French civil aviation authority

Civil aviation technical center

October 2016

Rational design method for flexible airfield pavements

Technical guide

Ministry of Environment, Energy and the Sea

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Civil aviation technical center
Pavement Structures and Friction

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Summary

This technical guidance sets out a french design methodology for new flexible airport pavements. It is based on rational principles as well as material performances and it implements the last findings in pavement engineering. The methodology has been established from a theoretical model – multi-layer linear elastic isotropic – the inputs of which are mechanical behavior parameters. These ones are determined from laboratory tests on pavement materials. The validation of pavement structures is based on a mechanical design which results in the determination of pavement layers thickness and nature. This computation is then completed by frost-thaw verification.

This document constitutes a technical frame of reference which enables comprehending pavement design studies at best and with a unique approach. This is why elements relative to pavement foundation, surface course and pavement materials, are tackled complementarily to the design procedure itself.

The method has a vocation to be applied not only to the French airfields, but also abroad. Local specificities (climate, geotechnics, infrastructure management...) may be implemented by designers through various parameters.

Keywords

airfield pavement, pavement structure, design, asphalt materials, unbound materials, damage law, fatigue, permanent deformations, frost-thaw.

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1 Introduction

1.1 Background and purpose

The method traditionally used in France to design airfield pavements is an empirical method, inspired by the American method developed by the United States Army Corps of Engineers, which is based on the CBR (California Bearing Ratio) method for flexible pavements, and the PCA (Portland Cement Association) for rigid pavements.

The CBR method supposes that the pavement perishes due to excessive pressure on the subgrade. The strength of the subgrade is qualified by the CBR penetration test. Therefore, it determines the design of the pavement structures that is acceptable in view of this risk.

The PCA method supposes that the concrete slabs rest on an elastic bed of a given stiffness. The thickness of the slab is defined so that the maximum tensile stress of the concrete cannot be reached.

These empirical methods were developed on the basis of tests that allowed the effects of the numerous factors in play to be taken into consideration by interpolation: the thickness of the layers of materials, the geometry and load of the landing gear, the climatic conditions, etc.

But today, these methods are reaching their limits, in view of both changing aircraft, which have new landing gear configurations and apply new loads, and of the changes in available materials, offering higher levels of performance.

Faced with these changes, it has become necessary to review the French method used to design airfield pavements, by adopting a « rational » approach that is better suited to taking actual and new situations into consideration, and can be adapted to existing pavements overlay. This improvement will be achieved by adopting a more explicit method, based on a linear elastic method that takes account of the performances of the materials and of the silhouette of the landing gear of the aircrafts.

This type of method has been in use in France more than 30 years to design the structures of highway pavements. It is explained in the SETRA – LCPC technical guide to the « French design manual for pavement structures » [1], published in December 1994 (1997 for the English version), and is covered by the standard NF P 98-086 [2]. The two main aspects of this method are :

- the materials making up the pavements have a linear elastic behavior. The materials are characterized by their modulus of elasticity and their Poisson's ratio ;
- the permissible strains and stresses of the different materials is assessed on the basis of models of damage under repeated loads, the parameters of which result from laboratory tests, full-scale tests on experimental instrumented pavements and feedback from actual pavements in operational service.

The decision was taken to develop the rational design method of airfield pavements on the basis of the method for roads, in order to make it more adaptable to other situations, and in particular to different weather conditions, so that the method can be exported.

The goal is, ultimately, to devise a rational design method that can be used for all types of pavements (flexible, rigid and others) and in all types of design contexts (new pavements, renovations and overlays). In future, the adoption of this method by the International Civil Aviation Organization (ICAO) and its inclusion in the ACN/PCN (Aircraft Classification Number/Pavement Classification Number) classification should come under scrutiny.

1.2. Scope

Initially, the scope of the initiative was limited to new flexible pavements, before being extended to include the cases mentioned above, which demand a more complex approach. This guide marks the completion of this first stage.

Figure 1 : view of San Francisco airport from the air

The method proposed in this document applies to runways, taxiways and aprons. However, regarding the latter, note that the use of a bituminous structure is definitely not recommended when there is a risk of serious punching, associated with stress levels NS3 or NS4 according to the « Guide to the application of standards (GAN), Bituminous mixtures and surface dressings for airport pavements » [3]. In this case, a rigid structure is recommended, the design of which is not covered by this guide. Moreover, this document does not cover the design for light aircraft, shoulders, RESAs (Runway End Safety Area) and stopways. These structures can be designed and defined according to the rules defined in the French document « Pavement design, volume 1 », 1983 [4], the TAC decree [5] and its annex, and the guide to « Cement concrete airfield pavements », 2000 [6]. Regarding shoulders, the « Study of the bearing capacity of shoulders for wide body aircraft » [7] can also be consulted.

By definition, a flexible airfield pavement is made of bituminous materials and comprises, from the top downwards:

- a wearing course made of bituminous materials,
- base course made of bituminous materials,
- a sub-base made of an untreated graded aggregate mix (GNT)

In addition to the three layers above, a binder course may also be inserted between the wearing course and the base course. The wearing course and the binder course form the surfacing. Also note that the base course – sub-base assembly forms the base layer. All the layers above the foundation make up the pavement structure.

Figure 2 below shows a complete illustration of a pavement structure.

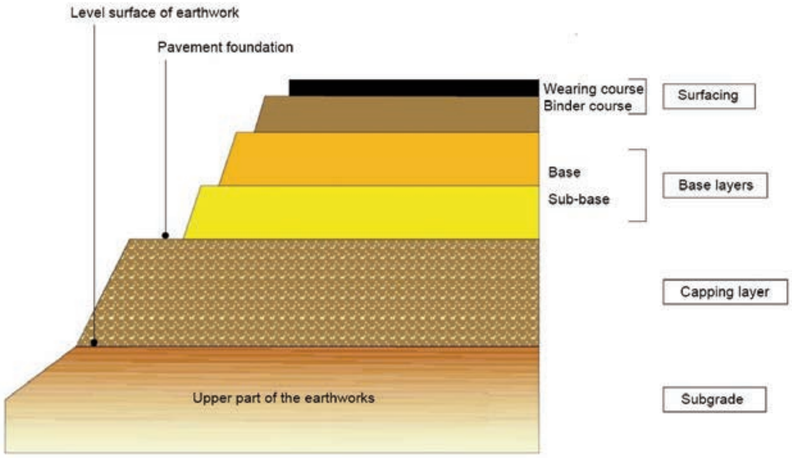

Figure 2: diagrammatic representation of a pavement structure

This structure rests either directly on the subgrade, which may be covered with a grading course, or on a capping layer, which may be treated. The use of a treated capping layer does not allow the structure shown above to be ignored, in particular with regard to the use of a sub-base. This point is developed in paragraph 4.5.

If no capping layer is included, then the foundation and the level surface of earthwork are one and the same.

If the design traffic mix is low and only moderately aggressive with regard to the pavement in question, the base course can be made of granular material, in which case the pavement structure only includes a single layer of bituminous materials (or two, if there is a binder course). This alternative is possible when the aggressiveness of traffic, as determined by a parameter called « Equivalent Single Wheel Load » (ESWL), explained in detail in 2.6, meets the following two conditions:

✓ the traffic mix is made up of **group 1 and 2** aircraft only (as per the GAN [3]),

✓ **ESWL < 10 t** for all the traffic (as calculated for a structure with an asphalt base course).

Design method refers to the complete process that is used to design an airfield pavement for a given service life and expected traffic. The process includes the collection of the basic data, the choice of the materials making up the pavement, modeling its behavior and verifying its resistance to mechanical and climatic strains throughout its service life.

This guide explains the steps of the process to design new flexible pavements that leads to the determination of the thicknesses of the different layers of the pavement structure, according to the traffic and the environmental conditions. It explains how the calculation parameters are linked to the physical and mechanical properties of the materials (standardized values in the basic solutions, or laboratory values in the variants). It also contains numerous considerations about the choice of materials, which must comply with the applicable regulations, and about the definition, characterization and improvement of pavement foundations.

Lastly, note that this guide attempts to make the strengths and the weaknesses of the method quite clear. In particular, there is much less data and feedback that is necessary to calibrate the damage laws of the materials, in the aeronautical sector than in the realm of roads. Therefore, the calibration parameters introduced in this guide may be subject to change in future versions in order to incorporate the findings resulting from their application.

1.3. Study phases applicable to the design of aeronautical infrastructures

Studies of aeronautical infrastructures always involve the production of two successive technical documents, for all operations apart from maintenance works:

- the pre-project file, which assesses the feasibility of the operation by specifying its general design, the main requirements of the constructions and the financial cost. The pre-project file must highlight any technical difficulties that will be encountered in the construction phase,
- the project file, which is compiled after the pre-project file has been approved. The purpose of this file is to add more details to the pre-project study, in particular by proposing technical solutions to the difficulties that were raised, without being investigated. The project file contains all the information required for the complete definition of the constructions involved in the operation.

Figure 3: example of a building site

Both of these studies require the design calculations of the pavement structures. Whether they take place in the pre-project or the project phase, these calculations demand the same type of input data. They cover not only the topographical, climatological, hydrogeological and geotechnical aspects, but also the period of calculation and the design risk used, the traffic and the properties of the materials making up the pavements.

The degree of precision required of all this data depends on the level of the study.

The hydrogeological and geotechnical data must come from specific analyses conducted by specialized engineers. Ground investigations must be conducted in several phases, which can be defined according to the standard NF P 94-500 [8], depending on the questions raised in the scheduling, design and construction phases of the project in question. The distinction is made between:

- ✓ preliminary investigations for the position and pre-design of the works;
- ✓ investigations for design purposes.

The geotechnical investigation techniques associated with the phases of the project are covered by « Eurocode 7 - Part 2 Ground investigation and testing » (EN 1997-2) [9]. The memo « Organization of geotechnical reconnaissance on road and motorway routes », 1981 [10] may also be referred to.

For pre-project studies, the earthworks section of the geotechnical report must refer to the LCPC-SETRA guide to « The production of backfill and capping layers", 1992, (hereafter referred to as the "Guide to road earthworks » or « GTR »), second edition, 2000 [11], and, if necessary, to the LCPC-SETRA technical guide to « The treatment of soils with lime and/or hydraulic binders », January 2000 (GTS) [12]. It must provide, at least, the characterization of the materials in place, as per NF P 11-300 [13], the possibility of reusing the soil and the class of the target foundations under the pavement. The document must also cover the possible need to use soil treatments in order to improve the class of the level surface of earthwork and of the pavement foundation, without precisely defining the treatment to be used. This report is compiled using bibliographical data and a limited number of in situ investigations. It introduces an initial approach to the geotechnical context and may elaborate hypotheses that the subsequent studies will attempt to verify.

In the geotechnical study in the project file, the number and the choice of the investigations (surveys, laboratory tests, etc.) will be defined according to the construction options in the pre-project. The geotechnical study must specify the data and ratify the conclusions contained in the pre-project study. It must define the composition of the treatment that may be necessary to improve the bearing capacity of the pavement foundation, with reference to the GTS [12].

These studies must result in the definition of the nature and the minimum thickness of the capping layer, in view of the minimum deformability of the pavement foundation demanded by the project manager, in the short term (in order to specify the contractual acceptance criteria) and in the long term (for specific design needs), either for the level surface of earthwork or for the capping layer, treated or untreated.

The approach can be approximate in pre-project studies, but it must be reliable and precise in the project studies. It must also define the drainage requirements associated with the design of the foundations and level surface of earthwork, as elaborated in the technical guides « Design of aerodrome drainage networks », STBA (2000) [14], « Road drainage », LCPC-SETRA (2006) [15], completed by the SETRA information memo N°120 « The contribution of drainage to the design of pavement foundations » [16].

1.4. Organization of the document

The remainder of this document is made up of seven chapters, a glossary and fives appendices:

Chapter 2: the principle of the design approach

This chapter starts with the classification of the various aeronautical areas to which the design method applies. It then describes the design approach and introduces the damage laws and the notion of cumulative damage.

Chapter 3: application of the design approach

This chapter describes the data required to design the pavement structure and gives details of the rational calculation method.

Chapter 4: the pavement foundation

This chapter looks at the characterization of the foundations by analysis of the subgrade and of a possible capping layer, on which the pavement rests.

Chapter 5: surfacing

This chapter goes over the main elements inherent to surface layers elaborated in the GAN [3], with a brief presentation of the approach to be adopted in order to guarantee that the surface layers can withstand the different attacks to which they are exposed.

Chapter 6: pavement materials

This chapter introduces all the texts and technical provisions existing in France, in which the various materials and products in the pavement courses are codified and described.

The following information is provided for the materials covered:

- ✓ a description of the physical and mechanical characteristics of the different reference materials classes, for which the road pavement structure design method has been calibrated,
- ✓ and the manner in which the values of the calculation parameters are deduced from the results of the mechanical tests.

Chapter 7: freeze-thaw verification

This chapter lists the principles of the pavement freeze-thaw verification, which is a step of the design process in its own right.

Chapitre 8: examples of application

The rational design method of new flexible airfield pavements is approached from a practical perspective, with a few examples of application. The first example in particular gives a detailed illustration of the different calculation steps.

Glossary

The glossary at the end of the document contains the terms and definitions specific to aeronautics.

Appendices

The appendices look at the calculation mode of certain parameters (traffic distribution at a given point, the design risk, the equivalent temperature for all bituminous materials), of the design rules of the capping layers and their acceptance criteria.

This guide is intended to be self-standing. On occasions, the authors have chosen to include complete passages from the reference documents. In the chapters covering the surface layer and the pavement materials, excerpts from the GAN [3] have been inserted, whenever necessary in order to understand the text. On other occasions, this document is simply mentioned as reference, in order to simplify the writing of this guide. **Nevertheless, knowledge of the GAN [3] and its use in conjunction with this guide are necessary for the proper application of the design method.**

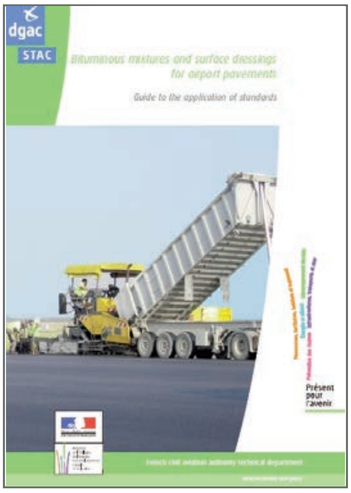

Figure 4: bituminous mixtures and surface dressings for airport pavements – Guide to the application of standards (GAN) [3].

2. The principle of the design approach

2.1. Purpose

The design process consists of selecting the materials, determining the thicknesses of the layers and checking the behavior of the structure when subjected to freezing and thawing. For the flexible pavements covered in this guide, the rational approach to the design of pavements applies to the structural design of the base layers, in relation to the mechanism of damage by fatigue of the layers made of asphalt materials, and of damage by permanent deformation of the layers made of granular materials, including the pavement foundation. On the other hand, the design method does not directly take into consideration the rutting mechanism due to the creep of asphalt materials. The problem is addressed by choosing suitable materials on the basis of laboratory tests.

The choice of the surface layer (constituent materials, thickness) is addressed in chapter 5 of this guide, and remains empirical.

2.2. Classification of aeronautical infrastructures for design purposes

The definitions of the terms describing airfield pavements are given in the glossary (p109).

In terms of design, the three following families of sections are introduced and taken into consideration:

- High-speed sections,
- Moderate-speed sections,
- Aprons and low-speed sections.

As shown in Table 1 and Figure 5, each of the commonly defined zones of an aerodrome can be included in one of these families. Each family is linked to the choice of the numerical values of two parameters (speed and lateral wander amplitude), as defined in paragraph 3.1.3.

High-speed sections	*Regular part of the runway*
Moderate-speed sections	*Aircraft stand taxilane used in taxiing mode* *Connecting taxiway* *Runway entrance/exit* *First 600 meters of the runway, including the threshold*
Aprons and low-speed sections	*Aircraft stand taxilane used in push-back mode* *Aprons* *Runway turn pads* *Waiting areas (or platforms)*

Table 1: map between the three families of sections taken into consideration for design purposes and the various usual airfield infrastructures

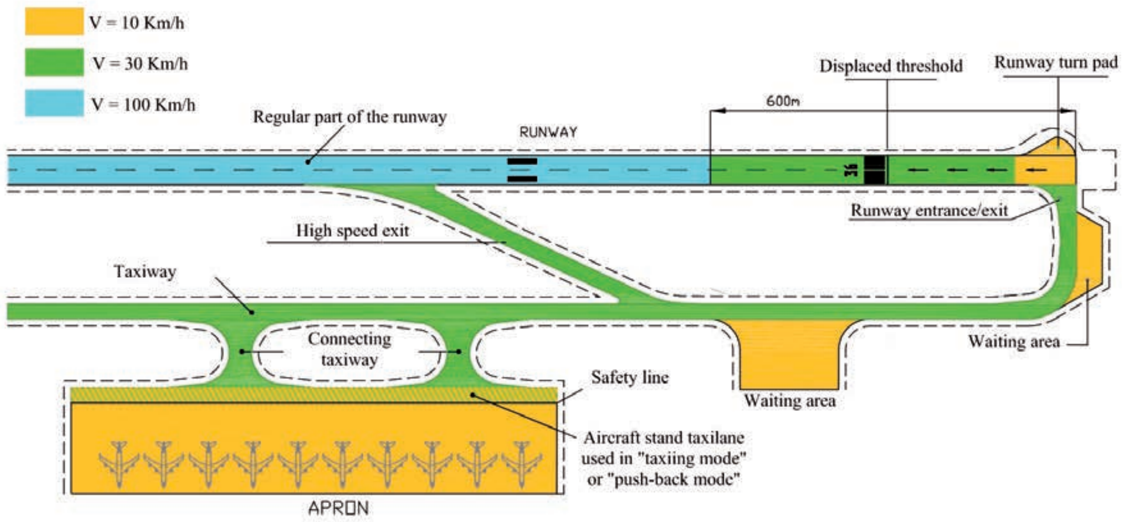

Figure 5: map between the three families of sections taken into consideration for design purposes and the various usual airfield infrastructures

2.3. Rational design method of airfield pavements

The rational method for the design of airfield pavements described in this guide is based on the design method of roads and motorways, described in the technical guide « French design manual for pavement structures » (SETRA-LCPC) [1] and in the standard NF P 98-086 [2]. The transposition to airfield pavements required a number of adaptations, which are described in this section and the next section of this guide, in terms of both the description of the data required for the calculations and the execution of the calculations and the processing of the results.

The method is broken down into two steps. The first step covers the mechanical design of the structure, while the second covers the freeze-thaw verification.

The mechanical design consists of verifying that the chosen structure can withstand the expected traffic of aircraft for the previously defined design period.

Note that the expression « design period » is deemed to be preferable to « life time », which is literally meaningless for a pavement. On the one hand, the deterioration of a pavement section is always uneven, and, on the other, for safety and economic reasons, maintenance works are always completed before the pavement structure is completely destroyed. Therefore, it is not really possible to talk about the « end of life » or the « life time » of a pavement.

The mechanical design is based on the calculation of:

- the strains generated in the structure by the aircraft, using the multi-layer, isotropic linear elastic model,
- the individual damage caused, in each material, by the passage of each aircraft. The damage law is calibrated on the basis of laboratory fatigue tests for bituminous materials (amongst others), or empirically for untreated granular materials and platforms. Damage in the bituminous layers is caused by fatigue, while the damage in the soils and untreated granular materials takes the form of permanent deformation,
- total damage, obtained by adding up the individual damage for the entire design period, in view of the traffic taken into consideration and the lateral wander probability laws[1]. In this case, the damage to the pavement is defined as the maximum value of the total calculated damage for each of the layers.

[1] Densities of probability relative to the lateral position of the aircraft on the various sections.

The determination of the characteristics to be taken into consideration in the design process, and in particular the elasticity parameters of the materials is described in the following chapters.

The method uses adjustment coefficients that are applied to the expression of the damage laws. These coefficients include a calibration coefficient called "shift factor", derived from the observation of the behavior of experimental pavements and from feedback. This parameter can be used to incorporate the effects that the model is unable to represent, due to the simplifications that are made, and the bias linked to the description of the properties of the materials and their use, into the results of the theoretical calculations.

The application of the design approach starts with the collection of the project data (aircraft traffic, design period, etc.) and the choice of the design parameters, in particular the bearing capacity characteristics of the foundation, the nature of the materials to be used in the project and their level of performance. In basic solutions, the materials must be selected from those that are covered by the product standards, i.e., the EN 13-108 [17] series of standards for bituminous materials, and EN 13-285 [18] for untreated graded aggregates. The next step of the approach consists of applying the previously described method (calculation of strain and the resulting damage), in an iterative manner, to the thicknesses of the different layers (or certain layers), in order to obtain a total damage to the pavement that is below, but as close to 1, as possible.

The thicknesses must be adjusted in order to:

- allow for the technological constraint of the minimum and maximum thicknesses of the layers in order to achieve the degree of compaction and roughness targets, which depend on the nature of the materials,
- limit the number of interfaces and, therefore, reduce the risk of bonding faults between them,
- take account of other project criteria (technical, economic, environmental, etc.).

2.4. Calculation of the strains produced by aircraft in the pavement structure

The strains produced in the pavement structure by the aircraft loads are calculated using a semi-infinite, multi-layer, isotropic linear elastic model, as shown in Figure 6.

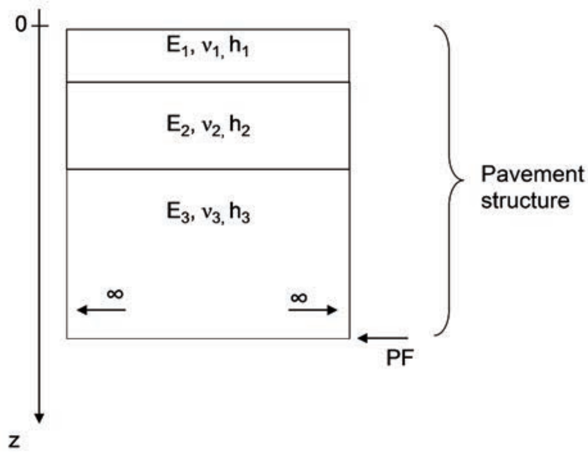

Figure 6: model of the pavement structure

This model describes the structure of the pavement as a succession of layers, of a finite and homogeneous thickness. The subgrade is represented by a layer of an infinite depth, except in particular geological situations.

If there is a rigid substratum, located at a depth h_{sub} (counted from the level surface of earthwork) between 2 m and 6 m, the supporting bed is divided into two layers. One is of a finite thickness h_{sub}, and the other, which represents the substratum, is of an infinite thickness. In this case, the substratum is defined as a horizon that can be considered as non-deformable, relative to the rigidity of the soil that it supports, and is therefore associated with a very high modulus of elasticity of about 10,000 MPa.

Specific investigations are necessary when the substratum is at a depth of 2 m or less, because the semi-infinite character of the calculation model no longer applies in this case.

The different cases are shown in Figure 7 below, with the corresponding models.

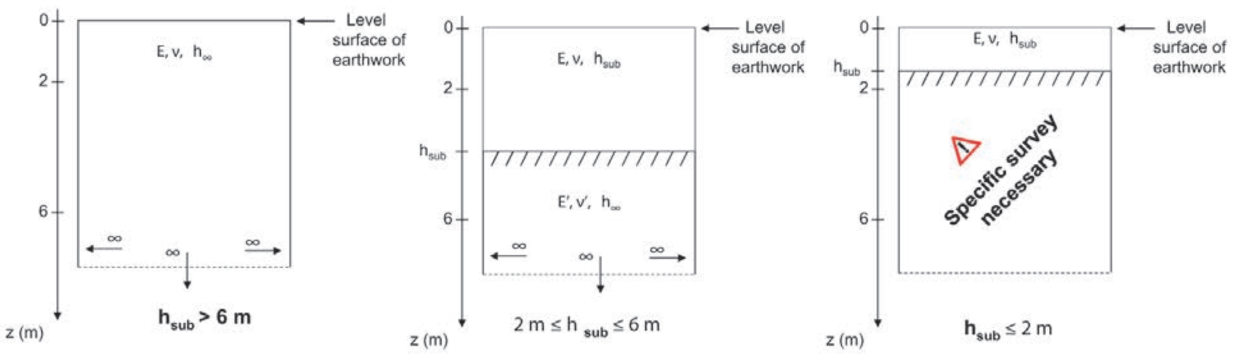

Figure 7: model of the subgrade according to the depth of the rigid substratum

For each layer, the material is assumed to be isotropic, linear elastic, with infinite extension in the horizontal plane, characterized by a Young's modulus E and a Poisson's ratio v. The interfaces between the layers are considered to be bonded when designing the structures of new flexible pavements. The calculation is made under static load conditions.

The traffic is usually described by the different types of aircraft expected to use the airfield pavement and by the frequency at which they use the pavement, while making the distinction between take-offs and landings. The aircraft are described by the geometry of their landing gear. By default, the footprint on the ground of each wheel is assimilated with a circular disk. The footprint radii, loads and pressures in the Civil Aviation Technical Center's « Ficav » aircraft database shall be taken into consideration.

The strains produced by each of the aircraft are calculated in horizontal planes (x, y), where is the longitudinal axis of the rolling load (assumed to be the same as the axis of the pavement) and is the transversal direction. The planes (x, y) are located at the base of the bonded layers (for the valuation of the horizontal extensions applied to the damage criterion of bituminous materials) and at the summit of the non-bonded layers (for the valuation of the vertical contractions of the permanent deformation criteria of these materials), where the strain values are highest. For each of these planes at the point z_k, the strains are calculated using a grid of points (x_i, y_j, z_k), separated by a constant Δx in x and Δy (which may be equal to Δx) in y (figure 8). $\Delta x, \Delta y$ are set to values less than or equal to 5 cm, while the dimensions of the grid must be defined such that the strains caused by the aircraft at its contour is negligible (< 5% of the maximum strain). The value $z_k = 0$ is usually used on the surface of the pavement.

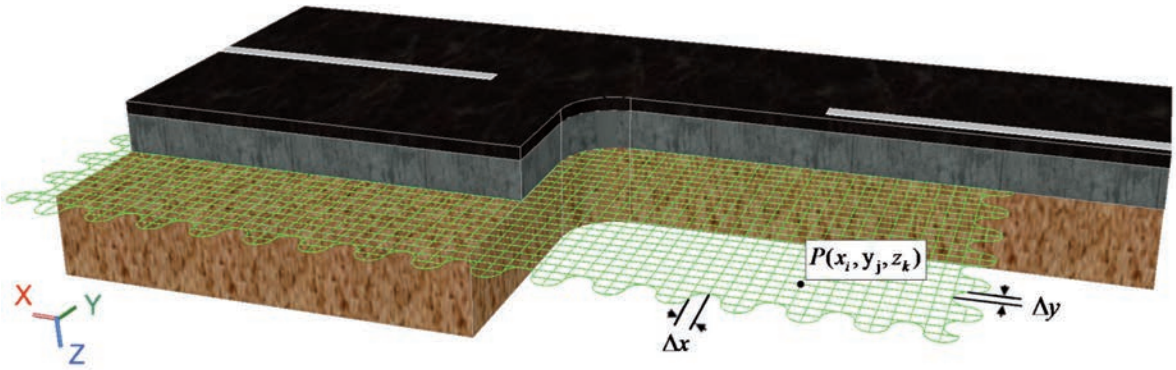

Figure 8: representation of a calculation grid at the interface between the base course and the sub-base, and of a calculation point $P(x_i, y_j, z_k)$.

