it is essential to ensure that the total number of months included in the analysis adds up to 12. Table 6 presents the values of $\Delta m'$ for the climatic conditions of Venezuela, considering climatic regions presented in Figure 1 and degree of compaction (DOC) of 95% and 98%.

Table 6. Δm' correction for different climatic zones of Venezuela.

Zone	Season length (months)		$\Delta m'$ @ DOC=95%	$\Delta m'$ @ DOC=98%
	Dry	Saturated		
I	2	6	-0.15	-0.11
II	5	3	-0.02	0.02
III	6	4	-0.04	0.00
IV	4	4	-0.07	-0.02
V	7	3	0.00	0.05
VI	6	3	-0.01	0.04
VII	8	2	0.04	0.09
VIII	9	1	0.09	0.14
IX	8	1	0.07	0.13
X	5	2	0.01	0.06
XI	10	1	0.10	0.15
XII	7	1	0.06	0.11

Note: $DOC = \rho/\rho_{max}$

4. Proposed methodology

The previous sections outline proposed methods for determining the value of m_i based on particle size and gradation (m_0, Table 5), as well as the variation of m_i as a function of density and degree of saturation (Δm_i, Equation (6)). These approaches were derived using rational methods applied to AASHO test conditions. Considering, in addition to the above, the effects of subgrade soil and unforeseen saturation conditions, an optimal m_i coefficient, denoted m_i^*, can be determined using the following Equation:

$$m_i^* = m_0 + \Delta m' + \Delta m_{sg} + \Delta m_{ex} \tag{7}$$

Where Δm_{sg} is the correction based on subgrade type, and Δm_{ex} is a correction to consider external conditions that can affect the pavement saturation regime.

The Δm_{sg} coefficient allows considering the subgrade's influence on the granular layers' saturation condition. As a general criterion, a low permeable subgrade (e.g. clays) will maintain granular materials at high saturation levels longer than less fine subgrades (e.g. sands). Table 7 presents Δm_{sg} values based on the type of subgrade.

On the other hand, consideration for external factors that can change the saturation regime in granular materials, using Δm_{ex} coefficient, allows the inclusion of the uncertainty associated with the expected saturation regime in granular materials. Efficient drainage systems are considered more reliable and economical to ensure good pavement performance. Therefore, it is not recommended to consider adverse conditions that could be mitigated through proper drainage in pavement design. However, incorporating a safety factor for climate change-related events is crucial in risk assessment. In this regard, using the Δm_{ex} factor represents a viable and practical option. Table 8 presents Δm_{ex} values for different situations.

Table 7. Δm_{sg} values as a function of subgrade material

Subgrade (USCS)	Δm_{sg}
GW, GP	0.00
SP	-0.05
GM, GC, SW, SM, SC	-0.12
ML, MH, CL, CH	-0.19

Table 8. Δm_{ex} values for different conditions.

Increase of time in the saturated condition	Δm_{ex}
100%	-0.38
75%	-0.29
50%	-0.19
25%	-0.10

The values suggested for Δm_{sg} and Δm_{ex} have been established through Equation (6), acknowledging that the impacts of subgrade drainage or external factors will extend the period under near-saturated conditions.

4.1 Application example

This section presents an application example to illustrate the expected differences between the approach described in this article and common practice. The following conditions will be evaluated: (i) variations of m_0 for different gradations within the same specification, and (ii) variations in thicknesses when m_i^* is considered in the structural design.

4.1.1 Considering materials within the same granulometric specification

A simple analysis is performed to evaluate the sensitivity of considering different granular materials in the pavement. In this analysis, only the particle size distribution associated with the granular base layer will be modified, ensuring the materials comply with the particle size distribution limits indicated in the Venezuelan specification Fondonorma 2000-1:2009 [28]. In the comparison, only the value of m_0 will be considered, omitting the rest of the parameters used for the calculation of m_i^*. Three materials will be included in the analysis: one near the upper limit of the grain size distribution band, one near the lower limit and one in the middle of the specification. Figure 5 shows the limits established by the specification and the grain size distribution for each of the cases considered. The three established cases present proportions of gravel, sands and fines that place them in Zone 3 of Figure 3. The parameters and m_0 determined for each material are shown in Table 9.