These calculations must respect the following hypotheses:

- a description of the structure in the form of a stack of layers that are infinitely horizontally extensible, and with a thickness that is infinite for the soil and finite for the others,,
- bonded interfaces between the layers, i.e., that guarantee the continuity of displacements in all three axes (this hypothesis is applied to new flexible pavements),
- an isotropic linear behavior law for each of the constituent materials,
- a description of the loads by circular surfaces, subjected to uniform vertical pressures.

By applying Burmister's law, these calculations can be made using semi-analytical calculation codes that explicitly take the infinite horizontal and vertical extensions of the multi-layer calculation model into consideration.

These calculations can also be made using finite element codes (axisymmetric 2[2] or 3D) on the basis of finite extension meshes that are sufficiently large to be precise enough in comparison with the theoretical Burmister solution. The relative accuracy to be achieved is 5 thousandths of the maximum value of the strains used in the design process. This accuracy must be achieved at all points of a rectangle that exceeds the geometry of the landing gear by a distance at least three times the standard deviation S_{bal} of the lateral wander (Figure 9). Using Burmister's model, the examples in Chapter 8 can be used a benchmarks to verify the relative accuracy of the code.

The next step of the method uses the calculations of the damage caused by aircraft on the basis of the history of the strains caused at all coordinates (y_j, z_k) of the pavement when an aircraft passes over it. By ignoring the notion of physical time (which plays no explicit role in the damage laws considered hereafter), these histories can be reproduced using discrete and successive values of strain at the points (x_i, y_j, z_k), with an increasing i.

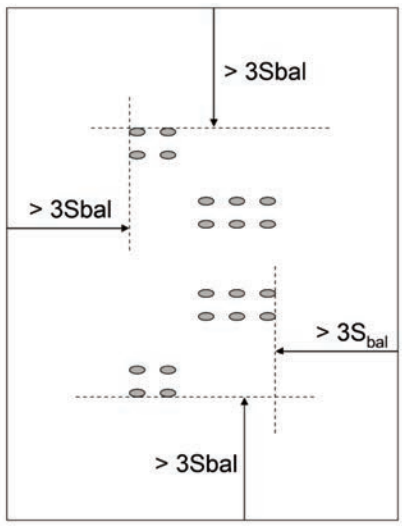

Figure 9: relative accuracy in relation to the Burmister model – boundary of the domain of comparison

[2] ***The linearity of the problem allows the calculations to be limited to finite elements for isolated loads and to deduce the response of the pavement when subjected to multiple loads using the superposition theorem. In this case, the axisymmetrical calculations can be made precisely and quickly using high-extension 2D meshes (vertical section) and a high number of elements.***

2.5. Damage laws and calculations

The damage calculations are based on the assessment of the strains within the pavement structure, as described in the preceding paragraph, and on the damage laws, which come down to the same Wöhler-Miner type formalism for all materials.

Two types of damage are considered (Figure 10):

- by fatigue, for bituminous materials, resulting in the gradual cracking of the material. In this case, the calculation uses the values of horizontal strains calculated at the base of the lower layer of bituminous materials,
- by permanent deformation of elasto-plastic origin, for non-bonded granular materials. The calculation uses the values of vertical strains calculated at the summit of the foundation. In practice, no calculations are made at the summit of the sub-base.

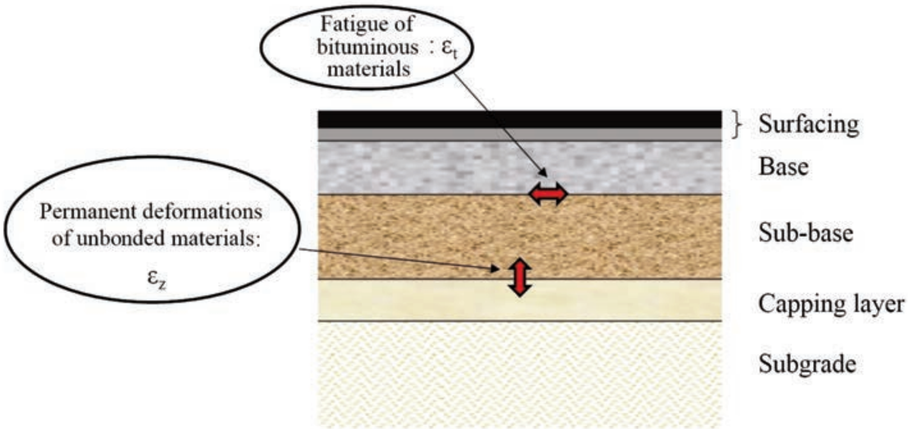

Figure 10: the two modes of damage considered for flexible airfield pavement structures

If the traffic is light and allows for the use of a base course made of granular materials (see Chapter 1.2), feedback has shown that the mode of failure in play is the permanent deformation of the soil. In this case, fatigue damage to the bituminous materials is no longer taken into consideration.

Note: *conventionally, strains are considered by the positive extension in the asphalt concrete and positive contraction in granular materials.*

2.5.1. Wöhler's law

Initially, Wöhler's law applies to laboratory fatigue tests performed on material specimen that are cyclically stressed at a constant amplitude of force or displacement.

Depending on the case in hand, it is expressed as a function of the theoretical maximum amplitude of stress (imposed force test) or strain (imposed displacement test) that is reached on the entire body of the specimen, which is assumed to be homogeneous and in its initial intact state.

By extension, and allowing for a few adjustments (see paragraph 2.7), the use of Wöhler's law is transposed to pavement structures for this design method by calculating the mechanical fields in the multi-layer model (reversible strains, stress) induced by the applied loads. This generalization can be used to demonstrate the fatigue mechanisms of the bituminous materials and the permanent deformations in the unbonded granular materials.

By noting in a general manner, s_{max} the maximum amplitude of strain or stress[3], Wöhler's law, relates the number of cycles N (s_{max}) resulting in failure[4] under these conditions in the following form:

$$N(s_{\max}) = \left(\frac{K}{s_{\max}}\right)^{\beta} \text{ or } s_{\max} = K\,N^{b} \qquad (b = -1/\beta)$$

where:

K constant

β positive exponent greater than 1

$b = -1/\beta$ negative scalar greater than -1 ($-1 < b < 0$) equal to the gradient of the fatigue straight line which equation is log (s_{max}) = $a + b\ log\ (N)$.

- For bituminous materials:

N comes from a reduction of the rigidity of the stressed material at a temperature of 10°C and at a frequency of 25 Hz. It is defined as the average value of the number of cycles resulting in failure, obtained by definition when the loss of stiffness of the material reaches 50 % (standard NF EN 12-697-24 [19]). In this case, the variable s_{max} corresponds to the maximum amplitude $\varepsilon_{t\ max}$ of the extension strain, calculated at the base of the structural course[5].

$$N(\varepsilon_{t\max}) = \left(\frac{K}{\varepsilon_{t\max}}\right)^{\beta} \text{and } K = k_{\theta f} k_r k_s k_c 10^{6/\beta}\,\overline{\varepsilon}_6$$

where:

$\overline{\varepsilon_6}$ the value of the strain ($\mu strain$) after 10^6 cycles, experimentally determined in a laboratory at 10°C and 25 Hz,

$k_{\theta f}$, k_r, k_s, k_c adjustments coefficients, whose meanings are explained in detail below.

***Note:** joint studies by the IFSTTAR and the Laboratoire Régional de Bordeaux (LRPC) have shown that the fatigue line obtained for about 10^5 to 10^6 repeated load cycles (the number of cycles used for roads) can be extrapolated to the aeronautical sector, in which order of scale of cycles is about 104. Therefore, the parameters taken from the fatigue tests remain valid.*

Chapter 6 contains the values of ε_6, the exponents β and the parameters required to calculate the adjustment coefficients for each of the families of bituminous materials. Depending on the context in which the method is applied, K will be taken from the product standards (e.g., analysis of basic solutions) or may be taken from laboratory measurements (e.g., analysis of variants).

- For unbonded granular materials:

Wöhler's law is used to describe the damage linked to cumulative permanent deformations due to the effect of repeated contraction strains.

Unlike the case of bituminous materials, the law used hereafter does not initially come from laboratory tests, but is taken directly from numerous observations and measurements taken on road pavements or, to a lesser extent, on airfield pavements. The number of cycles at « failure » is highly empirical in character and depends on the repercussions on the surface of the deep deformations, but it is not really possible to apply a quantified criterion.

[3] ***maximum value to be considered both in relation to the space variables and in relation to the time variable.***
[4] ***according to a predefined criterion that usually applies to the global rigidity of the test specimen.***
[5] ***This strain is the highest of the specific values of the 2x2 sub-matrix*** $\begin{bmatrix}\varepsilon_{xx} & \varepsilon_{xy}\\ \varepsilon_{xy} & \varepsilon_{yy}\end{bmatrix}$ ***extracted from the complete 3x3 strain matrix.***

In this case, the variable s_{max} equals the maximum amplitude (in space and time) $\varepsilon_{zz\ max}$ of the reversible vertical strain at the summit of the foundation. In this case, Wöhler's law becomes:

$$N(\varepsilon_{zz\max}) = \left(\frac{K}{\varepsilon_{zz\max}}\right)^{\beta}$$

The values of K and β are considered to be independent of the type of material and its water content, of the temperature and of the speed of loading.

The values of the K and β coefficients for the foundation can be found in Chapter 4, paragraph 4.6.1.2.

2.5.2. Elementary damage and the Wöhler-Miner law

The Wöhler-Miner law applies Wöhler's law to cases of high numbers of loadings, of varying intensity that occur in a random manner. It is based on the notion of elementary damage, the postulate of the additive effect of this damage and the failure criterion for cumulative damage that equals 1.

Wöhler's law, as exposed here above, can be used to define the elementary damage ΔD caused by a loading cycle of an amplitude of ε_{max} as:

$$\Delta D = \frac{1}{N(\varepsilon_{\max})} = \left(\frac{\varepsilon_{\max}}{K}\right)^{\beta}$$

where $\varepsilon_{max} = \varepsilon_{t\ max}$ for bituminous materials or $\varepsilon_{max} = \varepsilon_{zz\ max}$ pfor granular materials.

If the loadings are of varying amplitudes and occur in a random manner, the Wöhler-Miner law then stipulates:

- i) the additive effect of the elementary damage caused by each one of them, i.e.:

$$D = \sum_i n_i \Delta D_i = \sum_i \frac{n_i}{N(\varepsilon_{\max i})} = \sum_i n_i \left(\frac{\varepsilon_{\max i}}{K}\right)^{\beta}$$

where:

n_i = the number of loads of the amplitude $\varepsilon_{max\ i}$

D = damage caused after the application of the $N = \sum n_i$ loading cycles

- ii) the point at which the material "failure" occurs, according to the criterion referring to Wöhler's law, when $D = 1$.

It is immediately verified that, in the particular case of load cycles that are all identical, Wöhler's law and the Wöhler-Miner law both produce the same number of cycles at « failure ».

In addition to forecasting the failure, it should be noted that the damage value *D* can also be used in certain applications (e.g., expert investigations, calibration of material coefficients, etc.) as a continuous theoretical indicator of the state of health of a material or a structure, according to the load history, that can be compared with in situ observations.

2.5.3. Continuous integration of the Wöhler-Miner law and calculation of damage caused by passing aircraft

However, the preceding expression of damage is not totally suited to the integration of groups of rolling loads (bogies) that produce complex histories of strains in the pavement (often with multiple peaks, with no return to zero between the peaks), to which the notion of loading cycles cannot really be applied.

The application of the Wöhler-Miner continuous law (see annex A) in the design method, in the form of an integral law, allows the formalism to be extended accordingly and the increase in damage due to a given aircraft to be calculated. For the strain $\mathcal{E}$, associated with the envisaged damage mechanism by the Wöhler law, the expression is written as follows:

$$\Delta D(y,z_k) = \frac{\beta}{K^{\beta}}\int_{-\infty}^{+\infty} < \varepsilon(x,y,z_k) >^{\beta-1} < \frac{d\varepsilon}{dx}(x,y,z_k) > dx$$

where $\mathcal{E}\,(x,\ y,\ z_k)$ is the longitudinal profile of the variable ε, is the longitudinal profile of the variable (y, z_k), and <X> is the positive part of the variable X.

As we have already mentioned, this profile represents the history of the strain $\mathcal{E}$, parameterized in x, at all points$(x,\ y,\ z_k)$ of the pavement, when the load travels in the direction opposite to the x axis.

In practice, the preceding integral can be calculated in each profile $(y_j,\ z_k)$, either numerically, from the discretized values $\mathcal{E}\,(x_i,\ y_j,\ z_k)$ of $\mathcal{E}$ (e.g., the trapeze method in x), or analytically, on the basis of the detection of the relative maximum $(\mathcal{E}_{peak}\,(y,\ z_k))$ and minimum $(\mathcal{E}_{trough}\,(y,\ z_k))$ values of the function $\mathcal{E}$. The preceding integral is also equal to the following expression:

$$\Delta D(y,z_k) = \frac{1}{K^{\beta}}\left[\sum_{peak=first}^{peak=end} \varepsilon_{peak}^{\ \beta}(y,z_k) - \sum_{trough=first}^{trough=end-1} \varepsilon_{trough}^{\ \beta}(y,z_k)\right]$$

By way of example, Figure 11 illustrates the contribution of a signal with a succession of peaks and troughs, produced by a multiple-axle load (six-wheel bogie), to the damage.

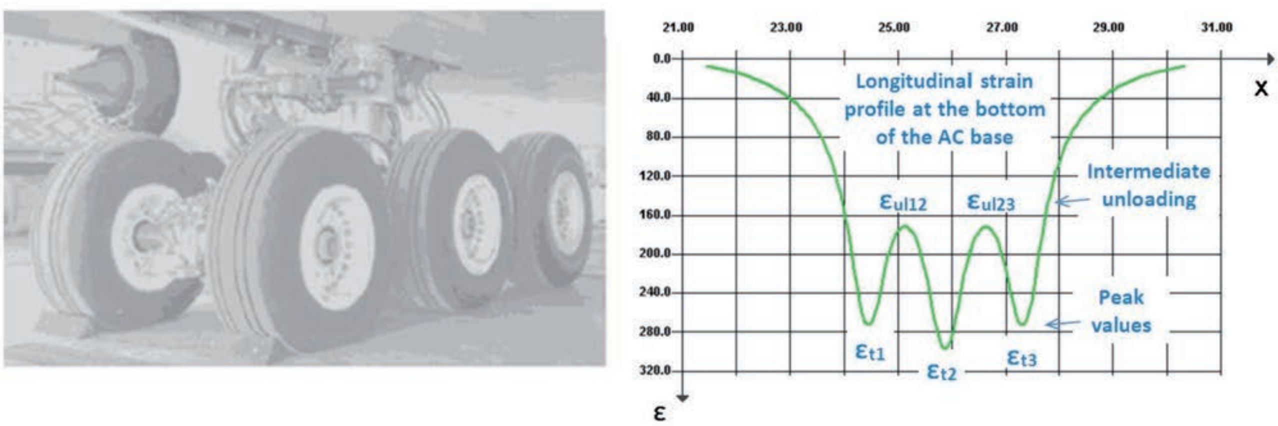

Figure 11: example of the longitudinal profile of deformation for six-wheel bogies

The damage associated with the longitudinal component of the strain caused by such loads is, according to the preceding formula, given by:

$$\Delta D_{tridem} = \frac{1}{K^{\beta}}\left(\varepsilon_{t1}^{\beta} - \varepsilon_{ul12}^{\beta} + \varepsilon_{t2}^{\beta} - \varepsilon_{ul23}^{\beta} + \varepsilon_{t3}^{\beta}\right)$$

In the end, the calculation establishes, for each aircraft and for each plane z_k examined, a transversal profile of elementary damage described by the set of values $\Delta D(y_j, z_k)$ for j varying between 1 and the number of longitudinal lines in the grid.

An example of a detailed damage calculation, that gives a concrete illustration of this statement, can be found in Chapter 8 (example 1).

2.5.4. Determination of elementary damage incorporating lateral wander

2.5.4.1. Definition of lateral wander

The passages of a given aircraft are off-centre to varying degrees in relation to the theoretical path that is centered on longitudinal axis of the section of pavement in question. This phenomenon is referred to as « lateral wander ».This phenomenon is illustrated in Figure 12.

Figure 12: aircraft landing off-centre from the runway axis

The distribution of these misalignments throughout the entire calculation duration of the pavement is assimilated to a centered Gaussian (or normal) distribution, with a standard deviation S_{bal} that depends essentially on the type of aircraft in question and its ground speed (see the values in paragraph 3.1.3.2).

The discretization of this law of probability, according to the transversal pitch Δy = 5 cm of the calculation grid (as defined in paragraph 2.4), results in the distribution of the paths on n_b lines $(y_j)_b$ of the grid, which are associated with percentages of the traffic $(P_j)_b$ (see annex B).

The integration of the lateral wander of an aircraft reduces the damage that would be caused by traffic on a single line: $\Delta D_{bal} < \Delta D$.

2.5.4.2. Calculation of ΔD_{bal}

This calculation is again based on the additive effect of the damage. It consists of adding up the damage profiles $\Delta D(y, z_k)$ calculated in 2.5.3, offset by the value $(y_j)_b$ and weighted with their probability of occurrence $(P_j)_b$ in the lateral wander law:

$$\Delta D_{bal}(y_j, z_k) = \sum_{b=1}^{n_b} (P_j)_b \times \Delta D(y_j - (y_j)_b, z_k)$$

2.5.5. Determination of the cumulative damage for the project traffic

Ultimately, the cumulative damage for all aircraft is given by the following relation, by applying the postulate of the additive effect of damage of the Miner law:

$$D_{bal,cumulated}\left(y_j, z_k\right) = \sum_{aircraft} N_{aircraft} \Delta D_{bal,aircraft}\left(y_j, z_k\right)$$

where:

aircraft = the index representing the types of aircraft in the traffic that is taken into consideration for the design,

$N_{aircraft}$ = the aggregate number of passages of the aircraft in question during the design period,

$\Delta D_{bal,\ aircraft}\ (y_j,\ z_k)$ = the damage profile of the aircraft in question, calculated according to the principles described above.

The result of this calculation is a curve representing the variation of the cumulative damage, according to the transversal position in relation to the axis of the section in question. An example of this type of curve is shown below (Figure 13).

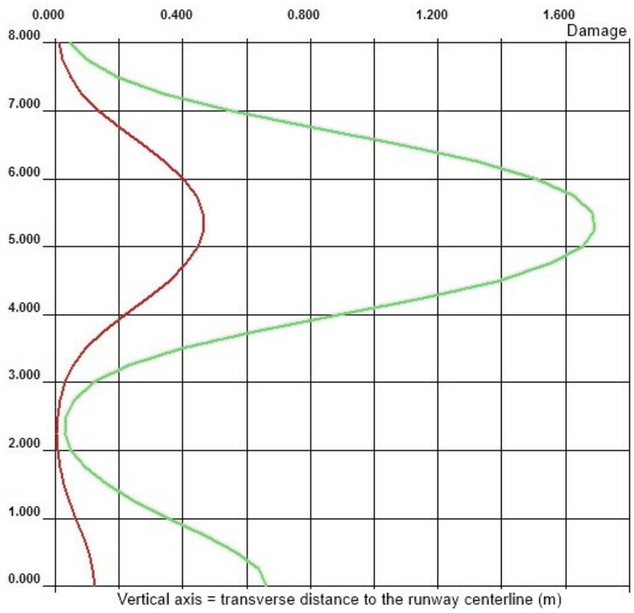

Figure 13: example of damage profiles (considering two criteria) for an A340-200

The different steps required to obtain these profiles are illustrated in detail in example 1 in Chapter 8.

2.6. Quantification of the aggressiveness of traffic : ESWL

The GAN [3] proposes a classification of aircraft according to two parameters: tire pressure and the number of wheels in the main landing gear. The product of these two parameters can be used to define « Groups » of aircraft (groups 1 to 5). Moreover, the GAN [3] defines « traffic classes » resulting from the combination of the group and the number of aircraft movements.

However, these traffic classes cannot be used to quantify the aggressiveness of traffic precisely enough.

The concept of damage, according to the principle of calculation described above, can be used to define a more rational approach to the characterization of the aggressiveness of traffic. This approach is based on the notion of the Equivalent Single Wheel Load (ESWL), which is illustrated in Figure 14, and can be defined as follows:

The ESWL associated with an aircraft traffic mix and a pavement structure is the simple, non-wandered[6] load (in tons) applied 10,000 times to the structure, with a footprint with a radius of 0.20 m, which produces the same value of fatigue damage of the asphalt concrete as the complete traffic mix.

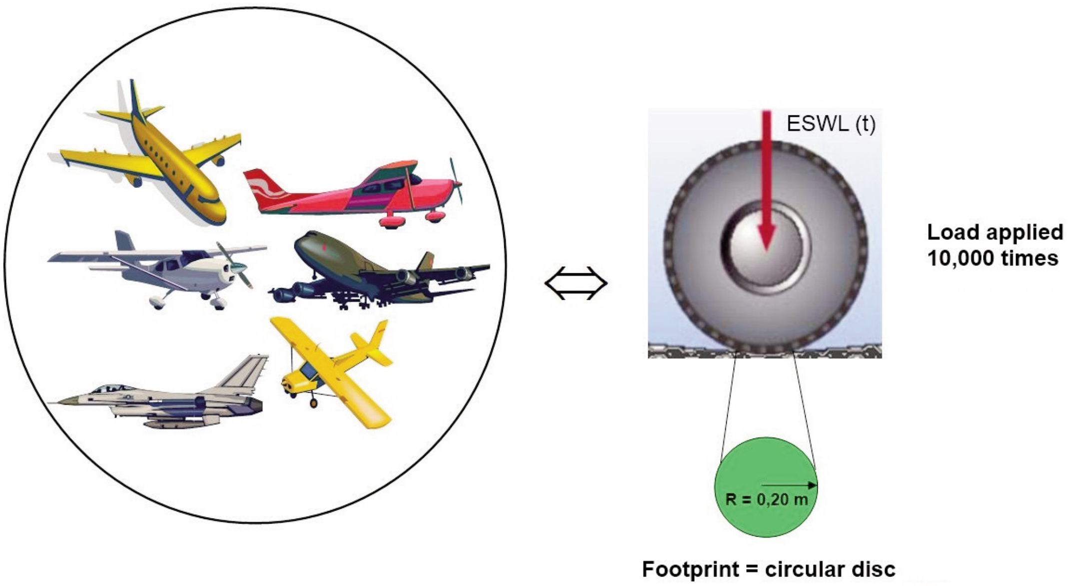

Figure 14 : diagrammatic representation of the ESWL

This definition of the ESWL is used to :

- ✓ decide on the possible placement of a base course made of granular materials (paragraph 1.2)
- ✓ check that the base course is thick enough, in view of current practice (paragraph 3.2.3)
- ✓ define the calibration coefficient (shift factor) for the bituminous materials (paragraph 2.7.1.2)

Since these three points are directly related to the bituminous materials, the ESWL is defined in terms of damage fatigue to the asphalt concrete, rather than damage to the granular materials by permanent deformation. In this way, the fatigue damage caused by actual traffic and by the ESWL is equaled for asphalt concrete fatigue.

[6] ***The phenomenon of aircraft lateral wander is defined in paragraph 3.1.3.2.***

Table 2 presents the values of the ESWL for different levels of simple traffic for structures designed at a temperature of 15°C and for a moderate speed section (e.g., a taxiway), with a risk factor of 5%. The materials used in these examples are EB-BBA 2 asphalt concrete in the surface layer and EB-GB 3 in the base course.

	Mrw (t)	Type of main landing gear	Mvts/d (for 10 years)	Associated structure				ESWL (t)
				BBA2	GB3	GNT	PF	
ATR 72-101	20.0	twin	1	6 cm	8 cm	25 cm	PF1	6.0
	20.0	twin	25	6 cm	12 cm	45 cm	PF1	11.5
	20.0	twin	50	6 cm	10 cm	30 cm	PF2	13.0
A320-200	77.4	twin	1	6 cm	10 cm	33 cm	PF2	14.0
	77.4	twin	25	8 cm	14 cm	42 cm	PF2	28.4
	77.4	twin	50	8 cm	16 cm	50 cm	PF2	32.9
A300-600R	171.4	bogie	1	6 cm	10 cm	34 cm	PF2	14.8
	171.4	bogie	25	8 cm	18 cm	53 cm	PF2	32.0
	171.4	bogie	50	8 cm	20 cm	62 cm	PF2	37.3
A340-500	381.2	bogie	1	6 cm	12 cm	27 cm	$PF2^{qs}$	18.9
	381.2	bogie	25	6 cm	15 cm	53 cm	$PF2^{qs}$	34.9
	381.2	bogie	50	8 cm	18 cm	52 cm	$PF2^{qs}$	42.8

Table 2 : examples of ESWL values for simple traffic

Note : *it is important to note that the value of the ESWL depends on the structure of the pavement, and therefore on the thickness of the asphalt concrete subjected to fatigue. Therefore, different values of the ESWL are possible for the same traffic.*

2.7. Coefficient K of the fatigue law for asphalt concretes

The design method provides for two types of adjustments of coefficient K of the Wöhler-Miner law obtained in the laboratory. One is attached to the conditions under which the fatigue test is performed and the other to the structural calculation.

The corrections made to the fatigue test take account of:

- the transposition of the fatigue test temperature of 10°C to the equivalent temperature considered in the design,
- the dispersion of the results of the fatigue test,
- an empirically established calibration coefficient or shift factor that provides for the change from the laboratory scale to the in situ behavior of the pavements.

The strain values from the multi-layer linear elastic model of the pavement are impacted by two effects:

- the dispersion of the thickness of the bituminous layers inherent in the construction of airfield works,
- the inconsistency of the bearing capacity inherent in the granular layer above the bituminous layer in the calculation.

In global terms, these adjustments are expressed by the following equation for coefficient K in the damage law:

$$K = 10^{6/\beta} k_{\theta f}\, k_s\, k_r k_c \bar{\varepsilon}_6$$

- $\bar{\varepsilon}_6$ is the value of the strain after 1 million cycles (ε_6) (*in* $\mu strain$) experimentally determined in the laboratory at 10°C and 25 Hz,
- β is the exponent determined on the basis of the fatigue test ($\beta = -1/b$), where *b* is the gradient of the fatigue test for the material in question,
- The coefficient $k_{\theta f}$ is used to transpose the results of fatigue tests on bituminous materials at 10°C and 25 Hz to the behavior at the equivalent temperature of the geographical site in question (15°C in mainland France and Corsica) and at the frequency *f*,
- k_s is a coefficient equal to 1 or a reduction factor of the permissible deformation of the bituminous materials, according to the rigidity of the underlying unbonded layer,,
- Coefficient k_r takes account of both the dispersion of the fatigue tests and the variation in the thickness of the base course made of bituminous materials.
- The calibration coefficient k_c depends on the nature of the bituminous materials. It takes account of the deviations observed between the calculations and the damage to the actual pavements.

The expression of these coefficients is explained in detail in the following paragraphs.

2.7.1. Coefficients relating to the fatigue law of the material

2.7.1.1. Temperature and frequency correction: $k_{\theta f}$

Since the pavement is generally designed at a temperature θ_{eq} different from 10°C (see paragraph 3.1.4) and at a variable frequency of loading, a first correction is made to the strain ε_6 (10°C, 25 Hz) = $\overline{\varepsilon_6}$, on the basis of the relation below:

$$\varepsilon_6(\theta_{eq}, f) = k_{\theta f}\ \overline{\varepsilon}_6$$

where:

$$k_{\theta f} = \sqrt{\frac{E(10°C, 10Hz)}{E(\theta_{eq}, f)}}$$

where:

- ε_6 (θ, f) is the value of the permissible strain after 1 million cycles (in µstrain), at the temperature θ (in °C) and at the frequency f (in Hz),
- E (θ, f) is the value of the norm of the complex modulus ($E = |E^*|$) *(in MPa)* at the temperature θ (in °C) and at the frequency f (in Hz). The map of the module E^* (θ, f) as a function of θ and f can be determined on the basis of conventional test campaigns (EN 12-697-26 [20]). Otherwise, the representative values in Tables 17, 19 and 22 in Chapter 6, taken from the curves showing sensitivity to temperature and frequency in Annex E, can be used.

Note: *the coefficient $k_{\theta f}$ should theoretically bring the module into play that corresponds to the experimental conditions of a fatigue test. However, the influence of the frequency (10 Hz instead of 25 Hz) is neglected in this case in order to guarantee the continuity of the calculation method with roads, for which this coefficient was defined.*

2.7.1.2. Prediction/observation calibration coefficient k_c

The calibration coefficient (or shift factor), k_c, corrects the discrepancy between the predictions of the calculation method and the observation of the behavior of experimental pavements. Amongst other things, this coefficient implicitly integrates the effect of the discrepancies between the mechanical loadings to which the material is exposed *in situ* and the loadings reproduced in the laboratory tests (e.g., a state of biaxial stress with rotation of the stresses produced by a rolling load, instead of the state of uniaxial stress with a constant direction of strain in the tests on trapezoidal specimens, distinct loading kinetics of the sinusoidal curve imposed in the laboratory, periods of rest between load from traffic, etc.).

Note that the use of the values of k_c cannot be dissociated from the calculation model proposed in this method and from the bending fatigue test protocol for trapezoidal specimens in EN 12-697-24 [19].

Unlike for roads, the high variability of airfield traffic in terms of loads leads to the adoption of a parameter k_c determined by the aggressiveness of the traffic (through the ESWL parameter). The evolution law considers the following points::

✓ for light traffic, the loads at the wheel of aircraft are of the same order as loads on roads (3.25 t per wheel load). Therefore, in order to guarantee the continuity of the rational method between these two fields of application, a calibration coefficient that is identical to roads is recommended (k_c = 1,3 for asphalt concrete, k_c = 1 for high-modulus asphalt concretes EB-EME 2),

✓ for heavy traffic, the values of the calibration coefficients are taken from observations of experimental analyses conducted by Airbus (A380 - Pavement Experimental Program (flexible PEP) [21] and High Tire Pressure Test (HTPT) [22]).

The notions of light and heavy traffic are expressed in terms of the ESWL, used to define the laws of evolution $k_c = f(ESWL)$. The latter are expressed analytically below and graphically in Figure 15.

For the materials EB-GB 2, EB-GB 3 and EB-GB 4 :

$$\begin{cases} k_c = 1.3 & \text{if } ESWL < 10t; \\ k_c = \dfrac{7}{150}.ESWL + \dfrac{5}{6} & \text{if } 10t \le ESWL \le 25t; \\ k_c = 2 & \text{if } ESWL > 25t; \end{cases}$$

For the material EB-EME 2 :

$$\begin{cases} k_c = 1 & \text{if } ESWL < 10t; \\ k_c = \dfrac{1}{30}.ESWL + \dfrac{2}{3} & \text{if } 10t \le ESWL \le 25t; \\ k_c = 1.5 & \text{if } ESWL > 25t; \end{cases}$$

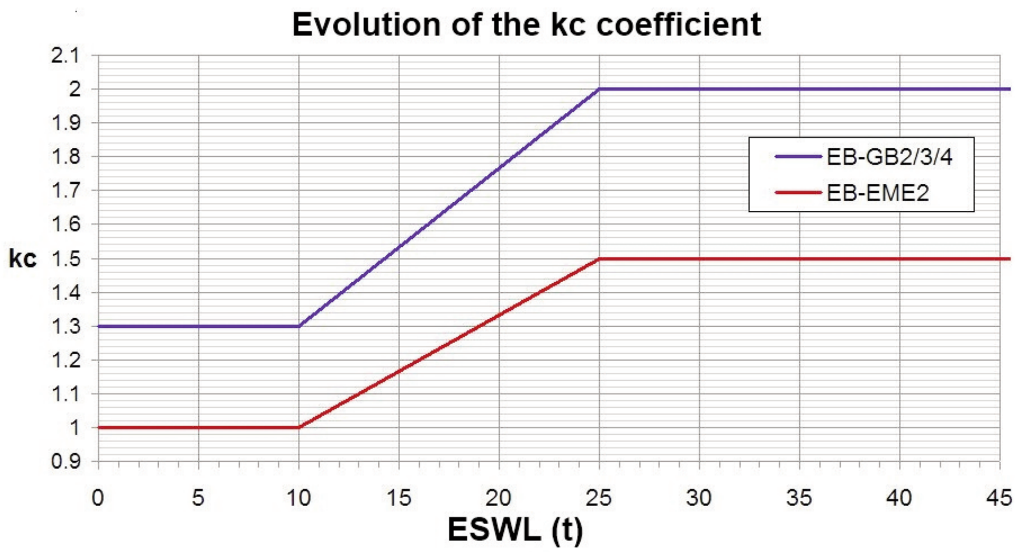

Figure 15 : evolution of the calibration coefficient k_c.

2.7.2. Coefficient relating to the structural calculation: the foundation coefficient k_s

Coefficient k_s is a reduction coefficient of de K that takes account of the effect of local inconsistencies in the bearing capacity of a low-rigidity granular layer supporting the bonded courses. This coefficient depends on the modulus of the granular layer directly beneath the bituminous base course. Table 3 shows the various cases.

Modulus of the layer directly beneath the bituminous layer in question	*E < 50 MPa*	*50 MPa ≤ E < 80 MPa*	*80 MPa ≤ E < 120 MPa*	*E ≥ 120 MPa*
k_s	*1/1.2*	*1/1.1*	*1/1.065*	*1*

Table 3 : values of coefficient k_s.

2.7.3. The risk coefficient k_r

The rational approach to the design of pavements is probabilistic in character, due to the dispersion of the resistance to fatigue of the bituminous materials and the dispersion of the thicknesses of the layers of the pavements, which both have a significant effect on the strength of the pavement over time. Therefore, this leads to the introduction of the risk coefficient k_r, defined according to a design risk r and the standard deviations S_N et S_h of the normal laws of probability that are taken into consideration to respectively demonstrate the dispersion of the fatigue tests and of the thickness of the bituminous layers upon construction.

The design risk, which belongs to the project data (see paragraph 3.1.1), represents the expectancy, in the sense of probabilities, of the length of pavement to be rebuilt in the absence of any overlay works at the end of design period.

In practical terms, this choice depends on the strategy of the contracting authority or its agent, which is based in particular on socio-economic criteria and the requirement to maintain the level of service assigned to the construction.

The precise definition of the parameters S_N, S_h and r, and the calculation of the function $k_r(r; S_N, S_h)$ are described in Annex C.

Note that this explicit probabilistic approach only applies to bituminous materials and not to untreated materials or the supporting bed. For the latter, the risk of deterioration is implicitly taken into consideration through the calculation of their permissible strains.

3. Implementation of the design method of new flexible pavements

3.1. Data required to design the structures of new pavements

The data required to apply the design method can be classified in four categories:

- strategic data (calculation parameters, such as the design period and the calculation risk),
- traffic data,
- climatic and environmental data. This data describes the climatic conditions on the site that directly influence the mechanical behavior of the pavement and the freeze-thaw verificationl,
- the parameters describing the materials. This data corresponds to the properties of the materials in the pavement and the pavement foundation that are required to calculate the fields of strains inside the pavement structure and to calculate the damage.

3.1.1. Design period and risk

These parameters are defined by the contracting authority.

The design period of a flexible airfield pavement is usually 10 years.

The risk is chosen according to the importance of the airfield. We propose to define the risk according to the traffic class (see glossary, VI), in the sense of the GAN [3], planned on the airfield. The risk for an airfield with an **annual traffic class higher or equal to CT3** (see paragraph 2.4.1 of the GAN [3]) **can be set at 2.5 %**. For airfields with an **annual traffic class lower than CT3**, or higher than CT3 but with the possibility of transferring the traffic to another runway or taxiway during the maintenance or repair works, **a risk of between 5 and 10% can be chosen**.

3.1.2. The traffic

The pavements to be designed must be defined according to the types shown in Table 1 (paragraph 2.2). The traffic data must be compiled for each of the sections.

Two sections are distinct when:

- the loads they are subjected to are different,
- they fulfill different functions, even if they are subjected to the same loads,
- the number of movements of the different loads they are subjected to is different.

3.1.2.1. General mode of description of the traffic

The traffic expected on the section in question is defined by the list of aircraft likely to use the section, combined with the planned number of passages, over the planned design period.

Each aircraft is characterized by:

- its type,
- the cumulative number of passages in the design period. This value can be determined on the basis of the daily or weekly traffic and the foreseeable increment (geometric or arithmetic) over the entire design period,
- the geometry of its landing gear and its loading conditions: the coordinates of the wheels in the horizontal plane, the weight supported by each wheel and the contact pressure between the tire and the pavement (see paragraph 3.1.2.2), which is usually assimilated with the tire pressure.

The latter parameters are defined in the STAC's « Ficav » database, which contains the characteristics of the 250 most common aircraft.

The data must be completed by the following information, which differs between aircraft and the type of area being studied :

- the speed of movement of the aircraft (see paragraph 3.1.3.1),
- the lateral wander of the aircraft, characterized by the standard deviation of the distribution of its longitudinal trajectories, assimilated to a normal centered distribution (see paragraph 3.1.3.2).

If the project manager does not have the precise speed and lateral wander data, the indicative values contained in paragraph 3.1.3 of this guide can be used.

3.1.2.2. Aircraft weight

The data supplied by aircraft manufacturers that are useful for design purposes includes :

- the maximum weight for maneuvers on the ground shown on the airworthiness certificate (or the **maximum ramp weight Mrw**), corresponding to the maximum acceptable weight of the aircraft during maneuvers on the ground on aprons
- the **maximum takeoff weight Mtow** on the airworthiness certificate, which corresponds to the maximum acceptable weight of the aircraft on takeoff.
- the **maximum landing weight Mlw** on the airworthiness certificate, which corresponds to the maximum acceptable weight of the aircraft when landing.
- the **maximum zero fuel weight Mzfw**, which corresponds to the weight of the empty aircraft, with its permanent equipment, cabin fittings and standard crew.

The effective weight of an aircraft is always between the Mzfw and the Mrw. It equals the Mzfw, plus the weight of the fuel and the payload.

In practice, the Mtow and the Mrw can considered to be practically the same, the only difference being the quantity of fuel consumed between the apron and the start of the runway.

On the other hand, the landing weight differs from the takeoff weight by the quantity of fuel consumed inflight, which accounts for a significant fraction of the total weight on long-haul flights (up to 30% of the Mrw).

It is necessary to determine the actual takeoff and landing weights to be taken into consideration in the design of the pavement, for all the aircraft making up the traffic. The contracting authority is responsible for providing this data to the project manager.

Since the thickness of the calculated flexible pavement is more sensitive to variations in load that to variations in the number of movements, it is important to precisely identify the loads applied by the aircraft traffic. Nevertheless, collecting this data can be difficult due to the uncertainty of the traffic forecasts, changes in the aircraft and the variation of the payload.

In the absence of more precise data, the weights supplied by the manufacturers and shown on the airworthiness certificates will be used:

- the **Mrw for takeoff.**
- the **Mlw for landing.**

Where appropriate, these values can be reduced due to operational constraints, such as runways that are not long enough to allow certain aircraft to take off when fully loaded.

3.1.2.3. Possible reduction of the list of aircraft taken into consideration

The least aggressive aircraft can be neglected in the design calculations in order to make the data collection more simple and less cumbersome. In this case, only the aircraft that really influence the design of the structure due to their weight and/or the number of passages in the design period of the pavement are taken into consideration in the calculation.

They can be selected according to the criterion defined below, which can be used to quickly assess the relative impact of each aircraft and its frequency on the behavior of the bituminous material courses and the soil.

For each aircraft j included in the traffic that is initially considered, the highest load at the wheel, Pr, is taken into consideration

The following ratio is calculated:

$$r_j = (\frac{Pr_j}{Pr_{max}})^5 . p_j$$

where:

- Pr_j the highest load at the wheel for the aircraft j,
- $Pr_{max} = max\,(Pr_j)$ the highest load at the of the entire traffic,
- $p_j = \frac{n_j}{n_{tot}}$ the percentage of the accumulated traffic represented by the aircraft j, rrelative to the total accumulated traffic, where:
  - ✓ n_j =the number of passages of the aircraft j,
  - ✓ n_{tot} = total number of passages of all the aircraft.

An aircraft is considered as a **decisive criterion if $r_j \geq 1\%$**.

3.1.3. Speed and lateral wander

3.1.3.1. Speed of movement of the aircraft

The speed of movement of the aircraft impacts the effective modulus of the asphalt concrete due to its viscoelastic character and, therefore, the fields of strain produced in the pavements. The speeds on the various sections therefore need to be specified.

On taxiways, all types of aircraft generally travel at about 30 kph.

Speeds on the runways depend on the type of aircraft and their position on the runway. For design purposes, the speed applied is 100 kph, even if much higher speeds can be reached, except for the first 300 meters, where a speed of 30 kph is taken into consideration.

Unless there are any particular reasons not to do so, the following speeds are proposed (Table 4), according to the type of section, as defined in Table 1 (paragraph 2.2).

These speeds are associated with frequencies of loading on the bituminous layers in order to calculate the modulus of rigidity of the asphalt concretes, according to the equation below. This equation is based on a relation of proportionality between speed and frequency and the following postulate: a speed of 100 kph corresponds to a frequency of 10 Hz[7], in other words:

$$f(Hz) = \frac{V(kph)}{10}$$

Pavement section	*Speed of movement in kph to be taken into consideration in the design calculations*
High-speed sections	***100***
Moderate-speed sections	***30***
Aprons and low-speed sections	***10****

**** a fictive values used in the calculation. See below « Special case of low-speed sections and aprons »***

Table 4: speed of movement of aircraft according to the type of section

Special case of low-speed sections and aprons:

The use of bituminous structures is not recommended for aprons, in view of the high risks of punching. This situation applies to areas associated with stress levels NS3 or NS4 (see glossary, VI) according to the GAN [3]. In this case, a rigid structure is recommended, the design of which is not covered by this guide.

When the use of a flexible pavement is deemed to be appropriate, it is designed by taking a calculation speed of 10 kph into consideration.

3.1.3.2. Lateral wander

Relative to the theoretical path centered on the longitudinal axis of the pavement to be designed (for high- and moderate-speed sections), the offset of the passages of a given aircraft varies and must be taken into consideration in the accumulation of damage. This reduces the damage caused by non-wandering traffic.

The distribution of these offsets throughout the design period of the pavement is assimilated to a Gaussian distribution defined by its standard deviation S_{bal}. The **amplitude of lateral wander** is the value corresponding to **twice the standard deviation of the offset variable**.

[7] This formula differs from roads, which associate a frequency of 10 Hz with loads travelling at 70 kph, due to the thickness of the bonded materials, which is usually greater, and the footprints, which are usually larger.

This amplitude depends on the type of aircraft and the type of section. It is high on high-speed sections, average on moderate-speed sections and concentrated on low-speed sections, including aprons. The following values are proposed, independently of the type of aircraft, unless any special reasons exist not to choose them:

Pavement section	*Standard deviation S_{bal} (in m)*
High-speed sections	*0.75*
Moderate-speed sections	*0.5*
Aprons and low-speed sections	*0*

Table 5: standard deviations according to the type of section

Recap of the traffic data required for design purposes.

The following information is necessary for each type of aircraft:

- ✓ its type,
- ✓ the accumulated number of passages over the entire design period, making the distinction between takeoffs and landings,
- ✓ the geometry of its landing gear and the loading conditions of its complete landing gear: the coordinates of the wheels in the horizontal plane, the weight supported by each wheel and the contact pressure between the tire and the pavement, which is usually assimilated to the tire pressure,
- ✓ the maximum ramp weight Mrw (on the airworthiness certificate) for takeoffs, unless more precise information is available,
- ✓ lthe maximum landing weight Mlw of the aircraft (on the airworthiness certificate) unless more precise information is available,
- ✓ the speed,
- ✓ the lateral wander amplitude.

3.1.4. Temperature data used to determine the modulus of the asphalt concretes

Climatic and environmental data allow the variations in the mechanical performances of the asphalt concretes according to temperature (modulus, resistance to fatigue) to be taken into consideration and to perform the freeze-thaw verification of the structure.

Data on the freeze-thaw verification of pavements is described in detail in Chapter 7.

Since the mechanical characteristics of bituminous materials (modulus, resistance to fatigue) are particularly dependent on the temperature, the temperature must be properly integrated in the design of the structures of flexible airfield pavements.

If detailed data is available (over a period that is sufficiently long to be representative) on the variations in the temperature on the project site and on the traffic, then it can be used to calculate the **equivalent temperature**. The equivalent temperature depends on the structure of the pavement, the damage criterion in question and the type of aircraft. It is defined as follows:

For a given aircraft, the equivalent temperature is defined as the constant temperature resulting in damage equal to the cumulative damage calculated at real temperatures.

The method used to calculate the equivalent temperature is described in Annex D.

If no data is available, a constant temperature is defined for the design calculations on the basis of the types of climates in Chapter 2.4.2 of the GAN [3].

Annex B of the GAN [3] indicates the type of climate to be taken into consideration for a number of towns in mainland France, Corsica and overseas territories. For regions that are not included in this annex, the project manager must either define the type of climate for the project, according to the criteria in Chapter 2.4.2 of the GAN [3], or proceed with a specific equivalent temperature investigation.

For oceanic, Mediterranean or continental climates (mainland France, Corsica and Saint Pierre et Miquelon), the equivalent temperature is **15°C**. For tropical climates (overseas territories), the equivalent temperature is **25°C**, except for Guiana, for which a temperature of **28°C** can be used.

3.1.5. Mechanical characteristics of the materials

To determine the mechanical characteristics of the materials, we will refer to Chapter 4 for the foundation materials, and to Chapter 6 for the materials making up the pavements themselves.

3.2. The steps of the design process

This section describes the different steps to be followed to make the design calculation.

As part of the preparation of this guide, a specific software, developed jointly by the IFSTTAR and the STAC, was programmed to automatically implement all of these steps.

3.2.1. Step 1 – Pre-design

Once the necessary data has been collected, we proceed with:

- a first choice of the wearing course, according to the principles outlined in Chapter 5,
- the pre-design of the structure, by referring to comparable situations.

3.2.2. Step 2 – Damage calculations of the structure and iterations on the thickness of the layers

3.2.2.1. Damage calculations of the structure for given thicknesses of the layers

This step is essentially based on the aspects described in Chapter 2.

For a given set of thicknesses, the calculations are made in an (x, y, z), system of coordinates, where x is the longitudinal axis of the pavement (the direction of movement of the aircraft), y is the transversal axis and z is the vertical axis. .

The calculations are discretized according to a grid (x_i, y_j) for each dimension plane z_k, subject to a damage criterion, i.e., the base of lower layer of bonded materials and the summit of the foundation.

1) For each plane z_k, the following sub-steps are completed.

For each of the aircraft in the traffic, the damage calculation is made in three sub-steps:

- the calculation, at all points (x_i, y_j) of the main major deformation ε_t[8] (horizontal extension at the base of the bonded layer) and of the deformation ε_{zz} (vertical contraction at the summit of the unbonded layer). The calculation is made for a given aircraft and the values of the modulus of rigidity of the layers of the pavement structure are adjusted beforehand according to its frequency of loading and its temperature.
- calculation of the transversal profile $\Delta D\ (y_j, z_k)$ of the damage increment created by the aircraft in the absence of lateral wander, for each of the design criteria taken into consideration. The calculation is based on the continuous integral of damage according to Miner's principle, mentioned in Chapter 2. It is also based on the adjustment coefficients described in the same chapter.
- calculation of the profile $\Delta D_{bal}\ (y_j, z_k)$ for each aircraft, taking the lateral wander into consideration.

[8] *This deformation is the greatest of the specific values of the 2x2 matrix:* $\begin{bmatrix} \varepsilon_{xx} & \varepsilon_{xy} \\ \varepsilon_{xy} & \varepsilon_{yy} \end{bmatrix}$

2) For the design period, the calculation of the cumulative damage :

$$D_{bal,cumulated}\left(y_j, z_k\right) = \sum_{aircraft} N_{aircraft} \Delta D_{bal,aircraft}\left(y_j, z_k\right)$$

and determination of the maximum damage per plane z_k :

$$D_{\max,cumulated}(z_k) = \underset{y_j}{Max}\{D_{cumulated}(y_j)\}$$

A design is then said to be « permissible » if **each of the values $D_{max,\ cumulated}$ (z_k) is lower than 1**.

3.2.2.2. Iteration on the layer thicknesses

As a general rule, the preceding step is repeated on different layer thicknesses in order to obtain a quasi-optimal permissible solution, according to the project data and constraints (direct and indirect costs, operating constraints, etc.).

Such a solution could ideally meet one or more criteria by attempting to satisfy one or more $D_{max,\ cumulated}$ (z_k) = 1 equalities.

3.2.3. Step 3 – Adjustment of the calculated thicknesses

The thicknesses of the layers determined at the end of Step 2 are then adjusted in order to :

- take account of the technological constraints of the minimum and maximum thicknesses (according to the type of material) in order to reach the degree of compaction and roughness targets defined in chapter 6,
- reduce the risks of bonding faults at the interfaces, by reducing the number of interfaces.

Furthermore, in order to avoid a pavement structure with an insufficient thickness of asphalt materials, Figure 16 includes, **as an indication**, minimum and maximum thicknesses of the base course materials to be used, in line with current practices. These thicknesses depend on the ESWL parameter.

It is important to mention that these minimum thickness values can be adapted according to implementation criteria, the thickness of the selected surface layer and the formulation of the asphalt materials in the base course (granulometry).

The relations below express the equations of the curves used to perform this verification. The values of e_{mini} are in centimeters, and depend on the type of material (GB or EME).

For EB-GB materials :

$$\begin{cases} e_{mini} = 8 & \text{if } ESWL < 10t; \\ e_{mini} = \frac{2}{5}.ESWL + 4 & \text{if } 10t \le ESWL \le 30t; \\ e_{mini} = 16 & \text{if } ESWL > 30t; \end{cases}$$

For the material EB-EME 2 :

$$\begin{cases} e_{mini} = 6 & \text{if } ESWL < 10t; \\ e_{mini} = \dfrac{3}{10}.\, ESWL + 3 & \text{if } 10t \le ESWL \le 30t; \\ e_{mini} = 12 & \text{if } ESWL > 30t; \end{cases}$$

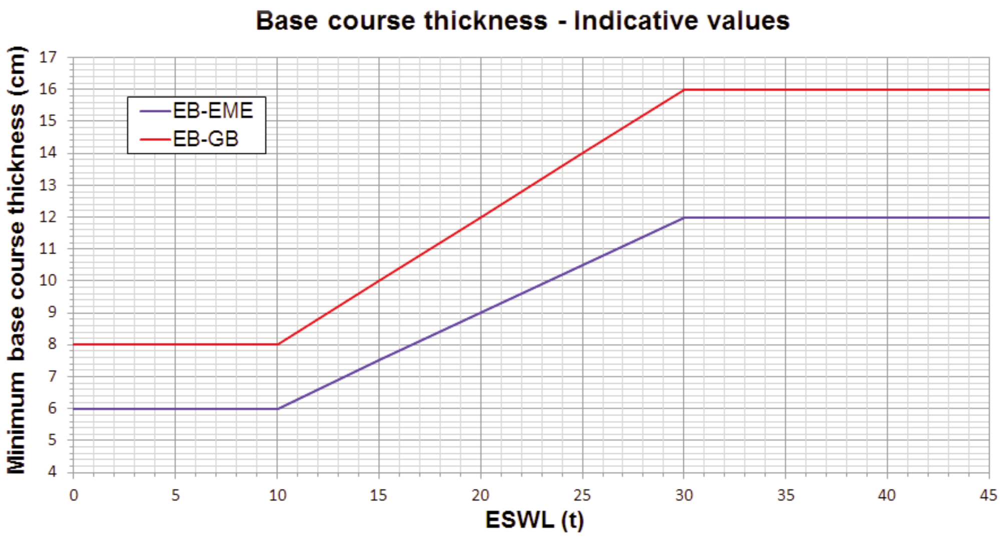

Figure 16 : graph used to check the minimum thickness of the base course

Note : *This graph does not apply for cases where:*

- *the design result leads to a sub-base layer thickness lower than the minimum values as indicated in paragraph 6.7.4.*
- *the base layer is made of granular materials (condition of low traffic with group 1 or 2 aircrafts and ESWL<10 t).*

3.2.4. Step 4 – Specific to the design of aprons and waiting areas

For aprons and waiting areas (see Figure 5), an additional calculation is made, in which the asphalt materials of the surface layer and the base course are modeled as an untreated graded aggregate, associated with a modulus of 800 MPa, in order to take account of the static nature of the loading. Parameter *K* in the damage law associated with the material in the foundation is increased to 24,000.

In this case, it is necessary to verify that the damage to the foundation by permanent deformation is less than 1. If this is not the case, the calculations must be repeated from Step 1, with new hypotheses for these thicknesses of the layers.

3.2.5. Step 5 – Freeze-thaw verification

The freeze-thaw verification is a separate operation performed at the end of Step 3 (or Step 4 for aprons or waiting areas). The principles and method of this verification are described in Chapter 7. If the result of the verification is insufficient, the thicknesses of the layers must be reviewed accordingly.

3.2.6. Step 6 – Definition of the longitudinal profiles and transversal cross-sections

The definition of the pavement structure is completed by defining the longitudinal profiles and the transversal cross-sections according to the applicable civil aviation recommendations (TAC decree, 10 July 2006 and its annex [5], Annex 14 of the ICAO, for international civil aviation [23]).

4. The pavement foundation

This chapter recaps the outline of the characterization of the foundation on which the pavement rests. Its characteristics affect the mechanical strains and stresses on the layers of the pavement. They constitute input data that is necessary in the rational design method.

4.1. Definition and référence of documents

The pavement structures are built on an assembly made up of:

- the subgrade (excavated earth or backfill, soil already in place or added), the uppermost part of which (about 1 meter) is called the **upper part of the earthworks (designated as PST), and of which the surface forms the level surface of earthwork (designated as AR)**;
- a capping layer, between the upper part of the earthworks and the pavement layers, the surface of which constitutes the **foundation**. The purpose of this layer is to make the characteristics of the support consistent and to produce and maintain the geometry and the mechanical, hydraulic and thermal performances used as hypotheses in the design and calculation of the pavement.

When the quality of the subgrade meets the requirements of a capping layer, the foundations and the level surface of earthwork are one and the same and can, if necessary, be limited to a simple grading course.

The design method of the structures making up the foundation of the pavement is elaborated in GTR [11] and the method for capping layers treated with lime and/or a hydraulic binder is described in the GTS [12].