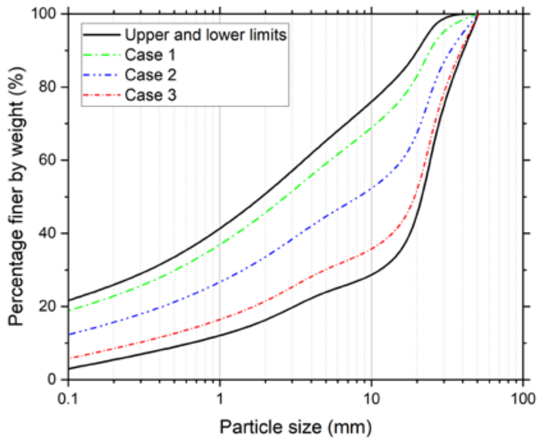

Figure 5. Granular material specification for Base, Type 2, and granulometric curves of case studies.

Table 9. Base materials associated with the same particle size distribution specification.

	Natural gravel, Type 2 [28]		Granular bases		
	Min	Max	Case 1	Case 2	Case 3
$P_{\#40}$ (%)	25	65	59	45	31
$P_{\#200}$ (%)	2	20	17	11	5
G (%)	-	-	41	55	69
S (%)	-	-	42	34	26
F (%)	-	-	17	11	5
m_0	-	-	0.85	0.98	1.13

As shown in Table 9, case 1 has a higher amount of fines, which is related to a lower m_0 value, indicating a possible higher susceptibility of the material to adverse drainage conditions. On the other hand, case 3 exhibits a lower proportion of fines, resulting in a high m_0 value. These values of m_0 are simple parameters that help to consider the expected performance of granular materials under local conditions. The analysis shows that even in materials within the same grain size distribution specification, the susceptibility to saturation and drainage conditions can vary significantly. In the cases studied, the values of m_0 range between 0.85 and 1.13. Failure to consider this material specificity, combined with the influence of climatic zone, soil type and other parameters of interest, can result in significant underestimation of material performance and increased uncertainty, leading to suboptimal, and in many cases very conservative, designs.

4.1.2 Considering different climatic zones

Three Venezuelan cities with different climates will be analyzed: Coro, in Falcón (Climatic zone XI, Figure 1), with a warm semi-arid climate and an average annual precipitation of approximately 380 millimeters. Maracay, located in Aragua (Climatic zone V, Figure 1), has a tropical climate with minimal seasonal variations in temperature and an annual rainfall of approximately 800 millimeters. El Callao, located in Bolívar (Climatic zone I, Figure 1), exhibits an equatorial climate with an average annual rainfall of 1026 millimeters, staying above 50 millimeters even in the less rainy months.

A hypothetical case will be considered in which the soil characteristics, traffic, and road importance will be equivalent, represented by a SN required equal to 5. A granular base material with a CBR of 80% and a subbase with a CBR of 40% will be considered. The analysis will use cases 1 and 3 of the granular materials presented in Table 9. The proportions of gravel, sand, and fines and the corresponding a_i values are presented in Table 10. Table 11 shows the drainage coefficient calculation using the conventional and the proposed method.

Table 10. Properties of materials to be considered in pavement design

Layer	G (%)	S (%)	F (%)	a_i
Asphalt concrete	-	-	-	0.44
Granular base		See Table 9		0.18
Granular subbase	40	45	15	0.17

Table 11. Determination of drainage coefficients to be used in the design phase

Method	Parameter	Layer/Reference	Coro *Climatic zone IX*		Maracay *Climatic zone V*		El Callao *Climatic zone I*	
Conventional	m_i, Table 3	-	0.90		0.80		0.80	
			Case 1	*Case 3*	*Case 1*	*Case 3*	*Case 1*	*Case 3*
Proposed	m_0 (Table 5 & Figure 3)	Base (Zone 3)	0.85	1.12	0.85	1.12	0.85	1.12
		Subbase (Zone 3)	0.89		0.89		0.89	
	$\Delta m'$ (Table 6)	By climatic zone (DOC=98%)	0.13		0.05		-0.11	
	m_i^* (Equation (7))	Granular base	0.98	1.25	0.90	1.17	0.74	1.01
		Granular subbase	1.02		0.94		0.78	

Table 11 shows a consistency between the conventional approach and the proposal of this paper regarding the influence of climate on the strength of granular layers. It is observed that climatic zones with lower rainfall (i.e. climatic zone IX) tend to experience a smaller decrease in the strength of these layers compared to zones with higher rainfall (climatic zones I and V). However, there is a difference in the order of magnitude defined for the coefficients m_i and m_i^*.

In the case of m_i, values lower than 1 can be seen in all cases, which suggests a decrease in the bearing capacity of the corresponding granular layer, which is associated with greater susceptibility to saturation and water balance conditions in the material. This trend is consistent with the values of m_i^* for case 1, where the granular material presents a proportion of fines close to the upper limit of the specifications. In contrast, for case 3, where the proportion of fines is lower, the m_i^* do not penalize the stiffness associated with the granular material (i.e., $m_i^* \geq 1$). This behavior is explained by the values of m_0, where those close to 1 indicate materials relatively similar to those used in the AASHO test. As previously discussed, case 1 is more susceptible to the saturation regime in the analysis of the materials considered. In contrast, case 3 exhibits a more favorable behavior than the reference granular materials.

The parameter $\Delta m'$ allows incorporating the expected stiffness variation in the material based on the probability of exposure to saturation conditions to the saturation conditions to which the materials used in the AASHTO93 method calibration were subjected more accurately. In the cases studied, sections in climate zones IV and V will operate in more favorable conditions (i.e. $\Delta m' > 0$), while climate zone I is presented as less favorable than the reference (i.e. $\Delta m' < 0$). The calibration tests used in the AASHTO93 guide, which are the reference for this study, had an average annual precipitation of 837 mm [8]. This implies that the results obtained for the parameter $\Delta m'$ are consistent with what is expected in the specific climate zones.

Based on the values of m_i and m_i^* shown in Table 11, the thicknesses needed to satisfy $SN = 5$ will be defined. For this case, the values of Δm_{sg} and Δm_{ex} will be considered as 0, assuming the same soil type and probability of change in saturation level during the comparison. Also, to simplify the comparison, the thicknesses of the asphalt concrete layer and the granular base will be considered constant, being 8 cm and 20 cm, respectively. In this way, the influence of the drainage coefficient can be observed in the variation of the thickness of the subbase layer, whose required thickness will be rounded to the nearest centimeter. The thicknesses that satisfy the design conditions are shown in Table 12, Table 13 and Table 14. The SN value is calculated using Equation (1).

Table 12. Pavement thicknesses for SN=5 in Coro, Falcón State, Venezuela

Layer	Conventional		Proposed (Base: case 1)		Proposed (Base: case 3)	
	h_i (cm)	$a_i \times m_i \times h_i$	h_i (cm)	$a_i \times m_i^* \times h_i$	h_i (cm)	$a_i \times m_i^* \times h_i$
Asphalt concrete	8	3.5	8	3.5	8	3.5
Granular base	20	3.2	20	3.4	20	4.4
Granular subbase	38	5.7	32	5.5	27	4.6
Total	66	SN=5.0	60	SN=5.0	55	SN=5.0

Table 13. Pavement thicknesses for SN=5 in Maracay, Aragua State, Venezuela

Layer	Conventional		Proposed (Base: case 1)		Proposed (Base: case 3)	
	h_i (cm)	$a_i \times m_i \times h_i$	h_i (cm)	$a_i \times m_i^* \times h_i$	h_i (cm)	$a_i \times m_i^* \times h_i$
Asphalt concrete	8	3.5	8	3.5	8	3.5
Granular base	20	2.8	20	3.2	20	4.1
Granular subbase	46	6.2	37	5.8	31	4.9
Total	74	SN=5.0	65	SN=5.0	59	SN=5.0

Table 14. Pavement thicknesses for SN=5 in El Callao, Bolivar State, Venezuela

Layer	Conventional		Proposed (Base: case 1)		Proposed (Base: case 3)	
	h_i (cm)	$a_i \times m_i \times h_i$	h_i (cm)	$a_i \times m_i^* \times h_i$	h_i (cm)	$a_i \times m_i^* \times h_i$
Asphalt concrete	8	3.5	8	3.5	8	3.5
Granular base	20	2.8	20	2.6	20	3.5
Granular subbase	46	6.2	49	6.4	41	5.4
Total	74	SN=5.0	77	SN=5.0	69	SN=5.0

The results show a significant influence on the overall pavement thicknesses depending on whether the conventional approach (m_i) or the proposed approach (m_i^*) is used, as illustrated in Figure 6. In most cases, applying the optimized method for determining the drainage coefficient reduces pavement thickness. However, in more adverse conditions, the thicknesses defined by the conventional approach may not be sufficient. These thickness variations are due to more detailed considerations in the proposed approach on how variations in saturation levels affect pavement performance relative to the empirical principles used to formulate the design method. An average variation of 12% in pavement structure thicknesses is expected when employing the optimized method.