The GTR [11] proposes a classification of soils and rock materials with identical behaviors for reuse in the earthworks. This classification also appears in NF P 11-300 [13]. For each of these classes, the GTR [11] defines the conditions of use and compacting that achieve the characteristics of the target structure, as backfill or as a capping layer.

The GTS [12] covers soil improvement techniques by treating with lime and hydraulic binders. It also defines the investigations to be made for the design of capping layers produced by treating the materials on the site, supplementing the GTR [11].

Figure 17: production of backfill and capping layers (GTR [11]) – Treatment of soils with lime and/or hydraulic binders (GTS [12])

The GTR [11] and the GTS [12] (Figure 17) propose an integrated approach to the design of the capping layer that results in a definition of the long-term performance of the pavement foundation, apart from freezing aspects. The remainder of this document recaps the major principles and specifies the hypotheses, models and the data required to describe the subgrade and the capping layers in the design of pavement structures, with reference to Chapters 2 and 3.

4.2. Characterization of the upper part of the earthworks

Initially, the design of the capping layer and of the structure of the pavement is based on the mechanical characterization of the upper part of the earthworks.

4.2.1. Identification of the materials and conditions of use

The materials are identified with reference to the standard for the classification of soils, rock materials and industrial sub-products (NF P 11-300 [13]), which is based on :

- parameters describing the granularity and clay content,
- the water content at the time of construction and, possibly, in the long term,
- behavioral parameters, taken from mechanical tests, to assess the evolutive character of the materials over time.

On the basis of this classification and the meteorological situation at the time of the construction works, the GTR [11] specifies the conditions of use (as is or with treatment) of the site materials, and the procedures for use..

4.2.2. Hydrous environment

Depending on the hydrological conditions (presence of water tables, infiltrations of water) and the construction characteristics of the project (route, geometry, drainage and sewerage systems), the hydrous condition of the subgrade and, by the same token, the mechanical characteristics of materials that are sensitive to water, change over time.

The drainage systems require particularly close attention and careful execution. The most frequently observed defects, in terms of durability, are related to poor water management with regard to the structures. Their design is described in the technical guide – « Design of aerodrome drainage networks », STBA (2000) [14], the SETRA technical guide to “Road drainage”, 2006 [15] and in the SETRA information memo N°120 « The contribution of drainage to the design of pavement foundations », [16]. The design must also take account of the corresponding regulations and legislation, such as the Water Law, the European Framework Directive, etc., to which the project manager must refer, but which are not described in detail in this document.

4.2.3. Classification of the upper part of the earthworks

On the basis of the above input (the nature and state of the material at the time of construction and the hydrous environment), the GTR [11] defines and describes seven cases for the upper part of the earthworks.

For class D2 materials (alluvial gravel mixes, sand), which are not included in the tables in the GTR [11], refer to case N°6, for materials with a modulus higher than 120 MPa at the time of use and in the long term (otherwise, case N°5).

The case of B3 materials is more complex. Even if they have relatively low clay content, with $0.1 < VBS < 0.2$, their insensitivity to water must be verified. The geotechnician decides on the definition of the upper part of the earthworks to be taken into consideration.

4.2.4. Bearing capacity classes of the level surface of earthwork

The description of the upper part of the earthworks is completed by the **long term** deformability on the level surface of earthwork, for which four classes, AR0 to AR4 (Table 6), exist. .

Modulus (MPa)	*20*	*50*	*120*	*200*
Class of the level surface of earthwork	*AR1*	*AR2*	*AR3*	*AR4*

Table 6: classes of the long-term bearing capacity on the level surface of earthwork

For N°0 cases (level surface class AR0), it is first necessary to improve the situation (by purging, draining, etc.) in order to reach a long-term modulus higher than 20 Mpa, before creating the pavement foundation.

The drainage systems of the upper part of the earthworks (see Chapter 4.2.2) contribute to the quality and the durability of its bearing capacity. Depending on the hydrogeological situation on the site (backfill, presence of a water table, etc.), the drainage can descend to 0.50 m, or even 1.00 m, beneath the level surface of earthwork. Specific measures (spur dykes, infill / backfill transitions) can also be defined in the geotechnical investigation.

Comments:

1) The moduli defined in Table 6 are the **long-term values** used in the design calculations of the structures. In practice, they may vary significantly on site, when the level surface of earthwork is accepted, according to the conditions of construction and the period in which the tests are made.

Therefore, they can only theoretically be confirmed on the basis of the plate load tests, Dynaplaque tests, or the measurement of the deflection under the 130 kN axle, if the ground is insensitive to water, or if the water content at the time of the test is representative of the unfavorable water contents that may occur in the pavement when in use. Otherwise, the recommendations for the special technical specifications may result in opting for higher acceptance values.

2) The minimum **acceptance** requirements can be adapted according to the hydrogeological conditions (risk of a loss of bearing capacity during the design period), rideability and construction constraints, or the choice of the structure of the capping layer:

- for AR1, a minimum bearing capacity of 30 to 35 MPa may be imposed to meet the rideability requirements,
- for AR1, a minimum bearing capacity of 35 to 40 MPa may be imposed if a treated capping layer is chosen,
- for AR2, a minimum bearing capacity 80 MPa may be imposed to guarantee the quality of the rock materials used in the level surface of earthwork or the quality of the lime or hydraulic binder treatment,
- finally, a minimum acceptance bearing capacity higher than the values in Table 6 may be defined by the geotechnician, for example 30 to 40 MPa for cases N°1, N°2 or N°3, to anticipate a possible loss of bearing capacity in the long term, depending on the hydrous conditions on the site.

3) If there is a possibility of frost penetrating beyond the capping layer, it is up to the project manager, the geotechnical engineer or the enterprise to provide the information required to measure the sensitivity to frost (as per NF P 98-234-2 [24]) of the soil making up the upper part of the earthworks.

4.3. Capping player

4.3.1. Roles and design of the capping layer

The capping layer is a transitional course that allows the characteristics of the infill or the terrain in place to be adapted to the essential functions of a pavement foundation. It performs the following short- and long-term functions:

- **in the short term** (when the pavement is built), the capping layer must guarantee the minimum characteristics, in terms of:
  - ✓ rideability, to allow the vehicles transporting the materials of the base layer to maneuver correctly during the season when the works take place,
  - ✓ leveling, to guarantee the regularity of the thicknesses of the courses and the roughness of the finished pavement,
  - ✓ deformability (bearing capacity), to allow for the proper compacting of the layers of the pavement,
  - ✓ if necessary, resistance to frost, in order to protect the subgrade.

The acceptance criteria of the capping layer in terms of rideability, leveling and deformability are defined in Annex G.

- **in the long term** (when the pavement is in service), the capping layer:
  - ✓ homogenizes the characteristics of the support,
  - ✓ maintains the minimum bearing capacity over time,
  - ✓ contributes to drainage,
  - ✓ if necessary, protects the subgrade against frost.

The foundation is characterized by a long-term bearing capacity class (see paragraph 4.4.1).

Depending on the nature of the site (type of soils, climate, hydrogeological environment, site traffic, etc.), the capping layer can be:

- non-existent or reduced to a thin grading course, if the materials making up the infill or the soil in place, treated or not, possess the required qualities,
- made up of one or more layers of different materials (the materials in place or added, treated or not), possibly including a geotextile.

The design of the capping layer is defined in the GTR [11], and in the GTS [12] for treated capping layers. Depending on the upper part of the earthworks and the chosen materials, these documents recommend the thicknesses required to guarantee the short- and long-term functions, apart from frost, which demands a specific design process (see Chapter 7).

4.3.2. Capping layer materials

Certain materials can be used in the capping layer as they come, while others require changes to their nature and/or state in order to meet the following criteria:

- insensitivity to water,
- the dimensions of the largest elements,
- resistance to the movements of the site vehicles,
- insensitivity to frost, where appropriate*

(*) Depending on the reference frost index, the structures of thick pavements may be sufficient to provide protection against freezing/thawing for moderate indexes. As a general rule, it is preferable to use a material that is insensitive to frost (according to the swell test in NF P 98-234-2 [24]) and to frost wedging (according to the frost resistance test of aggregates in NF P 18-545 [25]). Choices of other materials must be justified by a specific analysis of frost penetration.

The conditions of use and implementation of the capping layer materials are defined in the GTR [11] and the GTS [12].

However, for untreated granular materials, it is advisable to use more restrictive criteria by selecting materials that comply with the standards EN 13-285 [18] and EN 13-242 [26], by assimilating them to an equivalent GTR class [11] with a densification target *q3*. This safe approach guarantees the durability of the capping layer when exposed to high aeronautical loads.

Certain specific local materials, waste or industrial sub-products can be used as infill, or even in the capping layer, by referring to the regional technical guides and the SETRA information memo N°114 on the execution of compacting tests [27].

4.3.3. Specifications of the components of untreated graded aggregate mixes

The aggregates used in untreated graded aggregate mixes should preferably comply with the categories chosen in EN 13-242 [26], unless a specific geotechnical investigation is made. This standard is codified according to its French transcription, NF P 18-545 [25].

The minimum characteristics for the aggregates in untreated graded aggregates are shown in Table 7

Use	*Minimum characteristics, as per NF P 18-545 [25]*	*Traffic class*				
		CT1	*CT2*	*CT3*	*CT4*	*CT5*
Capping layer	*Granularity*	*0/63*	*0/63*	*0/63*	*0/63*	*0/63*
	Mechanical strength of the gravel	*Code E*	*Code E*	*Code E/D**	*Code D*	*Code D*
	Manufacturing characteristic of the gravel	*Code IV*	*Code IV*	*Code IV/III**	*Code III*	*Code III*
	Manufacturing characteristic of the sands	*Code c*	*Code c*	*Code c/b**	*Code b*	*Code b*
	Angularity of the gravel	*Ang 4*	*Ang 4*	*Ang 3/Ang 2**	*Ang 2*	*Ang 2*

** If group 4 or 5 aircraft use the pavement, with more than one movement per day, the following will be taken into consideration:*
✓ a code D for the mechanical strength of the gravel
✓ a code III for the manufacturing characteristics of the gravel
✓ a code b for the manufacturing characteristics of the sands
✓ Ang 2 for the angularity of the gravel

Table 7: minimum characteristics of the aggregates making up the untreated graded aggregate mix

4.3.4. Specifications of the mixtures of untreated graded aggregate mixes

Table 8 shows the minimum recommended characteristics of the mix

	Minimum characteristics, as per EN 13-285 [18]
Code	*GNT 1*
Designation	*0/63 mm*
Fines content	*UF12* *LF2*
Maximum dimensions	*OC_{80}*
Specification envelope	*G_B*

Table 8: minimum characteristics of the mix

If the untreated graded aggregate mixes may be exposed to frost, it is necessary to guarantee:

- preferably, insensitivity to frost of the mix, verified in laboratory swell tests (NF P 98-234-2 [24]),
- the resistance to frost of the aggregates, as specified in NF P 18-545 [25]).

4.4. Classification of pavement foundations

The long-term bearing capacity of the pavement foundations is determined by the pair (upper part of the earthworks-level surface of earthwork) / capping layer.

The long-term bearing capacity classes of the pavement foundation are shown in Table 9.

Modulus (MPa)	*50*	*80*	*120*	*200*
Foundation class	*PF2*	*$PF2^{qs}$*	*PF3*	*PF4**

***The foundation class PF4 is shown for reference only, because, in practice, and in view of the specifics and constraints of loading on airfields, it is not used for this type of application**

Table 9: long-term bearing capacity classes of the pavement foundation

Comments:

These values correspond to the long-term moduli. They can vary significantly upon reception of the foundation, depending on the construction conditions and the period when the tests are made:

a) For granular capping layers, they can only theoretically be confirmed on the basis of the plate load tests (NF P 94-117-1 [28]), Dynaplaque tests (NF P 94-117-2 [29]), or the measurement of the deflection under the 130 kN axle, if the materials in the upper part of the earthworks are insensitive to water, or if the water content at the time of the test is representative of the unfavorable water contents that may occur in the pavement when in use. Otherwise, the recommendations for the special technical specifications may result in opting for higher acceptance values.

b) For capping layers treated with hydraulic binders, these modulus values can never be measured by measuring the modulus of deformability in plate or Dynaplaque tests. The modulus of a treated capping layer is defined by laboratory investigations and continuous inspections, which guarantee that the final quality is reached, as defined in the SETRA information memo N°118 « Key aspects of the quality assurance of treated capping layers », 2009 [30].

Once the bearing capacity class of the level surface of earthwok and the material of the capping layer are known, the foundation is classified as follows:

- when the capping layer is at least as thick as the recommended thicknesses in the GTR [11], the class of the foundation is given in the tables in the latter document,
- when the capping layer is thinner than the recommended thickness, the class of the level surface of earthwork shall apply.

The design rules of the capping layers with regard to the target foundation classes are described in the GTR [11] and the GTS [12]. Annex F describes the outlines.

The possible presence of a **capping layer** in the linear elastic model is taken into consideration through the foundation class and, therefore, **is not considered individually as a layer of the pavement**.

4.5. Special case of capping layers treated with hydraulic binders

Current feedback in France on the use of capping layers treated with hydraulic binders is limited in the aeronautical sector.

It would not be satisfactory to outlaw their use for this reason, in particular since the use of these structures can be of interest as part of the application of a sustainable development policy and in order to save natural resources.

Today, it seems reasonable to authorize the use of these structures, provided that a few precautionary principles are applied, expressed in this case in terms of **the minimum thickness of sub-bases made of granular materials, the drainage systems and measures to limit the propagation of random cracks in the body of the pavement** (e.g., pre-cracking).

Hydraulic binders make capping layers very rigid. In view of the scale of airfield loads, this capping layer is likely to be exposed to significant strains and stresses. If it deteriorates over time, and if the sub-base is thin (minimum technological thickness), the risk of excessive strains and stresses on the base course, and subsequently the deterioration of the pavement, is high. This risk is even higher when EME-type rigid bituminous materials are used in the base course. Therefore, in an effort to limit these risks, and in view of current knowledge, the following precautionary rule is proposed:

The sub-bases of pavements resting on a capping layer treated with hydraulic binders must be **at least 20 cm thick**, irrespective of the class of the foundation and of the traffic (as per the GAN [3]).

Finally, treated capping layers must only be used together with a **drainage system** to prevent water from being trapped in the layers of untreated materials.

4.6. Mechanical characteristics of the foundations with regard to the design of pavements

The pavement design method is calibrated by selecting the mechanical characteristics that correspond to the least favorable hydrous conditions for the pavement (excluding periods of freezing and thawing) and incorporating the effect of any drainage systems. In this way, the calculation is made in a degraded situation, which often occurs in the winter, without taking the more favorable summer period into consideration. Therefore, seasonal variations in the hydrous conditions are not integrated in the calculation.

4.6.1. Calculation of strains and stresses in the pavement under traffic

The class of the foundation includes the global workings of the « upper part of the earthworks + capping layer » assembly and constitutes a key element of the characterization of the structure. It influences the mechanical strains and stresses in the layers of the pavement and, therefore, their design.

For the standard structural design calculations of new pavements, the pavement foundation is modeled by a linear elastic half-space of infinite depth. It is characterized by a Poisson's ratio of 0.35 and a Young's modulus. The value of the latter is taken to be equal to the lower boundary of the long-term bearing capacity class PFi of the pavement foundation. However, for class PF4, the modulus used for design purposes is **adjusted to 120 MPa**, as shown in Table 10, in order to **adopt a safe approach to the aggressiveness of airfield loads**, whether the capping layer is treated or not.

Platform class	*PF2*	*PF2^qs^*	*PF3*	*PF4*
Modulus considered in the calculation (MPa)	*50*	*80*	*120*	*120*

Table 10 : moduli associated with the long-term bearing capacity classes of the pavement foundation for the design of airfield pavements

When a rigid substratum is located at a depth of between 2 and 6 m, its presence must be incorporated into the calculation model by dividing the supporting bed into two layers. One is of a finite thickness, h_{sub}, and the other, which represents the substratum, is of an infinite thickness, as stated in paragraph 2.4. The substratum is defined here as a horizon that can be considered as non-deformable in relation to the rigidity of the soil it supports.

When the substratum is located at a depth of 2 m or less, a specific investigation is necessary.

4.6.2. Permanent deformation of the foundation

In view of the risk of the rutting of the foundation due to the accumulation of permanent deformations by repeated rolling loads (cycles), the design method consists of limiting the amplitude of vertical strain of the platform ε_{zz} under the load of the traffic. The relation used to define the maximum vertical strain *(in $\mu strain$)* corresponding to a number of cycles *N* for a given load, is defined by :

$$\varepsilon_{zz\,\max} = K.N^b$$

N is the number of cycles resulting failure and $\begin{cases} K = 16000 \\ b = -0.222 \end{cases}$

5. Surfacing

Once all the data required to make the calculation has been collected, the first step of the design of the pavement structures consists of making an initial choice with regard to the definition of the wearing course, according to the type of infrastructure (runway, taxiway, apron, etc.), the performance requirements and the expected usage. This approach consists of choosing a type of product and specifying its thickness, within a given range of variation.

Note : *the rational design approach does not apply to the asphalt materials in the surfacing, because the damage mechanisms in this layer are different from those in the base layer. The thickness of the surfacing is determined in an empirical approach based on feedback from the field.*

5.1. Définition and reference documents

The surfacing of a pavement conditions its properties when in use (e.g., skid resistance and noise). It must also withstand creep, punching, aging due to the elements, changes in temperature and attack by hydrocarbons.

The surfacing is made up of :

- a wearing course that is in contact with the tires and must possess the skid resistance hat meet aeronautical specifications. It also fulfils a waterproofing function,
- and, possibly, a binder course, which is an intermediate layer between the wearing course and the base course. This layer is mainly fabricated during maintenance operations in order to improve evenness or to delay the propagation of cracks from the lower layers to the wearing course. In this case, the binder course is located between the wearing course and the old pavement.

The Guide to the Application of Standards (GAN) [3] describes the procedure for the selection of the products to be used in the surfacing and to define the performances of the asphalt concrete (formulations) and the characteristics of the components (aggregates and binders) in order to best meet the demands of the project. This document also contains the recommendations for the characteristics to be obtained in the laboratory (type testing) and on site, from the manufacture of the asphalt concrete to their use.

The rest of this chapter describes the guidelines applicable to the choice of the nature and the thickness of the surfacing defined in the GAN [3]. It is necessary to refer to this document in order to apply this approach.

5.2. Choice of the type of surfacing

The choice of the type of surfacing depends on the selected goals and the required performance. The economic aspects of the project must also be taken into consideration.

The choice depends on the type of aeronautical surface in question. It is based on criteria applying to resistance to the deterioration of airfield pavements, skid resistance and resistance to chemical attack (accidental spillages of oil or hydrocarbons).

The main forms of deterioration affecting airfield pavements are :

- shear, which results from the horizontal stress due the tangential loads transmitted by the tires when the aircraft turn,
- rutting (small radius), due to permanent deformation caused by repeated slow-moving rolling loads,
- punching, caused by permanent deformation due to static loads,
- ageing, which only affects the wearing course. Ageing depends on the climate, the nature of the products and any possible pollution. The capacity to withstand this factor is called « durability ».

The skid resistance of a pavement is characterized by the quality and the durability of the roughness of its surfacing. It is defined in terms of macrotexture and microtexture. The macrotexture is determined by the type of surfacing and the manner in which it is used, deterioration and occasional surface treatments. The microtexture is related to the irregularities in the surface of the aggregates in contact with the rubber tires.

Table 11, taken from the GAN [3], shows an assessment of the level of attack and the quality of the characteristics of the pavement surface, according to the type of section.

		Shear	Rutting	Punching	Durability	Skid resistance
Apron		++	+++	+++	++	++
Runway	Common section	+	+	+	++	+++
	Runway turn pad	+++	++	+	+++	++
	Runway exit	+++	+	+	++	+++
	Runway threshold (*)	+++	+	++	+++	+++
Taxiway	Common part	+	++	+	++	++
	Connections	++	++	++	+++	++
Apron or waiting area		+	+++	+++	++	++

***(*)** including the touchdown zone*
***+ :** Low*
***++ :** Moderate*
***+++ :** High*

Table 11 : assessment of the level of aggression and the quality of the surface characteristics of the pavement

5.3. Definition of the surface layer

The behavior of the surface layer must be examined from the perspective of its durability, skid resistance and its resistance to shear, rutting and punching.

The design method exposed in Chapters 2 and 3 is not intended to be applied to the design of the actual surface layer. The action of the traffic subjects the wearing course to complex stresses caused by the tires. The mechanical behavior of the wearing course is, at this point, only approached in terms of the normal stresses applied to the surface. However, the latter, which is modeled as a structural layer, is integrated in the calculation through its modulus.

The GAN [3] proposes that the materials making up the surface layers should be selected according to a notion known as the « stress level » which, for a given airfield, results from the combination of two factors: the traffic class and the type of climate.

- The traffic class is determined by the tire pressure, the number of wheels in the main landing gear and the number of aircraft passages per day (the frequency).
- Four types of climate have been defined on the basis of the average maximum daily temperatures in the two hottest months of the year and the two coldest months: type 1 is predominantly oceanic, type 2 predominantly Mediterranean, type 3 predominantly continental or mountainous, and type 4 is predominantly tropical.

The GAN [3] defines four stress levels, from NS1 to NS4 (see glossary, VI), according to which it then lists the products that can be used in the wearing course and the binder course for each type of section. It also gives the average thickness of the products, according to their granularity.

Tables 12 and 13, taken from the GAN [3], illustrate the selection of the wearing course and the binder course.

Pavement section		NS 1	NS 2	NS 3	NS 4
Apron		EB-BBA 2, ESU, ECF, EP, EB-BBM 1	EB-BBA 3, EB-BBM 2 EB-BBME 1, EP	(***) EP (1)	(***) EP (1)
Runway (*)	Common part	EB-BBA 1, EB-BBM A1, EB-BBM B1 BBTM	EB-BBA 1, EB-BBM A2, BBTM	EB-BBA 2	EB-BBA 2
	Runway turn pad		EB-BBA 2, EB-BBME 1	EB-BBME 2(2)	EB-BBME 3(2)
	Runway entrance/exit		EB-BBA 2, EB-BBM A2	EB-BBA 3 EB-BBME 2	EB-BBA 3
	Threshold (**)		EB-BBA 2, EB-BBM A2	EB-BBA 3(2)	EB-BBA 3(2)
Taxiways	Common part	EB-BBA 1, ECF, EB-BBM B2, BBTM	EB-BBA 2, EB-BBM B3, BBTM	EB-BBA 2, EB-BBME 1	EB-BBA 3, EB-BBME 2
	Connection		EB-BBA 2, EB-BBM B3	EB-BBA 3, EB-BBME 2	EB-BBA 3, EB-BBME 2
Waiting area		EB-BBA 1, ECF, EB-BBM B2, BBTM	EB-BBA 3, EB-BBM B3	EB-BBME 3	EB-BBME 3

(*) The use of EB10-BBA C is to be prohibited (relatively low geometric roughness).
(**) On military airfields, where combat aircraft can cause damage to asphalt concrete pavements (surface burns / spillage of hydrocarbons) : cement concrete pavements are recommended.
(***) On aprons, where the risk of punching is high, the use of cement concrete pavements is strongly recommended.
(1) Its use depends on the support (base course), which must have a high modulus of rigidity (e.g., semi-rigid or bituminous structure). As a general rule, the support is made up of either a hydraulic-bound graded aggregate, a high-modulus asphalt concrete or a base asphalt concrete.
(2) Modified binders are recommended for higher resistance to shear stress.

Table 12: excerpt from the GAN [3] - Products that can be used in the wearing course

<table>
<tr><th colspan="2">Pavement section</th><th>NS 1</th><th>NS 2</th><th>NS 3</th><th>NS 4</th></tr>
<tr><td colspan="2">Apron</td><td>EB-BBM 1
EB-BBSG 1</td><td>EB-BBM 3
EB-BBSG 1</td><td>([1])</td><td>([1])</td></tr>
<tr><td rowspan="4">Runway</td><td>Common part</td><td rowspan="4">EB-BBM 1
EB-BBSG 1</td><td>EB-BBM 1
EB-BBSG 1</td><td>EB-BBM 2
EB-BBSG 1
EB-BBME 1</td><td>EB-BBSG1
EB-BBME1</td></tr>
<tr><td>Runway turn pad</td><td rowspan="3">EB-BBM 2
EB-BBSG 1</td><td rowspan="3">EB-BBM 3
EB-BBSG 1
EB-BBME 1</td><td>EB-BBSG 2
EB-BBME 2</td></tr>
<tr><td>Runway entrance/exit</td><td rowspan="2">EB-BBSG 2
EB-BBME 2</td></tr>
<tr><td>Threshold</td></tr>
<tr><td rowspan="2">Taxiway</td><td>Common part</td><td rowspan="2">EB-BBM 1
EB-BBSG 1</td><td rowspan="2">EB-BBM 2
EB-BBSG 1</td><td>EB-BBM 2
EB-BBSG 1
EB-BBME 1</td><td rowspan="2">EB-BBSG 1
EB-BBME 1</td></tr>
<tr><td>Connection</td><td>EB-BBM 3
EB-BBSG 1
EB-BBME 1</td></tr>
<tr><td colspan="2">Waiting area</td><td>EB-BBM 1
EB-BBSG 1</td><td>EB-BBM 2
EB-BBSG 1</td><td>EB-BBM 3
EB-BBSG 1
EB-BBME 1</td><td>EB-BBSG 2
EB-BBME 2</td></tr>
</table>

([1]) Not applicable. (***) On aprons, where the risk of punching is high, the use of cement concrete pavements is strongly recommended.
Note: products with higher performance classes can be used, provided that the economic balance remains satisfactory. The products used do not usually contain polymer modified bitumen.

Table 13: excerpt from the GAN [3] - Products that can be used in the binder (and reshaping) course

6. Pavement materials

6.1. Bituminous materials

Materials treated with bituminous binders are mixes of aggregates and bituminous binders, with the possible incorporation of additives that are dosed, heated and mixed in the plant. Also known as asphalt concretes, they are used in the wearing course or the binder course (bituminous concretes) or in the base course (base asphalt concrete and high-modulus asphalt concretes). Table 14, taken from the GAN [3], shows the materials that can be used in airfield pavements. In more general terms, this guide to the design of new flexible airfield pavements should be used together with the practices described in the GAN [3].

	Products
Wearing course	*EB-BBA, EB-BBSG, EB-BBME, EB-BBM, BBTM, SMA, ESU, ECF, EP*
Binder course	*EB-BBA, EB-BBME, EB-BBM, EB-BBSG*
Base course	*EB-GB, EB-EME*

Table 14: products that can be used to for an airfield pavement

The GAN [3] covers hot mix asphalt concretes in detail, including the normative references, the choice of aggregates and the bituminous binder and the conditions of use in view of the stress level (NS) (see glossary, VI). It offers guides to choosing the bituminous products best suited to the loadings of each airfield section, according to the expected aircraft and the climate on the platform.

This guide specifies the product data that is of use for design purposes, and in particular the value of the modulus and the fatigue performance.

6.1.1. Normative framework

The series of standards EN 13-108 [17] defines the formula of an asphalt concrete for bituminous mixes on the basis of the general characteristics, completed by the empirical characteristics or fundamental characteristics. In this way, it defines two approaches: the so-called **empirical** approach, which includes the general and empirical characteristics, and the so-called **fundamental** approach, which includes the general and fundamental characteristics.