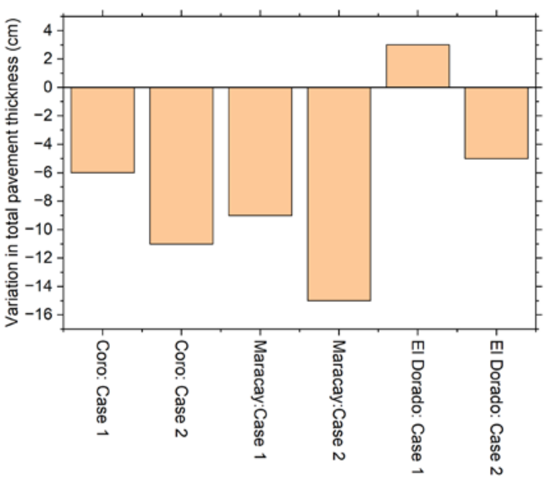

Figure 6. Thickness variation when using m_i^* (proposed approach) to thickness defined using m_i (conventional approach).

5. Conclusions

The AASHTO93 guide remains one of the most widely used pavement design methods in practice. It maintains its validity despite constantly evolving to more modern methods, such as those based on mechanistic-empirical criteria. In this study, the drainage coefficient selection criteria described in the AAHTO93 guide were reevaluated to provide a more rational approach to drainage coefficient selection.

Drainage coefficients (m_i) essentially modify the structural coefficients of the layers and consider the relative effects of the internal drainage of the pavement structure on its performance over the life of the pavement. This paper reviews the conditions and criteria used for the definition of the m_i coefficient and combines it with a rational review and more recent information and analysis methods to improve its determination under local conditions. The proposed approach is based on the development of permanent deformation in granular materials subjected to a high number of loading repetitions.

The study uses modern criteria and experiences to analyze the results reported in the AASHO track, which is the empirical basis on which the AASHTO93 guide is supported. From the above, a reference was produced in the form of a nomogram to associate a baseline drainage coefficient (m_0) from the granulometric characteristics of the material. Similarly, an analytical equation has been used to weigh the influence of different saturation levels on the performance of the materials. From this model, the estimated correction to m_0 could be derived as a function of climatic zone, soil type and the probability that the saturation level changes in the system in an unforeseen manner.

During the study, three materials meeting the same particle size specification were analyzed, which allowed estimating a variation of up to 32% in material susceptibility within the specification, represented by the parameter m_0. This finding indicates that particle size specifications can provide a wide spectrum of behaviors, including material susceptibility to saturation. In addition, it was determined that the proportion of fines is the parameter that presents a direct correlation with the drainage coefficient of granular materials.

Several situations were analyzed to evaluate the applicability of the proposed method. Pavement structures designed for the same requirement (SN=5) were compared, considering different climatic zones and granular materials. A 12% variation in thicknesses was observed between the conventional method (m_i) and the proposed method (m_i^*). The ability of the proposed method to define the effect of climatic zones more accurately, based on the weighting of dry and saturated months, as well as its ability to incorporate the granulometry of the material in the analysis, is considered to allow a better definition of the drainage coefficient, which is more representative of local conditions.

This improved definition of the drainage coefficient makes it easier to optimize the consideration of the performance of granular materials as a function of seasonal changes in saturation over the life of the pavement. As a result, an optimization of the designed pavement structure is achieved. This optimization can be translated into increased thicknesses under unfavorable conditions (i.e. susceptible materials, high probability of saturation) and decreased thicknesses under favorable conditions for the pavement (i.e. low susceptible materials, low probability of saturation).

This article introduced a parameter to address uncertainties related to possible changes in future climatic conditions (Δm_{ex}). However, this parameter is based on probable increases in saturation times, which is difficult to predict or consider during the design phase. Therefore, this parameter requires further investigation based on local experience and documentation.

Considering the seasonal effects of climate on the performance of materials is common in mechanistic-empirical design methods. It is recognized that this practice allows pavement design to be optimized by better representing local environmental conditions. The method developed in this paper seeks to convey the same philosophy of more local consideration when applying the AASHTO93 design guide. The method proposed in this article was developed considering the conditions of Venezuela, taking advantage of the information already established in this country; however, the resources offered here can be adapted to the characteristics of other countries. Notably, the described approach can be of great utility for countries with tropical and subtropical climates, where the climatic seasons are similar to those considered in this article and, at the same time, are the regions where more significant differences can be expected with the materials and climates used as reference in the development of the AASHTO93 method.

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