Note: *since these two approaches are incompatible, the empirical and fundamental specifications cannot be combined.*

General characteristics:

- These characteristics include the granularity, the percentage of voids (voids content), sensitivity to water (resistance to water) and resistance to permanent deformation (rutting test).

Additional characteristics:

- **Empirical characteristics :** mainly, the minimum bitumen content of the mix, expressed as a percentage of the total weight of the asphalt concrete, the nature of the binder and the scope of the grading envelopes on characteristic sieves.
- **Fundamental characteristics :** mainly, the modulus of rigidity and the resistance to fatigue. Resistance to permanent deformations can be characterized by replacing the rutting test with repeated compression tests (EN 12-697-25 [31]), which are not used in the common type testing.

The **fundamental** approach corresponds to the studies of level 3 or 4 formulations defined in 6.1.2. It applies essentially to materials fulfilling structural functions. It provides measured values for the design of base asphalt concretes, high-modulus asphalt concretes, high-modulus bituminous concretes and, less commonly, semi-coarse bituminous concretes.

Therefore,

- In most cases, the specifications of surface layers are based on the empirical approach (this is always the case for BBTM and EB-BBM).
- For the base layers, the fundamental approach is essential for EB-GB 4 and EB-EME 2 type materials. Both approaches are possible for EB-GB 3 type materials, but the fundamental approach is recommended.

Certain properties of asphalt mixes cannot be characterized by the empirical or the fundamental approach. For example, this is the case of the resistance to abrasion, impacts or polishing under the influence of the traffic. These properties are taken into consideration through the specifications relating to the characteristics of the aggregates. Furthermore, these approaches do not cover the manufacturing and usage methods.

6.1.2. Intrinsic properties of the mix – Type tests

In France, the characteristics of asphalt mixes are usually measured in laboratory type tests. Their purpose is to prove that the formula meets the specifications of the product standard. The tests are conducted in a laboratory, according to the procedures described in EN 13108-20 [32], and on materials that are representative of the building site.

The European approaches make the distinction between five levels of type tests, according to the nature of the material, its position in the pavement (wearing or binder course or the base course), the stress level and, finally, the contractual specifications. In France, these five levels are interpreted and applied as follows:

- level 0: consists of drawing up a gradation curve and determining a binder content. This level may be required for asphalt concretes in zones that are infrequently used (e.g., shoulders),
- level 1: this level is associated with the gyratory shear compactor tests and water sensitivity test.
- level 2: the level 1 tests, plus rutting tests,
- level 3: the level 2 tests, plus the modulus test (does not apply to EB-BBM and BBTM),
- level 4: the level 3 tests, plus the fatigue test.

Table 15 defines the formulation levels required for each type of pavement, the strain levels and the position of the asphalt concrete in the pavement (surface layers, base courses).

		NS1	NS2	NS3	NS4
Apron		*1*	*2 / 1*	*	*
Runway	*Common part*	*1*	*1 / 1*	*3 / 4*	*3 / 4*
	Runway turn pad	*1*	*2 / 3*	*3 / 4*	*3 / 4*
	Rapid exit	*1*	*2 / 1*	*3 / 4*	*3 / 4*
	Threshold	*1*	*2 / 1*	*3 / 4*	*3 / 4*
Taxiway and aircraft stand taxilane	*Common part*	*1*	*2 / 1*	*3 / 4*	*3 / 4*
	Connection	*1*	*2 / 3*	*3 / 4*	*3 / 4*
*Waiting area***		*1*	*2 / 1*	*2 / 4*	*2 + P / 4*

*** Cement concrete or grouted macadam***
***** See the definitions in the glossary***
P: test of resistance to static deformation
(direct tension-long load modulus test or punching test)

(i) Surface layers / *(i) Base layers*
i and j: level of the study

Table 15: levels of type tests according to the stress level

For the stress levels NS1 and NS2, the test is valid for a maximum of 5 years. For the NS3 and NS4 stress levels, it is only valid for 2 years. However, the validity can be extended, without exceeding 5 years, if the verification of the aptitude to compacting in the gyratory shear compactor test produces a result that is not more than 1.5 % different from the initial investigation, at the number of gyrations prescribed for the corresponding product.

The level 3 and 4 type tests require laboratory tests that identify the modulus and fatigue characteristics. Once the material meets the expected specifications, it is possible to integrate the values obtained in the laboratory into the design calculation, provided that these characteristics were actually obtained in the type tests on materials made from components from the site, with the prescribed percentage of voids, and without exceeding the normative values.

6.1.3. Mechanical performances for design purposes

The required mechanical characterizations apply to:

- the rigidity of the material, measured according to the value of the Young's modulus,
- the fatigue behavior.

This second aspect is not taken into consideration for the wearing course materials, which are subjected essentially to compression and shear.

Materials treated with bituminous binders are thermo-visco-elastic materials that, in the design, are assimilated to materials with an elastic behavior, of which the modulus is chosen according to the equivalent temperature and the speed *V* of movement of the load. This speed is associated with a loading frequency[9] equal to f (Hz) $= V$ (kph)/10. *Therefore,* $E = |E^*| = f(\theta_{eq}, f)$.

In practice, the value of $E^*(\theta_{eq}, f)$ is deduced from the value of the modulus measured in the laboratory at a temperature of 15°C and a frequency of 10 Hz (complex modulus test) or 0.02 s (indirect traction test). It is then corrected using the typical sensitivity curves that depend on the material family.

In the empirical approach, by default, the modulus at 15°C and 10 Hz (or 0.02 s) is taken to be equal to the lower value of the modulus range indicated in the material standard. In the fundamental approach, it can be determined by one of the following tests:

- the two-point bending test in Annex A of the standard EN 12697-26 [20], using the modulus at 15°C and 10 Hz,
- the MAER direct traction test, using the modulus calculated at 0.02 s and 15°C (Annex E of EN 12697-26 [20]),

The fatigue behavior is assessed using the two-point bending fatigue test (Annex A of EN 12697-24 [19]) at 10°C and 25 Hz. In this test (Figure 18), the load is applied to the head of the trapezoidal specimen embedded at the base by a sinusoidal movement of a constant amplitude, without any rest periods.

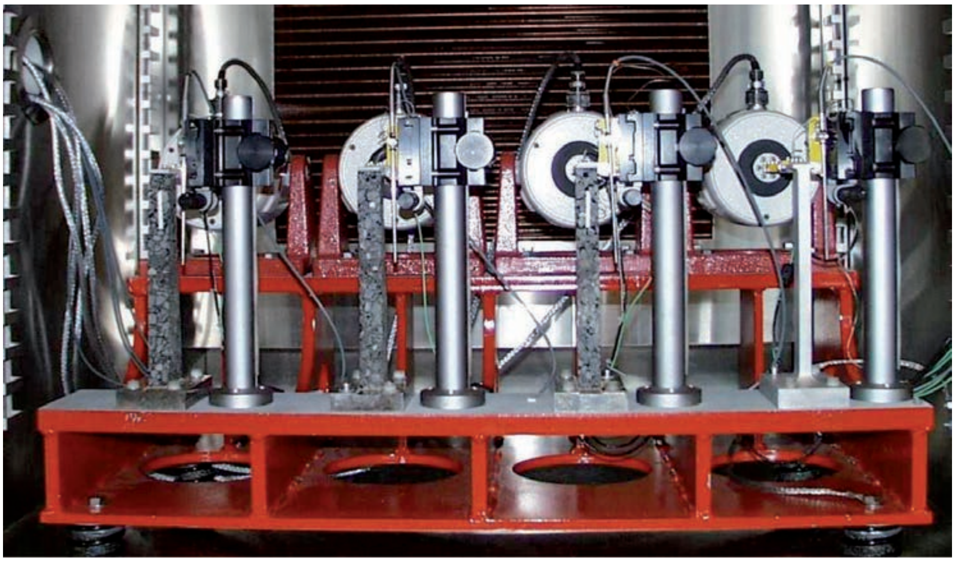

Figure 18: two-point bending test

[9] This formula differs from roads, which associate a frequency of 10 Hz with loads travelling at 70 kph, due to the thickness of the bonded materials, which is usually greater, and the footprints, which are usually larger.

The conventional failure corresponds to the number of cycles N for which the effort that must be applied is halved. The fatigue curve is represented by a relation of the following form:

$$\frac{\varepsilon_{t\,\max}}{\overline{\varepsilon}_6} = \left(\frac{N}{10^6}\right)^b$$

where $\overline{\varepsilon_6} = \varepsilon_6(10°\text{C},25\ \text{Hz})$: the value of the strain after 10^6 cycles, obtained in the fatigue test at a temperature of 10 °C and a frequency of 25 Hz.

The dispersion of the results is described by the standard deviation S_N of the variable log(N), upon failure.

6.1.4. Restrictions related to use

The thickness of the layers made of bituminous binders is variable, with a dispersion that is expressed by the standard deviation S_h.

On most sites, the value shown in Table 16 is chosen for the standard deviation S_h according to the thickness e of the bituminous materials in the base course.

e (cm)	$e \leq 10$	$10 < e < 15$	$e \geq 15$
S_h *(cm)*	*1*	*1 + 0.3 (e-10)*	*2.5*

Table 16: standard deviation of thicknesses when using layers made of bituminous materials

Note: *subject to the strict control of the geometric characteristics of the pavement foundation, a high bearing capacity and calibrated screw spreading, on certain sites, the dispersion of the thicknesses can be limited to 1.5 cm.*

6.2. Base asphalt concrete

Base asphalt concrete type materials are defined in the standard EN 13108-1 [33].

Class 1 EB-GB materials are not used in airfield pavements.

6.2.1. Use

The recommended thicknesses for EB-GB type materials defined in the GAN [3] are:

- 8 to 14 cm for EB14-GB, for a minimum thickness at all points of 6 cm,
- 10 to 16 cm for EB20-GB, for a minimum thickness at all points of 8 cm

6.2.2. Mechanical performances

The minimum performances required of base asphalt concrete are listed in Table 17.

	Conventional minimum values		Maximum values			
Class	Modulus (MPa) at 15°C and 10 Hz	ε_6 (µstrain) at 10°C and 25 Hz	Modulus (MPa) at 15°C and 10 Hz	ε_6 (µstrain) at 10°C and 25 Hz	Fatigue parameter β	S_N
2	9 000	80	11 000	90	5	0.3
3	9 000	90	11 000	100	5	0.3
4	11 000	100	14 000	115	5	0.3

Table 17: mechanical performances of base asphalt concrete

The sensitivity of the material to temperature is expressed by the evolution of its modulus.

In the absence of any laboratory measurements specific to the material in question, Table 18 gives an indication of the ratio $R(\theta,f) = \frac{E(\theta,f)}{E(15°C,10\ Hz)}$ for θ = -10, 0, 10, 20, 30 and 40°C and f = 10, 3 et 1 Hz, respectively corresponding to speeds of 100, 30 and 10 kph.

For frequencies and temperatures different from those shown in the table, this ratio R can be estimated by linear interpolations between two values.

Temperature θ (°C)		-10	0	10	20	30	40
R(θ,10 Hz)	EB-GB 2/3	2.53	2.03	1.32	0.68	0.30	0.11
	EB-GB 4	2.27	1.82	1.3	0.70	0.32	0.11
R(θ,3 Hz)	EB-GB 2/3	2.37	1.86	1.13	0.53	0.20	0.07
	EB-GB 4	2.13	1.66	1.11	0.54	0.22	0.07
R(θ,1 Hz)	EB-GB 2/3	2.23	1.71	0.98	0.42	0.14	0.04
	EB-GB 4	2.00	1.53	0.97	0.43	0.15	0.04

Table 18: sensitivity of the modulus of base asphalt concretes to variations in temperature and frequency

The graphs of the sensitivity of the modulus to temperature and frequency are shown in Annex E.

6.3. High-modulus asphalt concrete

High-modulus asphalt concretes are defined in the standard EN 13108-1 [33].

Class 1 EB-EME materials are not used in airfield pavements.

In zones exposed to long periods of severe cold, it is advisable to conduct a specific study to validate the use of EB-EME 2 materials, due to their high rigidity and the corresponding risks of cracking.

6.3.1. Use

The recommended thicknesses for EB-EME type materials defined in the GAN [3] are:

- 6 to 8 cm for EB10-EME, for a minimum thickness at all points of 5 cm,
- 7 to 13 cm for EB14-EME, for a minimum thickness at all points of 6 cm,
- 9 to 15 cm for EB20-EME, for a minimum thickness at all points of 8 cm.

6.3.2. Mechanical performances

The minimum performances required of high-modulus asphalt concrete are listed in Table 19.

	Conventional minimum values		Maximum values			
Class	Modulus (MPa) at 15°C and 10 Hz	ε_6 (µstrain) at 10°C and 25 Hz	Module (MPa) at 15°C and 10 Hz	ε_6 (µstrain) at 10°C and 25 Hz	Fatigue parameter β	S_N
2	14 000	130	17 000	145	5	0.25

Table 19: mechanical performances of high-modulus asphalt concretes

The sensitivity of the material to temperature is expressed by the evolution of its modulus.

In the absence of any laboratory measurements specific to the material in question, Table 20 gives an indication of the ratio $R(\theta,f) = \dfrac{E(\theta,f)}{E(15°\text{C},10\text{ Hz})}$ for θ = -10, 0, 10, 20, 30 and 40°C and f = 10, 3 et 1 Hz, respectively corresponding to speeds of 100, 30 and 10 kph.

For frequencies and temperatures different from those shown in the table, this ratio R can be estimated by linear interpolations between two values.

Temperature θ (°C)	-10	0	10	20	30	40
R(θ, 10 Hz)	2.14	1.71	1.21	0.79	0.43	0.21
R(θ, 3 Hz)	2.00	1.57	1.04	0.61	0.29	0.13
R(θ, 1 Hz)	1.89	1.44	0.90	0.49	0.20	0.08

Tableau 20: sensitivity of the modulus of high-modulus asphalt concretes to variations in temperature and frequency

The graphs of the sensitivity of the modulus to temperature and frequency are shown in Annex E.

6.4. Other asphalt concretes and standardized hot-mix asphalt concretes

This paragraph covers different hot-mix asphalt concrete techniques that can be used in the wearing and binder courses.

These products are defined in EN 13108-1 [33], apart from very thin asphalt concrete (BBTM)s, which is covered by EN 13108-2 [34].

6.4.1. Use

The nominal thicknesses for asphalt concretes and other hot-mix asphalt concretes defined in the GAN [3] are shown in Table 21.

Material	*Granularity*	*Average thickness (cm)*	*Minimum thickness at all points*(cm)*
Aeronautical asphalt concrete EB-BBA	*0/10C 0/14C 0/10D 0/14D*	*6 à 7 7 à 9 4 à 5 5 à 7*	*4 5 3 4*
Semi-coarse asphalt concrete EB-BBSG	*0/10 0/14*	*5 à 7 6 à 9*	*4 5*
Thin asphalt concrete EB-BBM	*0/10 0/14*	*3 à 4 3.5 à 5*	*2.5 3*
Very thin asphalt concrete BBTM	*0/6 et 0/10*	*2 à 3*	*1.5*
High-modulus asphalt concrete EB-BBME	*0/10 0/14*	*5 à 7 6 à 9*	*4 5*

**** this minimum thickness may require prior reshaping by milling or by adding materials, or a suitable thickness included in the corresponding ranges.***

Table 21: main composition characteristics of asphalt concretes

6.4.2. Mechanical performances

The minimum characteristics required of surface layer asphalt concretes are listed in Table 22.

		Conventional minimum values		*Maximum values*			
	Class	*Modulus (MPa) at 15°C and 10 Hz*	*$\mathcal{E}_6$ (µstrain) at 10°C and 25 Hz*	*Modulus (MPa) at 15°C and 10 Hz)*	*$\mathcal{E}_6$ (µstrain) at 10°C and 25 Hz*	*Fatigue parameter β*	*S_N*
EB-BBA	*1*	*5 500*	*130*	*9 000*	*145*	*5*	*0.25*
	2	*5 500*	*100*	*9 000*	*115*	*5*	*0.25*
	3	*7 000*	*100*	*11 000*	*115*	*5*	*0.25*
EB-BBSG	*1*	*5 500*	*100*	*9 000*	*115*	*5*	*0.25*
	2 or 3	*7 000*	*100*	*11 000*	*130*	*5*	*0.25*
EB-BBME	*1*	*9 000*	*100*	*11 000*	*115*	*5*	*0.25*
	2 or 3	*11 000*	*100*	*14 000*	*130*	*5*	*0.25*
EB-BBM	*all*	*5 500*	*Not applicable*				

Table 22: mechanical performances of asphalt concretes

The sensitivity of the material to temperature is expressed by the evolution of its modulus.

In the absence of any laboratory measurements specific to the material in question, Table 23 gives an indication of the ratio $R(\theta,f) = \dfrac{E(\theta,f)}{E(15°C,10\ Hz)}$ for θ = -10, 0, 10, 20, 30 and 40°C and f = 10, 3 et 1 Hz, espectively corresponding to speeds of 100, 30 and 10 kph.

For frequencies and temperatures different from those shown in the table, this ratio R can be estimated by linear interpolations between two values.

Temperature θ (°C)		-10	0	10	20	30	40
R(θ,10 Hz)	EB-BBSG 1 EB-BBA 1/2	2.69	2.18	1.33	0.67	0.24	0.18
	EB-BBSG 2/3 EB-BBA 3	2.29	1.93	1.33	0.67	0.26	0.14
	EB-BBME 1	1.92	1.71	1.33	0.67	0.33	0.21
	EB-BBME 2/3	1.77	1.65	1.33	0.67	0.35	0.21
R(θ,3 Hz)	EB-BBSG 1 EB-BBA 1/2	2.52	1.99	1.14	0.52	0.16	0.11
	EB-BBSG 2/3 EB-BBA 3	2.14	1.76	1.14	0.52	0.17	0.09
	EB-BBME 1	1.80	1.56	1.14	0.52	0.23	0.13
	EB-BBME 2/3	1.66	1.51	1.14	0.52	0.23	0.13
R(θ,1 Hz)	EB-BBSG 1 EB-BBA 1/2	2.37	1.84	0.99	0.41	0.11	0.07
	EB-BBSG 2/3 EB-BBA 3	2.01	1.63	0.99	0.41	0.15	0.08
	EB-BBME 1	1.69	1.44	0.99	0.41	0.16	0.08
	EB-BBME 2/3	1.56	1.39	0.99	0.41	0.16	0.08

Table23 : sensitivity of the modulus of various asphalt concretes to variations in temperature and frequency

Warning : The sensitivity of the EB-BBME material to temperature and frequency is deduced from that of the EB-BBSG material by homothety. This is an approximation, given that this characteristic is not listed for EB-BBME.

The graphs of the sensitivity of the modulus to temperature and frequency are shown in Annex E.

6.5. Warm-mix asphalt concrete

Warm-mix asphalt concretes are bituminous mixes produced at lower temperatures than hot-mix asphalt concretes. These temperatures are situated between 90 and 130°C. These products are covered by EN 13108 [17], along with the hot-mix products.

6.6. Grouted macadam

This technique consists of percolating a cement grout inside a very open asphalt concrete (void percentage of 15% to 25 %), resting on a support with a high modulus of rigidity.

Percolated asphalt concretes are industrial products that are not standardized.

6.7. Untreated graded aggregates

This paragraph deals with the materials to be used for sub-base courses, or even for base courses when applicable (ESWL<10 t and aircrafts from groups 1 and 2 only). EN 13-285 [18] groups together the various mixes of aggregates and water without binders under the term untreated graded aggregates.

6.7.1. Description in the standard

The standard EN 13-285 [18] refers to mixes of aggregates and water, without binders, with a granularity of 0/14 to 0/63 mm as untreated graded aggregates (GNT).

The standard makes the distinction between two types of untreated graded aggregates, according to the way in which they are prepared and certain characteristics:

- type A untreated graded aggregates are obtained in a single granular fraction, with an OPM degree of compaction equal to or higher than 80%, when the diameter of the largest aggregates (D) is lower than or equal to 31.5 mm,,
- type B untreated graded aggregates from at least two different granular fractions of defined proportions. They are recomposed, mixed and humidified in a plant (traditionally referred to as humidified graded aggregates).

6.7.2. Specifications of the components

The aggregates used in untreated graded aggregates should comply with the categories chosen in EN 13-242 [26].

The minimum required characteristics of the aggregates making up the untreated graded aggregates are listed in Table 24, according to the traffic classes:

Use	*Minimum characteristics, as per NF P 18-545 [25]*	*Traffic class*				
		CT1	*CT2*	*CT3*	*CT4*	*CT5*
Sub-base	*Granularity*	*0/20 – 0/31.5*	*0/20 – 0/31.5*	*0/20 – 0/31.5*	*0/20 – 0/31.5*	*0/20 – 0/31.5*
	Mechanical strength of the gravel	*Code E*	*Code E*	*Code D/C**	*Code C*	*Code C*
	Manufacturing characteristics of the gravel	*Code IV*	*Code IV*	*Code III*	*Code III*	*Code III*
	Manufacturing characteristic of the sands	*Code c*	*Code c*	*Code b*	*Code b*	*Code b*
	Angularity of the gravel	*Ang 4*	*Ang 4*	*Ang 3/ Ang 2 **	*Ang 2*	*Ang 2*

**** If group 4 or 5 aircraft use the pavement, with more than one movement per day, the following will be taken into consideration:***
- a code C for the mechanical strength of the gravel
- Ang 2 for the angularity of the gravel

Table 24: minimum characteristics of aggregates forming the untreated graded aggregates

6.7.3. Specifications of the mixes

The granularity of the untreated graded aggregates used in the sub-base corresponds to nominal values D of 20 or 31.5 mm.

Specification envelopes are defined for each of the values of D (class 2 untreated graded aggregates for 0/31.5 untreated graded aggregates and class 3 untreated graded aggregates for 0/20 untreated graded aggregates, as per EN 13-285 [18]).

If the untreated graded aggregates may be exposed to frost, it is necessary to guarantee:

- preferably, sensitivity to frost of the mix, verified in laboratory swell tests (NF P 98-234-2 [24]),
- the resistance to frost of the aggregates, as specified in NF P 18-545 [25].

6.7.4. Use

The untreated graded aggregates used in the sub-bases of airfield pavements are of the type B only. The degree of compaction of these untreated graded aggregates must meet a minimum OPM value of 82%, and their water content upon use is usually between w_{OPM}-1% and w_{OPM}+0.5%.

The other types of untreated graded aggregates (type A or type B, with an OPM degree of compaction equal to or higher than 80%) must only be used under shoulders or in the base layers of light aircraft pavements.

For satisfactory compacting, the minimum thickness of the untreated graded aggregates is 10 cm for a 0/20 and 15 cm for a 0/31.5. The thickness of the untreated graded aggregate compacted in a single layer must not exceed 35 cm.

6.7.5. Specifications of the mechanical performances

Granular materials have a non-linear behavior. The mechanical characteristics assigned to these layers in a design calculation must, therefore, depend on the structure (the thickness and rigidity of the bonded layers, the bearing capacity of the foundation).

The repeated load triaxial test described in EN 13286-7 [35] can be used to determine the mechanical performances of the untreated graded aggregates. This is currently the only test that can characterize the behavior of untreated graded aggregates in conditions that are close to those that exist in the pavement (specimens that are representative of the in situ characteristics, cyclic loading). It determines both the modulus of the untreated graded aggregates (quasi-reversible behavior) and their resistance to permanent deformation.

Tests performed on specimens with a water content representative of the least favorable hydrous conditions expected for the pavement layer can determine the values of E and v that are coherent with the level of stress present in the layer of untreated graded aggregate. This data is necessary to calculate the strains and stresses in the structure.

The Annex of EN 13286-7 [35] proposes a classification of untreated graded aggregates in three classes of mechanical performance (C1 to C3), based on two parameters taken from the test:

- a value of the characteristic modulus of elasticity E_c obtained for cyclical loads under an average stress p = 250 kPa, and a deviatoric stress q = 500 kPa,
- a value of characteristic permanent axial deformation ε_1^c obtained under a standardized load.

The untreated graded aggregates in the mechanical class C3 cannot be used in the structure of airfield pavement.

In the light of current knowledge, the design method adopts a highly simplified description of the behavior of these materials by using a linear elastic model. Therefore, in the absence of any results from triaxial tests, and pending a unified approach to the determination of the values to be assigned to the Young's modulus of the untreated graded aggregates, Table 25 shows the values that are used in practice.

Sub-base	*E_{GNT} {1} = k.$E_{pavement\ foundation}$ E_{GNT} {sub-layer i} = k.E_{GNT} {sub-layer (i-1)} k varies according to the category of the untreated graded aggregate*		
	Category	*1*	*2*
(untreated graded aggregate divided into 0.25 m thick sub-layers)	*k*	*3*	*2.5*
	E_{GNT} limited by: 600 MPa for a category 1 untreated graded aggregate 400 MPa for a category 1 untreated graded aggregate		

Table 25: mechanical characteristics of untreated graded aggregates

Note: *the variations in the values of the modulus over the total thickness of the untreated graded aggregate demonstrate their hardening non-linear character (the modulus increases with the intensity of the average stress) and the efficiency of the compacting that increases as the sub-base is laid.*

Note: *note that the distinction must be made between the untreated graded aggregate categories (1 or 2), with reference to the design parameters, and the untreated graded aggregate classes (1 to 6) defined in EN 13-285 [18], which are linked to the granularity of the mix.*

Category 1 untreated graded aggregates are used for traffic classes CT3, CT4 and CT5, and category 2 untreated graded aggregates are used for traffic classes CT1 and CT2.

Table 24 shows the minimum characteristics required of the aggregates making up the untreated graded aggregates, according to the traffic class.

7. Freeze-thaw verification

When the 0°C isotherm reaches the soil that is sensitive to frost, the surrounding water is pumped by cryosuction towards the frost front and forms pastels of ice that result in swelling during the frost and prevent the water from draining away when the frost thaws. In this state of quasi-saturation, the risk of a significant drop in the bearing capacity becomes serious and requires an investigation of the behavior of the pavement structure in freezing conditions.

This chapter defines the notions relating to the levels of protection and the characterization of the frost susceptibility of soil and then exposes the principle of the freeze-thaw verification. This verification must be considered as a fully-fledged step of the design process.

7.1. Protection levels

The rigor of a winter period against which a pavement structure must be protected is characterized by the atmospheric frost index *I*, which is based on two situations:

- the exceptional winter, which corresponds to the harshest winter between 1951 and 1997,
- and the unexceptionally harsh winter, defined as the harshest winter in a 10-year period between 1951 and 1997.

Note: *these values characterize the weather station, and are not always representative of an entire region. Therefore, in order to determine the frost index to be applied to a project that is distant from a major weather station, it is advisable to collect the data from the climatological stations closest to the project.*

The method used to calculate the frost index is described in NF P 98-080-1, Annex A [36].

There are three possible levels of protection against frost, which depend on the importance of the airfield and the function fulfilled by the pavement:

- **Total protection**, which is determined such that the depth of the frost corresponding to the exceptionally harsh winter cannot reach the frost-susceptible layers of the subgrade or the layers of the pavement, where appropriate.
- **High protection**, which is defined according to the same principle as total protection, except that the exceptionally harsh winter is replaced by the unexceptionally harsh winter.
- **Reduced protection**, which, under the conditions of the unexceptionally harsh winter, allows that:
  - ✓ lthe frost can slightly penetrate the frost-susceptible layers,
  - ✓ the traffic can be reduced during the thaw.

While not forgetting that each case should be considered individually, the distinctions made in Table 26 can act as a guide.

Annual passenger traffic on the airfield	*Level of protection*
more than 200,000	*total*
between 50,000 and 200,000	*high*
less than 50,000	*reduced*

Table 26: protection level against frost

Depending on the depth that the frost reaches and the chosen level of protection, the following measures should be taken:

- either to replace the frost-susceptible materials with materials that are not frost-susceptible in the layers that the frost reaches,
- or to increase the thickness of the upper layers that are not frost-susceptible.

7.2. Characterization of the frost-susceptibility of soil

The frost-susceptibility of materials can be characterized using the swelling test (NF P 98-234-2 [24]). This test, which is representative of the phenomenon, consists of measuring, at regular intervals, the elongation of a cylindrical test sample of the soil to be tested, the top surface of which is maintained at a negative temperature, while the base remains in distilled water.

Figure 19: test samples after swell tests

The swell test demonstrates the almost linear relation between the elongation of the test sample and the quantity of frost to which it is exposed, the value of which is, at all times, equal to the square root of the product of the negative temperature at the upper part of the test sample, by the elapsed time of exposure to the negative temperature.

This relation can be used to classify soils or granular materials according to the gradient p f their representative curve, expressed in *mm*/(°C.hour)^{½}. Consequently:

- materials are not frost-susceptible (SGn), when $p \leq 0.05$,
- materials are slightly frost-susceptible (SGn), when $0.05 < p \leq 0.40$,
- materials are very frost-susceptible (SGn), when $p > 0.40$.

7.3. The principle of the freeze-thaw verification

Since the atmospheric frost index I characterizes either the statistically proven exceptionally harsh winter or unexceptionally harsh winter for the site of the airfield, it is necessary to verify that the body of the pavement, which is designed to transmit only the permissible mechanical strains to the subgrade, also sufficiently protects the subgrade against frost.

To begin with, the method consists of translating this frost index into a quantity of frost Q_s that is transmitted to the surface of the pavement, then of verifying that the thermal protection provided by the body of the pavement only allows a quantity of frost Q_t that is lower than the permissible value Q_{PF} at this level, to be transmitted to the pavement foundation.

7.4. Frost-susceptibility of the pavement foundation

Since the body of the pavement must be made of materials that are not frost-susceptible, the soil under the level of the pavement foundation is cut into strata, from the frost-susceptible layer downwards, so that their susceptibility to frost can only increase as the depth increases. This amounts to saying that the frost-susceptibility of each layer of soil is equal to the most frost-susceptible layer above it, when this case occurs.

7.4.1. Permissible quantity of frost on the surface of the frost-susceptible layers situated under the pavement foundation.

In cases of **total or high protection**, the quantity of permissible frost Q_g on the surface of a frost-susceptible layer is zero.

The examination of the curves from numerous swell tests determined the permissible quantities of frost Q_g, corresponding to swelling limited to 5 mm, for **reduced protection** on the surface of a frost-susceptible layer. Expressed in (°C.hour)½ rather than (°C.day)½, these limits are, according to the frost-susceptibility of the frost-susceptible layer in question, considered to be equal to:

$$Q_g = \begin{cases} 4 \quad if \quad 0.05 < p \le 0.25 \\ 1/p \quad if \quad 0.25 < p \le 1 \\ 0 \quad if \quad p > 1 \end{cases} \qquad with\ p\ in\ mm/(°C.day)^{1/2}$$

The above formula is directly applicable when **a single frost-susceptible layer** is situated under the foundation (only one susceptibility is taken into consideration).

In cases of **reduced protection**, where **several frost-susceptible layers** with different thermal susceptibilities are positioned on top of one another, the following principle applies:

- if the first layer of frost-susceptible material under the foundation is **less than 20 cm thick**, the permissible quantity of frost on the surface of this layer is said to be possibly equal to:

$$Q_g = Q_{g2} + (Q_{g1} - Q_{g2}) \times \frac{h_1}{20}$$

where Q_{g1} = permissible quantity of frost of the first layer of frost-susceptible materials (upper layer),
Q_{g2} = permissible quantity of frost of the second layer of frost-susceptible materials (lower layer),
(Q_{g1} and Q_{g2} being defined according to the above rules),
h_1 = the thickness in centimeters of the upper layer of frost-susceptible materials.

- if the first layer of frost-susceptible material is **at least 20 cm thick**, the permissible quantity of frost on the surface of this layer is:

$$Q_g = Q_{g1}$$

7.4.2. Thermal protection provided by the non frost-susceptible materials situated under the pavement foundation.

The thermal protection Q_{ng}, provided by the non frost-susceptible materials in the capping layer and the subgrade situated above the first frost-susceptible layer of the latter is expressed by the formula:

$$Q_{ng} = \frac{A_n \times h_n^2}{h_n + 10}$$

where:

- h_n is the thickness in centimeters of the upper stratum of non frost-susceptible soil,
- A_n is a coefficient that depends on the nature of the material, the value of which is given in Table 27.

Materials	*Untreated*	*Treated*		
	Untreated graded aggregates and non-water sensitive material with a passing fraction at 80 mm ≤ 3%	*(C1) A1 – A2 – A3 treated with lime and hydraulic binder*	*(C1) B2 to B6 treated with hydraulic binder (with and without lime)*	*Fly ash*
A_n (°C.day)^{½}cm^{-1}	*0.12*	*0.14*	*0.13*	*0.17*

A – B – C – D: material classes defined in NF P 11-300
Untreated graded aggregates defined NF P 18-545 [25] and EN 13-285 [18]

Table 27: values of A_n according to the nature of the material in the capping layer

7.4.3. Quantity of permissible frost at the pavement foundation

The quantity of frost Q_{PF} deemed to be permissible at the pavement foundation is deduced from the preceding terms by the relation:

$$Q_{PF} = Q_{ng} + Q_g$$

7.5. thermal protection provided by the structure of the pavement

7.5.1. Quantity of frost transmitted to the surface of the pavement.

At moderate altitude, in low to average sunshine and with an atmospheric frost index I that does not exceed 210°C.day, and when the surface convection and radiation phenomena are taken into consideration, the quantity of frost Q_s transmitted to the surface of the pavement can be expressed by the formula:

$$Q_s = \sqrt{0.7 \times (I - 10)}$$

In all other cases (very harsh frost, high sunshine hours), a special investigation is necessary.

7.5.2. Quantity of frost transmitted to the pavement foundation

The quantity of frost Q_t transmitted to the foundation, protected by a pavement body measuring h, in thickness, can be determined in two ways:

- using a simplified method,
- using a one-dimensional heat propagation method (the Fourier model in two-phase media).

According to the simplified method,

$$Q_s = (1 + a \times h) \times Q_t + b \times h \quad \text{,whence} \quad Q_t = \frac{Q_s - b \times h}{1 + a \times h}$$

where *a* and *b* are characteristic coefficients of the materials making up the various layers of the body of the pavement, obtained from the expressions:

$h = \sum h_i$, where h_i is the thickness of the layer i in cm.

$a = \frac{1}{h}\sum(a_i \times h_i)$, $b = \frac{1}{h}\sum(b_i \times h_i)$, where a_i and b_i in (°C.day)^{½} cm^{-1} the values of which are shown in Table 28.

	a_i	b_i
Bituminous materials	***0.008***	***0.06***
Granular materials	***0.008***	***0.10***

Table 28: values of a_i and b_i en (°C.day)^{½}cm^{-1}.

7.6. Comparison de Q_t and Q_{PF}

The method consists in comparing the quantity of frost Q_t transmitted to the pavement foundation with the permissible quantity of frost Q_{PF} at the pavement foundation. The following condition must be met:

- for total protection: $Q_t(I_{\mathrm{HRE}})<Q_{ng}$
- for high protection: $Q_t(I_{\mathrm{HRNE}})<Q_{ng}$
- for reduced protection: $Q_t(I_{\mathrm{HRNE}})<Q_{ng}+Q_g$

Since approximations are made in the calculation formulae of the protection against frost in the simplified approach, the conclusions of the frost verification procedure presented here can only be applied if they show that the pavement structure defined according to only the mechanical design criteria provide sufficient thermal protection. If the conclusion does not allow a decision to be taken on this point, the one-dimensional heat propagation method must be used (use of a finite difference or finite element type software). The principle of this method is described in Annex J of NF P 98-086 [2]. If the calculation confirms that the thermal protection is insufficient, then the thicknesses of the non frost-susceptible materials must be adapted.

An example of a freeze-thaw verification can be found in Chapter 8.5.

8. Examples of applications

The purpose of this chapter is to illustrate the method described in this guide using a number of examples of calculations. This chapter covers three examples of the mechanical design of new flexible pavements, with varying input parameters (type of section, traffic, climate, etc.). It also includes an example of a freeze-thaw verification.

To begin with, an example of a calculation is described in detail in order to illustrate, step by step, the damage calculation described in paragraph 2.4. In practice, the sequence of these calculations can be reproduced using a dedicated software application.

8.1. Detailed example of a damage calculation

8.1.1. Description of the example

This example considers a predefined structure made up of **6 cm of EB-BBA 3 + 12 cm of EB-GB 3 + 47 cm of category 1 untreated graded aggregate (in terms of the design) on a PF2** (50 MPa) foundation. The selection of this structure corresponds to the pre-design phase. Depending on the results of the damage calculation for the traffic in question, iterative calculations of the thicknesses may be necessary in order to optimize the structure, if the total damage is significantly less than 1 (reduction of the thicknesses), or if the damage is greater than 1 (increase of the thicknesses).

In this example, we propose to calculate the total fatigue damage (for a 10-year design period) of an EB-GB 3 material. This calculation is based on the assessment of the tensile strains at the base of this material. The damage at the summit of the soil is calculated according to the same principle, while considering the vertical strains.

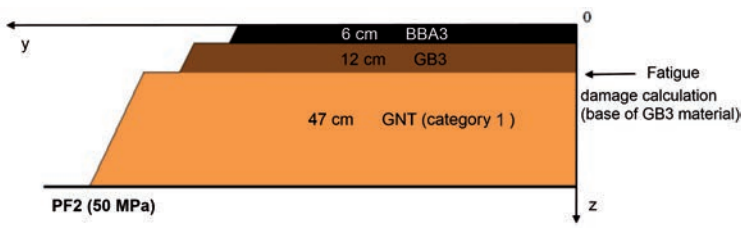

Figure 20 : cross-section of the considered pavement structure

The traffic is made up of an Airbus A330-200, with one takeoff per day, i.e., one movement per day with an Mrw = 233.9 t. The structure in question is a moderate-speed section, which means that V = 30 kph for the calculation, corresponding to a loading frequency of 3 Hz, and the standard deviation of the lateral wander law S_{bal} = 0,5 m. The selected design risk is 2.5%, because, according to the GAN [3], the aircraft belongs to group 5, and the associated traffic class is CT4. The equivalent temperature is 15°C.

8.1.2. Associated data

In accordance with the rules defined in Chapter 6 (Table 25), the untreated graded aggregate is divided into two layers: one measuring 25 cm, with a modulus taken to equal E_{GNT}\{1\} = 150 MPa (corresponding to 3xE_{PF}, i.e. 3x50) and the other 22 cm, with a modulus taken to equal E_{GNT}\{2\} = 450 MPa (corresponding to 3xE_{GNT}\{1\} i.e. 3x150).

The characteristics of the bituminous materials required to make the calculation are shown in Tables 17, 18, 22 and 23 in Chapter 6. Table 29 below shows modulus values for the chosen design parameters.

Materials	*Modulus at 15°C/10 Hz*	*R(15°C, 3 Hz)*	*Modulus at 15°C/3 Hz*
BBA3	***7000***	***0,83***	***5810***
GB3	***9000***	***0,83***	***7470***

Table 29 : Modulus values of bituminous materials.

8.1.3. Preliminary calculations

8.1.3.1. Coefficient K, related to the fatigue criterion of the EB-GB 3 material

For bituminous materials, coefficient K is written as follows:

$$K = 10^{6/\beta} k_{\theta f} k_s k_r k_c \,\overline{\varepsilon}_6$$

The standard values associated with the EB-GB 3 lead us to take:

- $\beta = 5$ and $\overline{\varepsilon}_6 = 90\,\mu strain$
- $k_{\theta f} = \sqrt{\dfrac{E(10°C,10Hz)}{E(\theta_{eq},f)}} = \sqrt{\dfrac{E(10°C,10Hz)}{E(15°C,3Hz)}} = \sqrt{\dfrac{\mathrm{R}(10°C,10Hz) \times E(15°C,10Hz)}{\mathrm{R}(15°C,3Hz) \times E(15°C,10Hz)}} = \sqrt{\dfrac{\mathrm{R}(10°C,10Hz)}{\mathrm{R}(15°C,3Hz)}}$

The values of R are shown in Table 18, which means that $k_{\theta f} = \sqrt{\dfrac{1.32}{0.83}} = 1.26$

The modulus of the layer of unbonded materials under the EB-GB 3 layer is higher than 120 MPa, so the coefficien k_s = 1 (Table 3).

- For the calculation of k_r:
  - a risk of 2,5 % corresponds to the value u = -1.960.
  - $b = -1/\beta = -0.2$
  - $S_N = 0.3$ for EB-GB 3
  - $S_h = 0.016m$ for 12 cm of EG-GB 3
  - $c = 2m^{-1}$

$$\delta = \sqrt{S_N^{\,2} + \left(\frac{cS_h}{b}\right)^2} = 0.340 \qquad k_r = 10^{-ub\delta} = 0.736$$

- For the calculation of k_c:

By definition, the coefficient kc is a function of traffic agressiveness, itself characterized by the ESWL (see definition of ESWL §2.6). However, the ESWL value results from a damage computation for a specific kc value previously fixed. The determination of kc thus requires to implement the iterative process as follows:

1) a damage computation is performed for a kc value (initial value equal to 1.3 for instance),

2) the ESWL value corresponding to this damage is computed,

3) the updated kc value is determined using the kc=f(ESWL) relationship (figure 21).

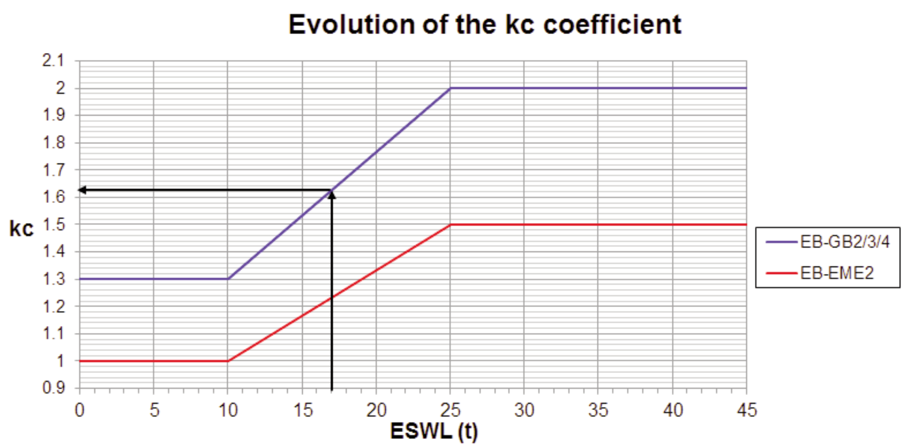

Figure 21: determination of the coefficient k_c on the basis of the ESWL

These steps are repeated with this updated value of kc, and the process is lead until the kc value converges.

For this example, the computation details are not presented, the ESWL value being determined from a software implementing the prinicples described in this guidance. A value of ESWL = 17.06 t is finally obtained. The use of figure 21 allows determining the kc value, equal to 1.63 in this case.

Therefore : $K = 10^{6/5} \times 1.26 \times 1 \times 0.736 \times 1.63 \times 90 = 2\,156$

8.1.3.2. Fields of deformation at the base of the base asphalt concrete layer

The assessment of the fatigue damage in the base asphalt concrete layer starts with the calculation of the fields of strains (reversible) ε_{xx}, ε_{yy}, ε_{xy} et $\varepsilon_{t\ max}$ at the base of this layer (z = 0.18 m), when the aircraft in question passes over it. This calculation is made by applying Burmister's model.

As stated in paragraph 2.4, a grid of points is used (x_i, y_j) where x = the longitudinal axis, y = the transversal axis of the runway, located at z = 0.18 m which allows for the discretion of the calculations. Supposing that the aircraft is centered on the axis y, the calculations can generally be made by symmetry on the half-grid $y \geq 0$[10].

Figure 22 shows the configuration of the landing gear of an Airbus A330-200 centered on the axis of the runway, and the landing gear considered for the calculation.

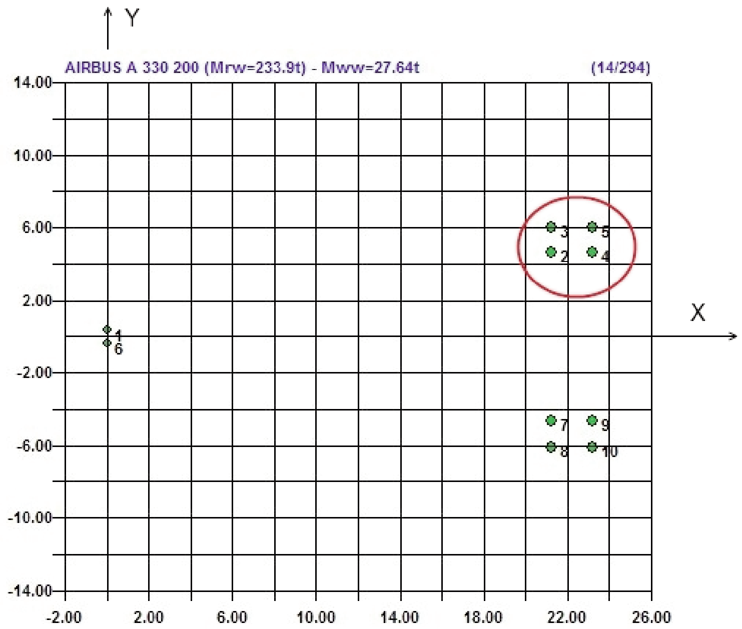

Figure 22: configuration of the landing gear of an Airbus A330-200 and selection of the landing gear for calculation without lateral wander

[10] The interactions between symmetrical landing gears can sometimes be neglected (distant landing gears) or taken into consideration using ad hoc calculation techniques with symmetry.

The strain maps ε_{xx}, ε_{yy}, ε_{xy} and the maximum tensile strain $\varepsilon_{t\ max}$[11] obtained at the base of the base asphalt concrete, are shown in Figure 23.

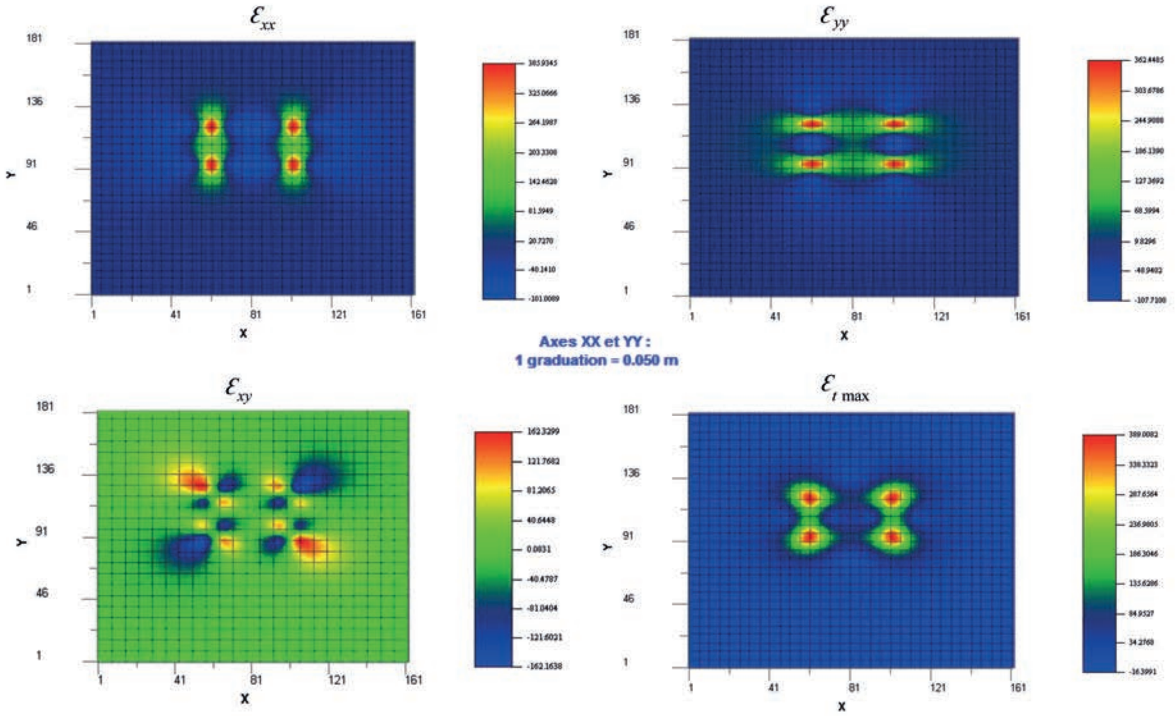

Figure 23 : maps of the strains ε_{xx}, ε_{yy}, ε_{xy} et $\varepsilon_{t\ max}$ at the base of the base asphalt concrete

Note : *the coordinates in these figures are expressed according to a local system determined by the calculation grid.*

In this case, since the strain basin do not extend as far as the axis of the runway (longitudinal axis), the interaction between the right and left landing gear of the aircraft is negligible.

[11] $$\varepsilon_{t\max} = \frac{\varepsilon_{xx} + \varepsilon_{yy} + \sqrt{(\varepsilon_{xx} - \varepsilon_{yy})^2 + 4\varepsilon_{xy}^{\ 2}}}{2}$$

8.1.4. Calculation of the transversal damage profile without lateral wander

Initially, the transversal damage profile $\Delta D(y, z_k)$ associated with each passage of an aircraft is calculated without taking the lateral wander phenomenon into consideration.

In this case, the strain $\varepsilon_{t\,max}$ map obtained previously can be used directly as input for this calculation.

Here, we can illustrate the damage calculation for a point of the transversal profile located at a position y_j = 4.65 m, which corresponds to the longitudinal axis of the inner wheels of the main landing gear (shown by the red line in Figure 24). Note that the same operation must be repeated at every point of the calculation grid in the y axis.

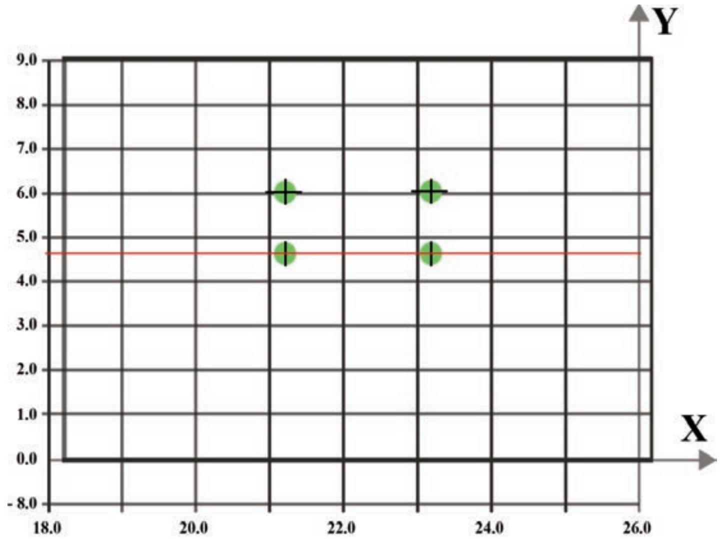

Figure 24: identification of the transversal position (red line) for the calculation of damage without lateral wander

In order to consider the history of the strain at a point of the calculation grid where y_j = 4.65 m, it is necessary to know the longitudinal profile of the strains in order to continuously integrate Miner's law (simplified formula for the manual calculation).

The considered longitudinal profile (y_j = 4.65 m and z_k = 0.18 m), taken from the strain map $\varepsilon_{t\,max}$, is shown in Figure 25.

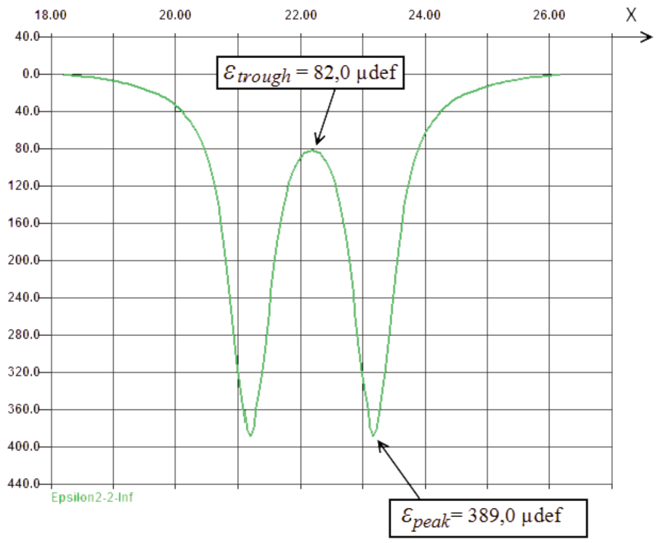

Figure 25: traction strain history at the base of the base asphalt concrete with y_j = 4.65m.

The damage caused by one passage of the A330-200 is then calculated on the basis of the integral expression of Miner's postulate. This calculation can be made numerically or analytically, by taking account of the values at the peaks and the troughs of the deformation signal (see paragraph 2.5.3). The result is:

$$\Delta D(y_j = 4.65\,m, z_k = 0.18\,m) = \frac{1}{K^\beta}({\varepsilon_{peak}}^\beta - {\varepsilon_{trough}}^\beta + {\varepsilon_{peak}}^\beta) = \frac{1}{2156^5}(389{,}0^5 - 82{,}0^5 + 389{,}0^5) = 3{,}822 \times 10^{-4}$$

The cumulative damage for the entire traffic is deduced by applying Miner's accumulation law. Since the traffic is made up of a single aircraft, the cumulative damage equals the value of the preceding damage, multiplied by the total number of movements in the design period:

$$\underline{\Delta D_{cumulated}(y_j = 4.65\,m) = \Delta D(y_j = 4.65\,m) \times 1\,mvts\,/\,day \times 365\,days \times 10\,years = 1.44}$$

The results of these calculations for each transversal position y_j of the calculation grid are shown in Table 30, and the cumulative transversal damage profile without lateral wander is shown in Figure 26.

y_j (m)	ε_{peak} (µdef)	ε_{trough} (µdef)	ΔD	$\Delta D_{cumulated}$
4	100.9	3.6	4.49E-07	0.00
4.05	115.5	10.8	8.83E-07	0.00
4.1	132.8	17.4	1.77E-06	0.01
4.15	152.7	24.1	3.56E-06	0.01
4.2	175.6	31.8	7.17E-06	0.03
4.25	202.3	41.2	1.45E-05	0.05
4.3	233.1	50.2	2.95E-05	0.11
4.35	267.4	56.5	5.87E-05	0.21
4.4	302.1	62.1	1.08E-04	0.39
4.45	333.1	68.4	1.76E-04	0.64
4.5	357.3	74.1	2.50E-04	0.91
4.55	374.6	78.3	3.17E-04	1.16
4.6	385.4	81.2	3.65E-04	1.33
4.65	389.0	82.0	3.82E-04	1.40
4.7	385.7	81.3	3.66E-04	1.34
4.75	376.5	79.9	3.25E-04	1.19
4.8	361.3	77.4	2.64E-04	0.96
4.85	339.5	73.9	1.93E-04	0.71
4.9	311.9	70.0	1.27E-04	0.46
4.95	281.8	65.2	7.62E-05	0.28
5	252.9	59.3	4.44E-05	0.16
5.05	227.8	54.3	2.63E-05	0.10
5.1	207.2	50.4	1.64E-05	0.06
5.15	191.1	46.3	1.09E-05	0.04
5.2	179.0	42.7	7.88E-06	0.03
5.25	170.8	40.3	6.23E-06	0.02
5.3	166.2	38.9	5.44E-06	0.02
5.35	165.0	38.2	5.25E-06	0.02

y_j (m)	ε_{peak} (µdef)	ε_{trough} (µdef)	ΔD	$\Delta D_{cumulated}$
5.4	167.3	39.0	5.62E-06	0.02
5.45	173.0	41.4	6.66E-06	0.02
5.5	182.5	44.3	8.68E-06	0.03
5.55	195.8	47.3	1.23E-05	0.05
5.6	213.4	50.9	1.90E-05	0.07
5.65	235.5	55.9	3.11E-05	0.11
5.7	261.9	61.9	5.29E-05	0.19
5.75	291.5	67.3	9.04E-05	0.33
5.8	321.3	71.3	1.47E-04	0.54
5.85	347.3	74.6	2.17E-04	0.79
5.9	367.0	77.8	2.86E-04	1.04
5.95	380.2	81.0	3.41E-04	1.24
6	387.5	83.0	3.75E-04	1.37
6.05	388.6	82.4	3.80E-04	1.39
6.1	382.7	79.8	3.52E-04	1.29
6.15	370.0	76.6	2.98E-04	1.09
6.2	350.5	72.6	2.27E-04	0.83
6.25	324.0	67.5	1.53E-04	0.56
6.3	291.4	61.6	9.02E-05	0.33
6.35	256.4	54.5	4.76E-05	0.17
6.4	223.1	46.3	2.37E-05	0.09
6.45	193.6	38.3	1.17E-05	0.04
6.5	168.1	30.7	5.77E-06	0.02
6.55	146.3	22.9	2.87E-06	0.01
6.6	127.2	15.5	1.43E-06	0.01
6.65	110.9	8.4	7.19E-07	0.00
6.7	97.1	0.9	3.70E-07	0.00
-	-	-	-	-

Table 30: values of cumulative damage along a transversal profile

Note : *the damage profile above was produced using calculation software that achieves greater precision, which is why there is a slight difference with the values calculated « by hand ».*

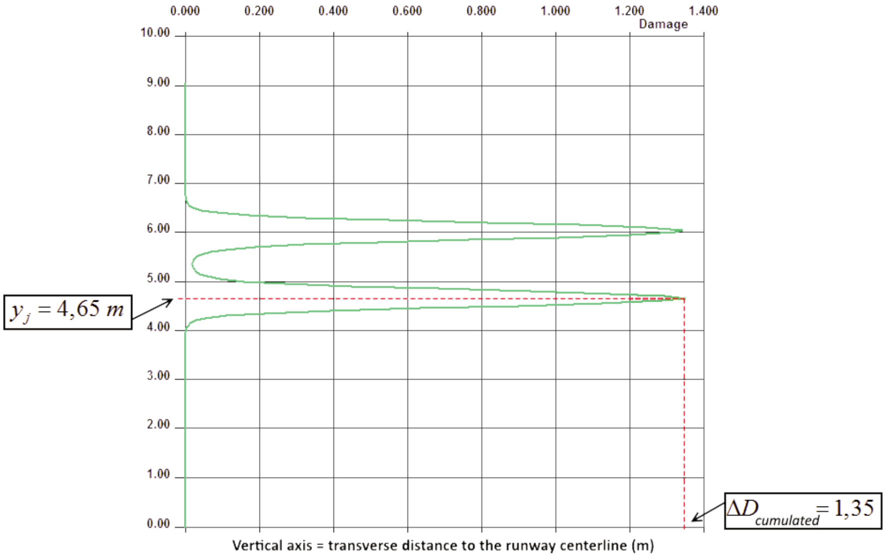

Figure 26 : transversal cumulative damage profile without lateral wander

8.1.5. Calculation of the transversal damage profile with lateral wander

The damage profile with lateral wander is again based on Miner's postulate on cumulative damage. It can be deduced from the damage profile without lateral wander (symmetrized[12]) and from the lateral wander law of the aircraft in question. We will illustrate the calculation for a point located at a distance y_j = 5.35 m from the longitudinal axis, corresponding to the centre-to-centre of the bogie, when the aircraft is centered on the runway (Figure 27).

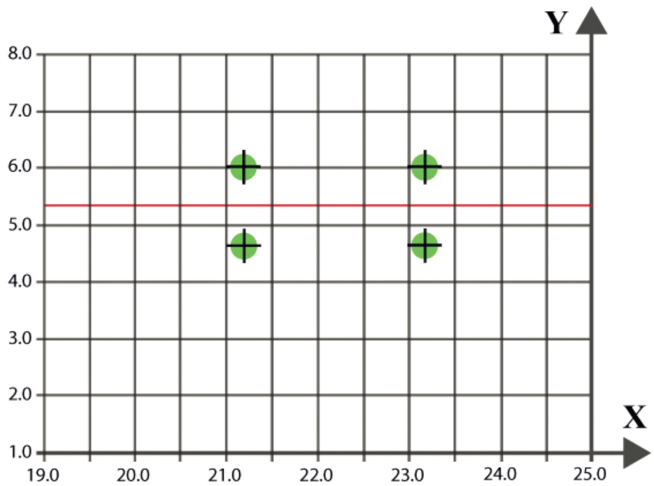

Figure 27 : identification of the transversal position (red line) for the calculation of damage with lateral wander

[12] This profile takes account of the presence of two landing gears and is obtained by symmetrisation relative to the axis x of the profile calculated for one landing gear

The lateral distribution of the traffic (according to *y*) is represented by a normal distribution, with discrete trajectories $(y_j)_b$ for j = 1,..., n_b according to a pitch Δy = *5 cm*. Figure 28 illustrates this distribution and the position of a landing gear when the aircraft is centered, on the one hand, and when it is on an off-centre trajectory

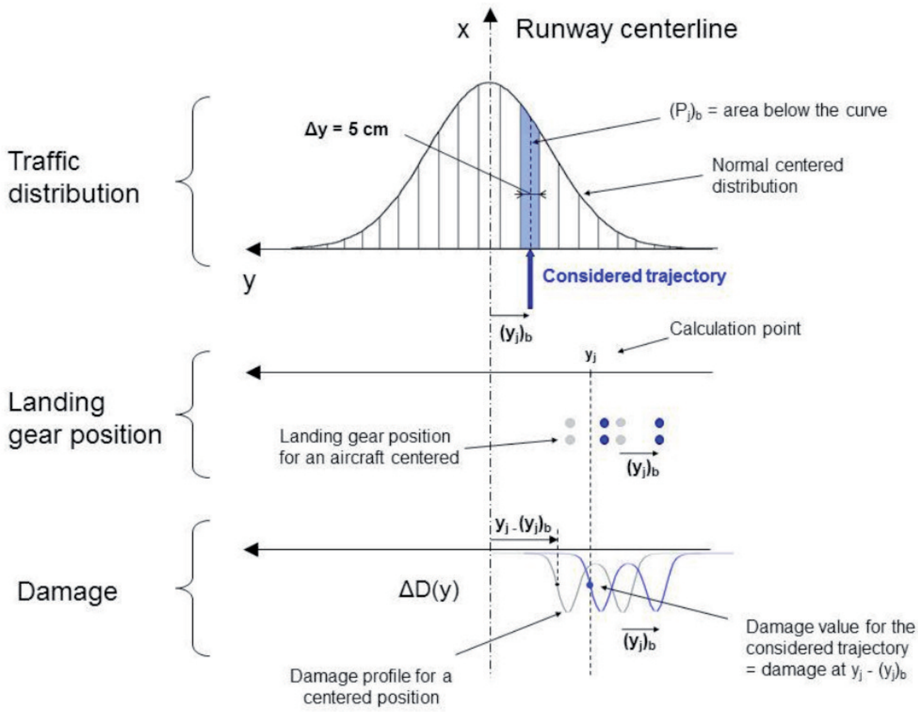

Figure 28 : diagram explaining the lateral dispersion of traffic

$(y_j)_b$ on the other. The corresponding transversal damage profiles are also represented.

To calculate the damage at one point y_j (y_j = 5.35 m in this case), the sum total of the damage (as calculated at this point) caused by an aircraft traveling on each of the trajectories of the distribution must be taken into consideration. To do this, the damage profile without lateral wander must be laterally translated, as shown in Figure 28. For one trajectory $(y_j)_b$, this amounts to taking the value of the damage in y_j - $(y_j)_b$.

The damage for each trajectory is then multiplied by a coefficient $(P_j)_b$ representing the proportion of the traffic on this trajectory (its value is equal to the area under the curve shown in blue). Therefore, for a Gaussian lateral wander law (see Annex B) :

$$(P_j)_b = \frac{1}{\sqrt{2\pi}} \int_{y_1}^{y_2} e^{-y^2/2} dy \quad \text{where } y_1 = \frac{(y_j)_b - \Delta y/2}{S_{bal}} \quad \text{and} \quad y_2 = \frac{(y_j)_b + \Delta y/2}{S_{bal}}$$

The number of trajectories of the discretion is chosen (for this manual calculation) such that, beyond this number, the corresponding weighted damage become negligible in relation to the maximum damage (<2%).

Table 31 shows the results of the calculation with n_b = 41. For each trajectory $(y_j)_b$, the translated value of the damage $\Delta D(y_j - (y_j)_b)$ is calculated, then weighted by $(P_j)_b$.

$(y_j)_b$ (m)	$y_j - (y_j)_b$ (m)	$\Delta D\ (y_j - (y_j)_b)$	$(P_j)_b$	$\Delta D\ (y_j - (y_j)_b) \times (P_j)_b$
1	4.35	5.87E-05	0.54%	3.17E-07
0.95	4.4	1.08E-04	0.66%	7.10E-07
0.9	4.45	1.76E-04	0.79%	1.39E-06
0.85	4.5	2.50E-04	0.94%	2.35E-06
0.8	4.55	3.17E-04	1.11%	3.51E-06
0.75	4.6	3.65E-04	1.30%	4.73E-06
0.7	4.65	3.82E-04	1.50%	5.73E-06
0.65	4.7	3.66E-04	1.71%	6.28E-06
0.6	4.75	3.25E-04	1.94%	6.31E-06
0.55	4.8	2.64E-04	2.18%	5.75E-06
0.5	4.85	1.93E-04	2.42%	4.68E-06
0.45	4.9	1.27E-04	2.66%	3.37E-06
0.4	4.95	7.62E-05	2.90%	2.21E-06
0.35	5	4.44E-05	3.12%	1.39E-06
0.3	5.05	2.63E-05	3.33%	8.77E-07
0.25	5.1	1.64E-05	3.52%	5.77E-07
0.2	5.15	1.09E-05	3.68%	4.02E-07
0.15	5.2	7.88E-06	3.81%	3.01E-07
0.1	5.25	6.23E-06	3.91%	2.44E-07
0.05	5.3	5.44E-06	3.97%	2.16E-07
0	5.35	5.25E-06	3.99%	2.09E-07
-0,05	5,4	5,62E-06	3,97%	2,23E-07
-0.1	5.45	6.66E-06	3.91%	2.60E-07
-0.15	5.5	8.68E-06	3.81%	3.31E-07
-0.2	5.55	1.23E-05	3.68%	4.55E-07
-0.25	5.6	1.90E-05	3.52%	6.68E-07
-0.3	5.65	3.11E-05	3.33%	1.03E-06

Table 31: values of weighted damage for each path

$(y_j)_b$ (m)	y_j - $(y_j)_b$ (m)	ΔD (y_j - $(y_j)_b$)	$(P_j)_b$	ΔD (y_j - $(y_j)_b$)x$(P_j)_b$
-0.35	5.7	5.29E-05	3.12%	1.65E-06
-0.4	5.75	9.04E-05	2.90%	2.62E-06
-0.45	5.8	1.47E-04	2.66%	3.91E-06
-0.5	5.85	2.17E-04	2,42%	5.24E-06
-0.55	5.9	2.86E-04	2.18%	6.22E-06
-0.6	5.95	3.41E-04	1.94%	6.62E-06
-0.65	6	3.75E-04	1.71%	6.42E-06
-0.7	6.05	3.80E-04	1.50%	5.69E-06
-0.75	6.1	3.52E-04	1.30%	4.57E-06
-0.8	6.15	2.98E-04	1.11%	3.30E-06
-0.85	6.2	2.27E-04	0.94%	2.14E-06
-0.9	6v25	1.53E-04	0.79%	1.21E-06
-0.95	6.3	9.02E-05	0.66%	5.93E-07
-1	6.35	4.76E-05	0.54%	2.57E-07
			ΔD_{bal} =	1.05E-04

Table 31: values of weighted damage for each path

The value of the damage with lateral wander for one passage of an aircraft at one transversal position y_j = 5.35 m is, therefore, the sum total of all the weighted damage (the last column in Table 31):

$$\Delta D_{bal}(y_j = 5,35\,m) = \sum_{b=1}^{41} \Delta D(y_j - (y_j)_b) \times (P_j)_b = 1,05.10^{-4}$$

Finally, in order to obtain the **cumulative damage with lateral wander** for the traffic in question at the transversal position y_j = 5.35 m, ΔD_{bal} must be multiplied by the cumulative traffic in the design period:

$$D_{bal,cumulated}(y_j = 5.35\,m) = \sum_{aircraft} N_{aircraft} \Delta D_{bal,aircraft} = \Delta D_{bal,A330-200}(y_j = 5.35\,m) \times 1\,mvt/d \times 365\,d \times 10\,years = 0.383$$

This calculation must be repeated for each transversal position in the calculation grid in order to obtain the profile of cumulative damage with lateral wander. Finally, the maximum value of cumulative damage with lateral wander along this profile is the decisive value for the fatigue criterion of the asphalt concretes. As Figure 29 shows, this maximum value is not reached for the transversal position y_j = 5.35 m, but for the position situated directly under the wheels of the main landing gear.

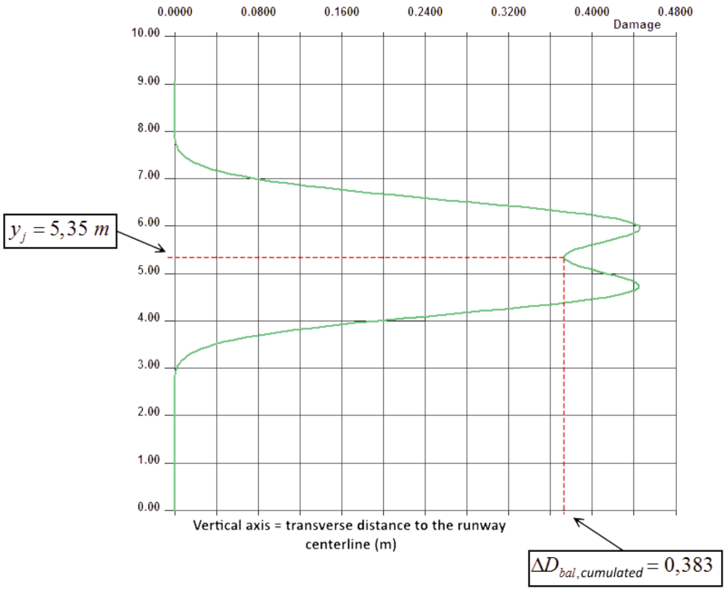

Figure 29 : transversal damage profile with lateral wander

The rest of this chapter illustrates three mechanical designs and a freeze-thaw verification. This time, the goal is to demonstrate the application of the method itself.

8.2. Example of the design of a taxiway

8.2.1. Data

8.2.1.1. Traffic

◆ **Traffic characteristics**

The information on the aircrafts likely to use the airfield pavement to be designed is shown in Table 32. The distinction is made between takeoffs and landings. The weight and the number of movements are shown for each of them. Note that one movement represents either a takeoff or a landing.

Aircraft in the project traffic	*Mrw (t)**	*Mlw (t)**	*Takeoffs per year*	*Landings per year*	*Cumulative traffic per type of aircraft (for 10 years)*
HERCULES C 130	*70*	*70*	*125*	*125*	*2 500*
AIRBUS A310-300	*150*	*150*	*25*	*25*	*500*
AIRBUS A330-200	*233*	*182*	*25*	*25*	*500*
AIRBUS A340-300	*257*	*192*	*25*	*25*	*500*
ANTONOV AN-124	*400*	*330*	*200*	*200*	*4 000*
BOEING B747-200C	*340*	*285.7*	*50*	*50*	*1 000*
ILYUSHIN IL 76 TD	*190*	*151*	*50*	*50*	*1 000*

** since the data are taken from actual traffic, the weights are different, and more precise, that those in the « Ficav »database*

Table 32 : traffic taken into consideration for design purposes (1)

The **design period is 10 years**. Growth in traffic in this period is assumed to be zero. In this case, it is possible to calculate the cumulative number of passages of each aircraft in the design period. These results are shown in the « Cumulative traffic per type of aircraft (for 10 years) » column in Table 32.

The geometry and the loading conditions of the landing gear of each aircraft are defined in the STAC's « Ficav » database.

The taxiway to be designed is considered as a moderate-speed section. Therefore, the **speed of movement** of all the aircraft used in the calculation is **30 kph** (corresponding to a frequency of 3 Hz) and the **lateral wander** of each aircraft is characterized by a standard deviation of **0.5 m** (see paragraph 3.1.3 in the guide).

◆ Determination of the traffic class « CTi »

The method used to determine the traffic class (see glossary, Table 53) is defined in the GAN [3]. The application of this method to the selected traffic resulted in Table 33.

Aircraft in the project traffic	*Group*	*F (mvts/d)*	*Traffic class*
HERCULES C 130	*2*	*0.68*	*CT2*
AIRBUS A310-300	*4*	*0.14*	*CT3*
AIRBUS A330-200	*5*	*0.14*	*CT4*
AIRBUS A340-300	*5*	*0.14*	*CT4*
ANTONOV AN-124	*5*	*1.10*	*CT4*
BOEING B747-200C	*5*	*0.27*	*CT4*
ILYUSHIN IL 76 TD	*4*	*0.27*	*CT3*

Table 33: determination of the traffic class by type of aircraft (1)

The highest traffic class is applied to the project. In this example, it is **traffic class CT4**.

8.2.1.2. Parameters dependent on the contracting authority

A **design risk of 2.5%** is applied, because the annual traffic class is higher than CT3 (see recommendation in paragraph 3.1.1 of the guide).

8.2.1.3. Surface layer

The choice of the surfacing (the method is described in detail in the GAN [3]) depends on the stress level, which is defined according to the traffic class and the type of climate (see glossary, Table 54).

◆ Determination of the stress level « NSi »

The traffic class defined above is class CT4.

The **type 3 climate**, which is predominantly continental, has been chosen for design purposes (selected equivalent temperature: θ_{eq} = 15°C).

- Therefore, the **stress level is NS3**.

◆ Choice of the surfacing

The GAN [3] recommends two types of wearing course products for the stress level NS3 and for common taxiways: either EB-BBA 2 or EB-BBME 1 (see Table 12 in this guide).

The three products proposed for the binder course (see Table 13) are EB-BBM 2, EB-BBSG 1 or EB-BBME 1.

For the rest of this study, the decision was taken to opt for a single **wearing course made of EB-BBA 2** (no binder course). The thickness of this layer is 6 cm.

8.2.1.4. Base course

For a stress level of NS3 and for common taxiways, the GAN [3] recommends the use of an EB-GB 3 product as a base course (EB-EME 1 is not used for airfield pavements). In this example, the material chosen for the **base course** is an **EB-GB 3**.

8.2.1.5. Sub-base

The decision was taken to use a **sub-base** made up of **category 1 untreated graded aggregate** (in terms of the design).

8.2.1.6. Pavement foundation

The target bearing capacity of the project is **PF2**. To determine the strains and stresses in the body of the pavement in the calculation model, the modulus associated with the pavement foundation is the lower limit of the class, i.e. 50 MPa.

8.2.1.7. Mechanical characteristics of the materials

The mechanical characteristics of the bonded materials chosen for this example are shown in Table 34. These are the conventional minimum values. The values of the moduli at different temperatures and frequencies are deduced from Tables 18 and 23 (Chapter 6), showing the susceptibility of the moduli to variations in temperature and frequency.

Product	E (15°C, 10Hz) (MPa)	E (10°C, 10Hz) (MPa)	E (15°C, 3Hz) (MPa)	ε_6 (10°C, 25Hz) (µstrain)	$\beta = -1/b$	S_N	S_h
EB-BBA 2	5 500	7315	4 512	100	5	0.25	-
EB-GB 3	9 000	11 880	7 383	90	5	0.30	(1)

(1) S_h = 1 if e ≤ 10 cm, S_h = 1+0.3 (e-10) if 10 cm < e < 15 cm, S_h = 2.5 if e ≥ 15 cm

Table 34: mechanical characteristics of bituminous materials (1)

The untreated graded aggregate is divided (for design purposes) into 0.25 m thick sub-layers. The modulus of the category 1 untreated graded aggregate is equal to three times the modulus of the underlying layer, up to a maximum limit of 600 MPa. Therefore, the modulus of the first sub-layer will equal 50 MPa x 3 = 150 MPa, the modulus of the second layer will be 150 MPa x 3 = 450 MPa, and the modulus of the following layers will be 600 MPa.

8.2.2. Calculation of the damage

The damage is calculated at two locations in the structure of the pavement:

- at the base of the base course (EB-GB 3) – traction fatigue damage (criterion ε_t),
- at the summit of the subgrade – compression damage by permanent deformation (criterion ε_{zz}).

Three calculations with different thicknesses of the untreated graded aggregate are presented in order to illustrate the iterations applied to the thickness of the sub-base:

- Structure 1 – 6 cm of EB-BBA 2 + 11 cm of EB-GB 3 + 22 cm of category 1 untreated graded aggregate,
- Structure 2 – 6 cm of EB-BBA 2 + 11 cm of EB-GB 3 + 40 cm of category 1 untreated graded aggregate,
- Structure 3 – 6 cm of EB-BBA 2 + 11 cm of EB-GB 3 + 36 cm of category 1 untreated graded aggregate.

8.2.2.1. Damage at the base of the EB-GB 3 layer

◆ General

The elementary damage is defined by the following formula:

$$\Delta D = \frac{1}{N} = \left(\frac{\varepsilon_{t\,max}}{K}\right)^{\beta} \text{ where } K = 10^{6/\beta} k_{\theta f}.k_s.k_r.k_c.\bar{\varepsilon}_6$$

Where:

- Temperature and frequency correction coefficient:

$$k_{\theta f} = \sqrt{\frac{E(10°C,10Hz)}{E(\theta_{eq},f)}} = \sqrt{\frac{1.32}{0.82}} = 1.27 \quad \text{for} \quad \theta_{eq} = 15°C \quad \text{and} \quad f = 3Hz$$

- Risk coefficient:

$$k_r = 10^{-ub\delta} \quad \text{and} \quad \delta = \sqrt{S_N{}^2 + \left(\frac{c.S_h}{b}\right)^2} \quad \text{where} \begin{cases} S_N = 0.3 \\ c = 2\,m^{-1} \\ b = -0.2 \\ u = -1.960 \end{cases}$$

Since the value of S_h depends on the thickness e of the EB-GB 3 layer used in the model, so does the value of k_r. Table 35 shows the values of S_h and k_r as a function of e.

e (m)	*$e \leq 0.10$*	*0.11*	*0.12*	*0.13*	*0.14*	*$e \geq 0.15$*
S_h (m)	*0.010*	*0.013*	*0.016*	*0.019*	*0.022*	*0.025*
k_r	*0.752*	*0.744*	*0.736*	*0.726*	*0.715*	*0.703*

Table 35: values of k_r according to the thickness of EB-GB 3

Since the thickness of the base asphalt concrete used in this example is 11 cm for the three structures, the value of k_r = 0.744 is taken.

- Calibration coefficient (Table 36), determined for each structure, using the curve of k_c (see paragraph 2.7.1.2) versus the ESWL (see paragraph 2.6).

Structure	*1*	*2*	*3*
ESWL (t)	*16.1*	*15.8*	*15.9*
k_c	*1.58*	*1.57*	*1.57*

Table 36: values of k_c according to the ESWL (1).

- Foundation coefficient: k_s = 1 (modulus of the material under the EB-GB 3 layer > 120 MPa)

◆ Calculation of the damage

The damage calculations are made using software that applies the principles laid down in this guide. They take the lateral wander of the different aircraft into consideration. The damage values associated with each aircraft on takeoff and landing, and the cumulative damage are given for each structure in Table 37.

Project traffic aircraft		Structure 1	Structure 2	Structure 3
HERCULES C 130	Takeoff	< 0.001	< 0.001	< 0.001
	Landing	< 0.001	< 0.001	< 0.001
AIRBUS A310-300	Takeoff	0.118	0.025	0.036
	Landing	0.118	0.025	0.036
AIRBUS A330-200	Takeoff	0.338	0.066	0.095
	Landing	0.150	0.031	0.044
AIRBUS A340-300	Takeoff	0.262	0.052	0.074
	Landing	0.098	0.021	0.029
ANTONOV AN-124	Takeoff	1.311	0.273	0.386
	Landing	0.675	0.149	0.207
BOEING B747-200C	Takeoff	0.133	0.029	0.041
	Landing	0.072	0.017	0.023
ILYUSHIN IL 76 TD	Takeoff	0.172	0.028	0.043
	Landing	0.069	0.012	0.018
Cumulative damage		3.515	0.727	1.031

Table 37 : calculation of cumulative damage and the contribution of each aircraft in the material EB-GB 3

Note (valid for all the damage calculations): *in the tables below, the damage corresponding to each aircraft is not the maximum value of the transversal profiles of individual damage (of each aircraft). These individual damage profiles are accumulated and form a global cumulative transversal damage profile, from which the maximum cumulative damage is deduced. The contributions of each aircraft to this maximum damage are read on the individual profiles at the transversal position corresponding to the maximum cumulative damage. These are the values shown in the damage tables.*

The structure is suitable if the damage is less than 1, while remaining as close as possible to 1. Therefore, regarding the damage to the EB-GB 3 material, the first iteration (structure 1) produces a damage value that is too high, which means that the thickness of the untreated graded aggregate must be increased. Structures 2 and 3 are suitable, because the cumulative damages with lateral wander are respectively 0.727 and 1.031. Structure 1 is eliminated.

Now, we must check that the other criterion applying to the permanent deformation of the soil is met.

8.2.2.2. Damage at the top of the subgrade

The elementary damage is defined by the following formula:

$$\Delta D = \frac{1}{N} = \left(\frac{\varepsilon_{zz\max}}{K}\right)^{\beta} \quad \text{where} \quad K = 16000 \quad \text{and} \quad \beta = 4.5$$

The damage calculations are made using software that applies the principles laid down in this guide. They take the lateral wander of the different aircraft into consideration. The contribution of each aircraft on takeoff and landing, and the cumulative damage, are given for the permanent deformation of the subgrade of the three structures in Table 38. The damage for structure 1 is provided for reference only.

Project traffic aircraft		Structure 1	Structure 2	Structure 3
HERCULES C 130	Takeoff	< 0.001	< 0.001	< 0.001
	Landing	< 0.001	< 0.001	< 0.001
AIRBUS A310-300	Takeoff	0.033	0.011	0.015
	Landing	0.033	0.011	0.015
AIRBUS A330-200	Takeoff	0.058	0.017	0.023
	Landing	0.021	0.006	0.008
AIRBUS A340-300	Takeoff	0.042	0.012	0.016
	Landing	0.012	0.003	0.005
ANTONOV AN-124	Takeoff	0.684	0.247	0.317
	Landing	0.300	0.107	0.137
BOEING B747-200C	Takeoff	0.021	0.007	0.009
	Landing	0.010	0.003	0.004
ILYUSHIN IL 76 TD	Takeoff	0.043	0.016	0.021
	Landing	0.016	0.006	0.008
Cumulative damage		1.272	0.445	0.575

Table 38: calculation of the cumulative damage and the contribution of each aircraft at the summit of the subgrade (1)

Structures 2 and 3 meet the two design criteria and they are both acceptable from a mechanical perspective. However, the second iteration that produced structure 3 optimized the thickness of the materials to be used by reducing the thickness of the untreated graded aggregate, compared with structure 2. The structure with damage values closest to 1 is selected. Consequently, structure 3 is considered to be the result of the mechanical design process, provided that the base layer thickness is sufficient.

Figure 30 shows the curves of the variation in damage along the transversal profile of the runway for this structure.

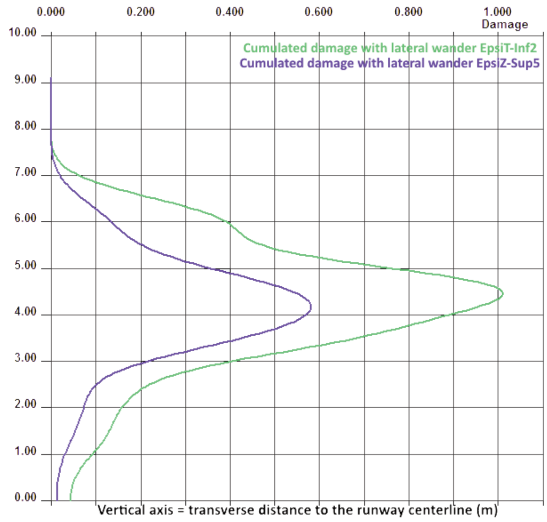

Figure 30: Structure 3 – transverse profiles of fatigue damage at the base of the base asphalt concrete (green) and of permanent deformation at the summit of the foundation (blue) (1)

8.2.3. Conclusion

Note that, in this example, the fatigue criterion of the asphalt concrete is decisive for the design.

The selected structure is the one that meets the two design criteria: traction at the base of the EB-GB 3 material and vertical deformation at the summit of the subgrade.

Structure 3 is selected, with **6 cm of EB-BBA 2 + 11 cm of EB-GB 3 + 36 cm of category 1 untreated graded aggregate**.

It should be checked that the base course of this structure is thick enough using Figure 31 below. This requires the ESWL as input data, which is equal to 15.9 t for structure 3.

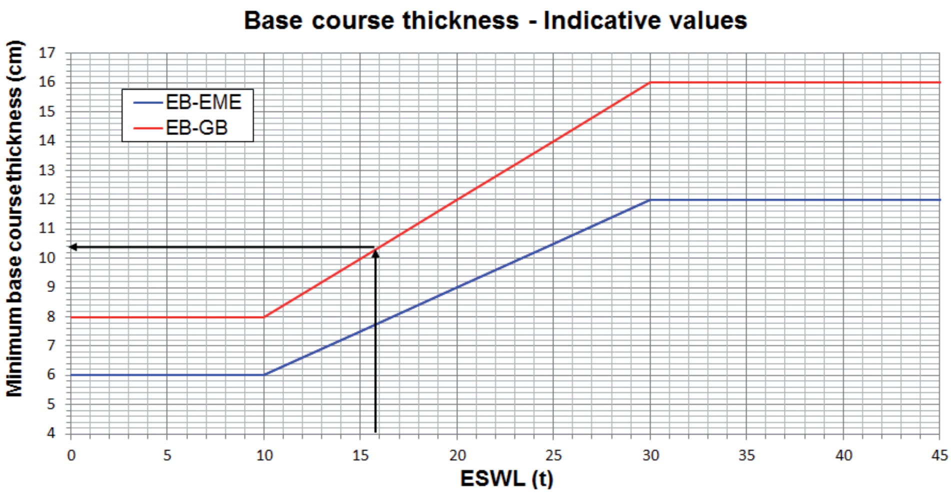

Figure 31: verification of the thickness of the asphalt concrete for the chosen structure (1)

This thickness of the base course of the selected structure is 11 cm, which is just sufficient in comparison with the 10.4 cm given in Figure 31. Therefore, the selected thickness is OK.

8.3. Example of the design of a runway

8.3.1. Data

8.3.1.1. Traffic

◆ **Traffic characteristics**

The information on the aircrafts likely to use the airfield pavement to be designed is shown in Table 39. The distinction is made between takeoffs and landings. The weight and the number of movements are shown for each of them. Note that one movement represents either a takeoff or a landing.

Project traffic aircraft	*Mrw (t)**	*Mlw (t)**	*Takeoffs/year*	*Landings/year*	*Cumulative traffic by aircraft type (for 10 years)*
A340-200	*260*	*174.7*	*315.4*	*182.6*	*4980*
B777-300 ER	*341*	*243.8*	*399*	*231*	*6300*
A330-300	*230*	*184.0*	*296.4*	*171.6*	*4680*
A320-200 JUM	*71*	*59.5*	*1086.8*	*629.2*	*17160*
EMB 190 LR/AR	*50.3*	*45.8*	*691.6*	*400.4*	*10920*
B737-100	*42.4*	*35.1*	*509.2*	*294.8*	*8040*
AN124	*392*	*319.5*	*22.8*	*13.2*	*360*
B747-400 Cargo	*377.8*	*287.1*	*98.8*	*57.2*	*1560*
KC135	*136.8*	*101.1*	*38*	*22*	*600*
B707-320B	*146.8*	*108.3*	*38*	*22*	*600*
A310-300	*157*	*125.6*	*1387*	*803*	*21900*
CASA CN325-100	*15.1*	*13.6*	*1387*	*803*	*21900*

*** since the data are taken from actual traffic, the weights are different, and more precise, that those in the « Ficav » database, which contains the Mrw and the Mlw on the airworthiness certificates**

Table 39: traffic taken into consideration for design purposes (2)

The design period is **10 years**. Growth in traffic in this period is assumed to be zero. In this case, it is possible to calculate the cumulative number of passages of each aircraft in the design period. These results are shown in the « Cumulative traffic per type of aircraft (for 10 years) » column in Table 39.

The geometry and the loading conditions of the landing gear of each aircraft are defined in the STAC's « Ficav » database.

Since the list of aircraft using the runway is long, we propose to use the method described in paragraph 3.1.2.3 of this guide in order to reduce the number of aircraft taken into consideration in the design process.

Remember that, for each aircraft, this method takes account of the highest load at the wheel, and compares it with the highest load of all the traffic. The coefficient, which is calculated for each aircraft, allows the aircraft to be considered as a decisive, if $r_j \geq 1\%$.

Table 40 shows the results of the application of this method. Only five aircraft will be taken into consideration for the rest of the calculation (the aircraft shown in green in Table 40).

Project traffic aircraft	Pr_j (in tons)	n_j	P_j	r_j
A340-200	27.42	4 980	5.0%	4.6%
B777-300 ER	27.17	6 300	6.4%	5.5%
A330-300	27.93	4 680	4.7%	4.7%
A320-200 JUM	18.02	17 160	17.3%	1.9%
EMB 190 LR/AR	11.98	10 920	11.0%	0.2%
B737-100	11.57	8 040	8.1%	0.1%
AN124	19.31	360	0.4%	0.1%
B747-400 Cargo	24.23	1 560	1.6%	0.8%
B KC135	17.82	600	0.6%	0.1%
B707-320B	17.55	600	0.6%	0.1%
A310-300	19.51	21 900	22.1%	3.7%
CASA CN325-100	3.8	21 900	22.1%	0.0%
	$Pr_{jmax} = 27.93$	$n_{tot} = 99\,000$		

Table 40: determination of the aircraft considered as design criteria (green)

The runway to be designed is considered as a high-speed section. Therefore, the **speed of movement** of all the aircraft is **100 kph** (corresponding to a frequency of 10 Hz) and the **lateral wander** of each aircraft is characterized by a standard deviation of 0.75 m (see paragraph 3.1.3 in the guide).

◆ Determination of the traffic class « CTi »

The method used to determine the traffic class (see glossary, Table 53) is defined in the GAN [3]. The application of this method to the selected traffic resulted in Table 41.

Project traffic aircraft	*Group*	*F (mvts/d)*	*Traffic class*
A340-200	*5*	*1.27*	*CT4*
B777-300 ER	*5*	*1.73*	*CT4*
A330-300	*5*	*1.28*	*CT4*
A320-200 JUM	*3*	*4.70*	*CT2*
A310-300	*4*	*6.00*	*CT3*

Table 41 : determination of the traffic class by type of aircraft (2)

The highest traffic class is applied to the project. In this example, it is **traffic class CT4**.

8.3.1.2. Parameters dependent on the contracting authority

A **design risk of 2.5%** is applied, because the annual traffic class is higher than CT3 (see recommendation in paragraph 3.1.1 of the guide).

8.3.1.3. Surfacing

The choice of the surfacing (the method is described in detail in the GAN [3]) depends on the stress level, which is defined according to the traffic class and the type of climate.

◆ Determination of the stress level « NSi »

The traffic class defined above is class CT4.

The **type 4 climate**, which is predominantly tropical, has been chosen for design purposes (selected equivalent temperature : θ_{eq} **= 28°C** (corresponding to the special case of Guiana, see paragraph 3.1.4).

- Therefore, the **stress level is NS4** (see glossary, table 54).

◆ Choice of the surfacing

The GAN [3] recommends one type of wearing course products for the stress level NS4 and for common parts of runways: EB-BBA 2 (application of table 12, paragraph 5.3).

The two products proposed for the binder course (application of Table 13, paragraph 5.3) are EB-BBSG 1 or EB-BBME 1 (not recommended by this guide).

For the rest of this study, the decision was taken to opt for a single **wearing course made of EB-BBA 2** (no binder course). The thickness of this layer is **6 cm**.

8.3.1.4. Base course

For stress level NS4 and for common parts of runways, the GAN [3] recommends a base course made of either a EB-GB 3 product, or an EB-EME 2 product. For this example, an **EB-EME 2** product has been selected for the **base course**.

8.3.1.5. Sub-base

The decision was taken to use a **subgrade** made up of **category 1 untreated graded aggregate**.

8.3.1.6. Pavement foundation

The target bearing capacity of the project is **PF2**^{qs}. To determine the strains and stresses in the body of the pavement in the calculation model, the modulus associated with the pavement foundation is the lower limit of the class, i.e. 80 MPa.

8.3.1.7. Mechanical characteristics of the materials

The mechanical characteristics of the bonded materials chosen for this example are shown in Table 42. These are the conventional minimum values. The values of the moduli at different temperatures and frequencies are deduced from Tables 20 and 23 (Chapter 6), showing the susceptibility of the moduli to variations in temperature and frequency.

Product	E (15°C, 10Hz) (MPa)	E (10°C, 10Hz) (MPa)	E (28°C, 10Hz) (MPa)	ε_6 (10°C, 25Hz) (µstrain)	$\beta = -1/b$	S_N	S_h
EB-BBA 2	5 500	7315	1 760	100	5	0.25	-
EB-EME 2	14 000	16 940	7 000	130	5	0.25	(1)

(1) S_h = 1 if e ≤ 10 cm, S_h = 1+0.3 (e-10) if 10 cm < e < 15 cm, S_h = 2.5 if e ≥ 15 cm

Table 42: mechanical characteristics of bituminous materials (2)

The untreated graded aggregate is divided (for design purposes) into 0.25 m thick sub-layers. The modulus of the category 1 untreated graded aggregate is equal to three times the modulus of the underlying layer, up to a maximum limit of 600 MPa. Therefore, the modulus of the first sub-layer will equal 80 MPa x 3 = 240 MPa, the modulus of the second layer will be 240 MPa x 3 = 720 MPa, this value being limited to 600 MPa.

8.3.2. Calculation of the damage

The damage is calculated at two locations in the structure of the pavement:

- at the base of the base course (EB-EME 2) – traction fatigue damage (criterion $\mathcal{E}_t$),
- at the summit of the subgrade – compression damage by permanent deformation (criterion $\mathcal{E}_{zz}$).

Since the thicknesses are defined by an iterative procedure resulting in one of the two damages close to 1 (by inferior value), the damage calculations are made for different structures. Two of them are presented here:

- Structure 1 – 6 cm of EB-BBA 2 + 15 cm of EB-EME 2 + 40 cm of category 1 untreated graded aggregate,
- Structure 2 – 6 cm of EB-BBA 2 + 12 cm of EB-EME 2 + 33 cm of category 1 untreated graded aggregate.

8.3.2.1. Damage at the base of the EB-EME 2 layer

◆ General

The elementary damage is defined by the following relation: $\Delta D = \frac{1}{N} = \left(\frac{\varepsilon_{t\,max}}{K}\right)^{\beta}$ where $K = 10^{6/\beta} k_{\theta f}.k_s.k_r.k_c.\bar{\varepsilon}_6$

With:

- Temperature and frequency correction coefficient: $k_{\theta f} = \sqrt{\frac{E(10°C,10Hz)}{E(\theta_{eq}, f)}} = \sqrt{\frac{1.32}{0.70}} = 1.37$ for $\theta_{eq} = 15°C$ and $f = 1Hz$

- Risk coefficient: $k_r = 10^{-ub\delta}$ and $\delta = \sqrt{S_N{}^2 + \left(\frac{c.S_h}{b}\right)^2}$ where $\begin{cases} S_N = 0.3 \\ c = 2\,m^{-1} \\ b = -0.2 \\ u = -1.645 \end{cases}$

Since the value of S_h edepends on the thickness e of the EB-EME 2 layer used in the model, so does the value of k_r. Table 43 shows the values of S_h et k_r as a function of e.

e (m)	$e \leq 0.10$	*0.11*	*0.12*	*0.13*	*0.14*	$e \geq 0.15$
S_h *(m)*	*0.010*	*0.013*	*0.016*	*0.019*	*0.022*	*0.025*
k_r	*0.784*	*0.775*	*0.765*	*0.753*	*0.740*	*0.727*

Table 43: values of k_r according to the thickness of EB-EME 2

- Calibration coefficient (Table 44), determined for each structure, using the curve of k_c (see paragraph 2.7.1.2) versus the ESWL (see paragraph 2.6).

Structure	*1*	*2*
ESWL(t)	*23.4*	*22.8*
k_c	*1.45*	*1.43*

Table 44: values of k_c according to the (2)

- Foundation coefficient: k_s = 1 (modulus of the material under the EB-EME 2 layer > 120 MPa)

◆ Damage calculations

The damage calculations are made using software that applies the principles laid down in this guide. They take the lateral wander of the different aircraft into consideration. The damage by type of aircraft on takeoff and on landing and the cumulative damage for each structure are shown Table 45.

Note (valid for all the damage calculations: *in the tables below, the damage corresponding to each aircraft is not the maximum value of the transversal profiles of individual damage (of each aircraft). These individual damage profiles are accumulated and form a global cumulative transversal damage profile, from which the maximum cumulative damage is deduced. The contributions of each aircraft to this maximum damage are read on the individual profiles at the transversal position corresponding to the maximum cumulative damage. These are the values shown in the damage tables.*

Project traffic aircraft		Structure 1	Structure 2
A340-300	Takeoff	0.014	0.037
	Landing	0.002	0.007
B777-300 ER	Takeoff	0.023	0.064
	Landing	0.005	0.014
A330-300	Takeoff	0.016	0.044
	Landing	0.005	0.014
A320-200 JUM	Takeoff	0.008	0.019
	Landing	0.002	0.006
A310-300	Takeoff	0.031	0.081
	Landing	0.014	0.034
Cumulative damage		0.120	0.320

Table 45: calculation of cumulative damage and the contribution of each aircraft in the material EB-EME 2

The structure is suitable if the damage is less than 1, while remaining as close as possible to 1. Therefore, regarding the damage to the EB-EME 2 material, both structures are suitable, because the cumulative damages with lateral wander are respectively 0.120 and 0.320.

Now, we must check that the other criterion applying to the permanent deformation of the soil is met.

8.3.2.2. Damage at the top of the subgrade

The elementary damage is defined by the following formula: $\Delta D = \frac{1}{N} = \left(\frac{\varepsilon_{zz\max}}{K}\right)^{\beta}$ where $K = 16000$ and $\beta = 4.5$

The damage calculations are made using software that applies the principles laid down in this guide. They take the lateral wander of the different aircraft into consideration. The contribution of each aircraft on takeoff and on landing, and the cumulative damage for the permanent deformation of the subgrade for both structures are shown in Table 46. For structure 1, the damage at the summit of the subgrade is slight, with a value close to 0.34. This structure meets this criterion, but since the value is not close to 1, the structure is not optimized.

Project traffic aircraft		Structure 1	Structure 2
A340-300	Takeoff	0.033	0.100
	Landing	0.003	0.011
B777-300 ER	Takeoff	0.083	0.225
	Landing	0.011	0.031
A330-300	Takeoff	0.041	0.123
	Landing	0.009	0.028
A320-200 JUM	Takeoff	0.008	0.029
	Landing	0.002	0.008
A310-300	Takeoff	0.111	0.316
	Landing	0.042	0.108
Cumulative damage		0.344	0.979

Table 46: calculation of the cumulative damage and the contribution of each aircraft at the summit of the subgrade (2).

Structure 2 meets this criterion for the subgrade, with a damage value of 0.979. Therefore, this structure is optimized. It may be selected provided that the base layer thickness is sufficient.

Figure 32 shows the curves of the variation in damage along the transversal profile of the runway for this structure.

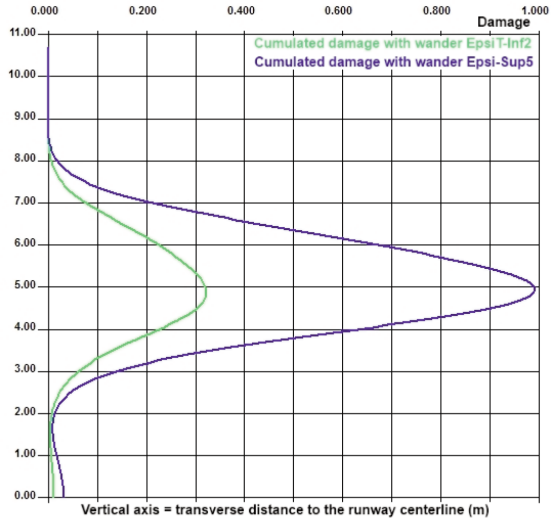

Figure 32 : Structure 2 – transverse profiles of fatigue damage at the base of the high-modulus asphalt concrete (green) and of permanent deformation at the summit of the foundation (blue) (1)

8.3.3. Conclusion

In this second example, the decisive criterion is the permanent deformation of the subgrade.

The selected structure is the one that meets the two design criteria: traction at the base of the EB-EME 2 material and vertical deformation at the summit of the subgrade.

Structure 2 is selected, with **6 cm** of **EB-BBA 2 + 12 cm of EB-EME 2 + 33 cm of category 1 untreated graded aggregate**.

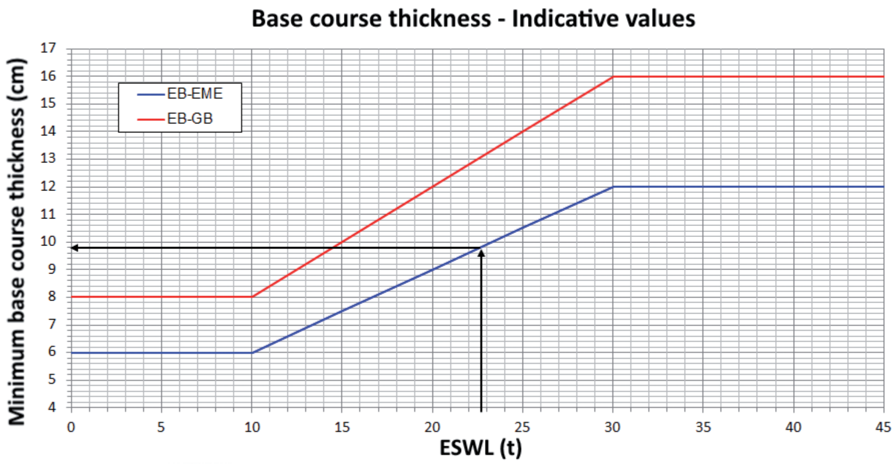

Figure 33 : verification of the thickness of the asphalt concrete for the chosen structure (2)

It should be checked that the base course of this structure is thick enough using Figure 33.

This thickness of the base course of the selected structure is 12 cm, which more than the minimum of 9.8 cm given in Figure 33, for an ESWL of 22.8 t. Therefore, the selected thickness is OK.

8.4. Example of the design of an apron

8.4.1. Data

8.4.1.1. Traffic

◆ **Traffic characteristics**

The information on the aircrafts likely to use the airfield pavement to be designed is shown in Table 47. Note that a movement represents either a takeoff or a landing.

Project traffic aircraft	*Mrw (t)*	*Mlw (t)*	*Takeoffs/year*	*Landings/year*	*Cumulative traffic by type of aircraft (for 10 years)*
TU 134A	*47*	*47**	*2 000*	*2 000*	*40 000*
FOKKER F100 STD	*40*	*40**	*1 000*	*1 000*	*20 000*

(*) : since the landing weights are not known, they are taken to be identical to the ramp weights

Table 47 : traffic taken into consideration for design purposes (3)

The **design period** is **10 years**. Growth in traffic in this period is assumed to be zero. In this case, it is possible to calculate the cumulative number of passages of each aircraft in the design period. These results are shown in the « Cumulative traffic per type of aircraft (for 10 years) » column in Table 47.

The geometry and the loading conditions of the landing gear of each aircraft are defined in the STAC's « Ficav » database.

The apron to be designed is considered as a low-speed section, on which the **speed of movement** of each aircraft is **10 kph** (corresponding to a frequency of 1 Hz) and the **lateral wander** of each aircraft is then characterized by a **standard deviation** S_{bal} = 0 (see paragraph 3.1.3).

◆ **Determination of the traffic class « CTi »**

The method used to determine the traffic class (see glossary, Table 53) is defined in the GAN [3]. The application of this method to the selected traffic resulted in Table 48.

Project traffic aircraft	*Group*	*F (mvts/d)*	*Traffic class*
TU 134A	*3*	*10.9*	*CT2*
FOKKER F100 STD	*2*	*5.48*	*CT2*

Table 48 : determination of the traffic class by type of aircraft (3)

The highest traffic class is applied to the project. In this example, it is **traffic class CT2**.

8.4.1.2. Parameters dependent on the contracting authority

A **design risk of 5 %** is applied, because the annual traffic class is lower than CT3 (see recommendation in paragraph 3.1.1 of the guide).

8.4.1.3. Surfacing

The choice of the surface layer (the method is described in detail in the GAN [3]) depends on the stress level, which is defined according to the traffic class and the type of climate (see glossary, Table 54).

◆ Determination of the stress level « NSi »

The traffic class defined above is class CT2.

The **type 3 climate**, which is predominantly continental, has been chosen for design purposes (selected equivalent temperature : θ_{eq} **= 15°C**).

- Therefore, the **stress level is NS1**.

◆ Choice of the surfacing

For stress level NS1 and for aprons, the GAN [3] recommends several types of products for the wearing course: EB-BBA 2, ESU, ECF, EP or EB-BBM 1 (application of Table 12 in Chapter 5).

The two products proposed for the binder course (application of Table 13 in Chapter 5) are EB-BBM 1 or EB-BBSG 1.

For the rest of this study, the decision was taken to opt for a single **wearing course made of EB-BBA 2** (no binder course). The thickness of this layer is **6 cm**.

8.4.1.4. Base course

For stress level NS1 and for aprons, the GAN [3] recommends the use of an EB-GB 2 product in the base course. Since this is a minimum recommendation, for this example a material with superior characteristics was selected for the base course : an **EB-GB 3**.

8.4.1.5. Sub-base

The sub-base chosen for this example is made up of a **category 2 untreated graded aggregate**, according to the recommendations in paragraph 6.7.5.

8.4.1.6. Pavement foundation

The target bearing capacity of the project is **PF2**. To determine the strains and stresses in the body of the pavement in the calculation model, the modulus associated with the pavement foundation is the lower limit of the class, i.e. 50 MPa.

8.4.1.7. Mechanical characteristics of the materials

The mechanical characteristics of the materials chosen for this example are given in Table 49. These are conventional minimum values. The values of the moduli at different temperatures and frequencies are deduced from Tables 18 and 23 (Chapter 6), showing the susceptibility of the moduli to variations in temperature and frequency.

Product	E (15°C, 10Hz) (MPa)	E (10°C, 10Hz) (MPa)	E (15°C, 1Hz) (MPa)	ε_6 (10°C, 25Hz) (µstrain)	$\beta = -1/b$	S_N	S_h
EB-BBA 2	5 500	7315	3 740	100	5	0.25	-
EB-GB 3	9 000	11 880	6 300	90	5	0.3	(1)

(1) S_h = 1 if e ≤ 10 cm, S_h = 1+0.3 (e-10) if 10 cm < e < 15 cm, S_h = 2.5 if e ≥ 15 cm

Table 49 : mechanical characteristics of bituminous materials (3)

The untreated graded aggregate is divided (for design purposes) into 0.25 m thick sub-layers. The modulus of the category 2 untreated graded aggregate is equal to 2.5 times the modulus of the underlying layer, up to a maximum limit of 400 MPa. Therefore, the modulus of the first sub-layer will equal 50 MPa x 2,5 = 125 MPa, the modulus of the second layer will be 125 MPa x 2,5 = 312,5 MPa, and the modulus of the following layers will be 400 MPa.

8.4.2. Calculation of the damage

The damage is calculated at two locations in the structure of the pavement:

- at the base of the base course (EB-GB 3) – traction fatigue damage (criterion ε_t),
- at the summit of the subgrade – damage by permanent deformation (criterion ε_{zz}).

Three calculations with different thicknesses of the untreated graded aggregate are presented in order to illustrate the iterations applied to the thickness of the sub-base:

- Structure 1 – 6 cm of EB-BBA 2 + 13 cm of EB-GB 3 + 25 cm of category 2 untreated graded aggregate,
- Structure 2 – 6 cm of EB-BBA 2 + 13 cm of EB-GB 3 + 40 cm of category 2 untreated graded aggregate,
- Structure 3 – 6 cm of EB-BBA 2 + 13 cm of EB-GB 3 + 34 cm of category 2 untreated graded aggregate.

8.4.2.1. Damage at the base of the EB-GB 3 layer

◆ General

The elementary damage is defined by the following formula:

$$\Delta D = \frac{1}{N} = \left(\frac{\varepsilon_{t\max}}{K}\right)^{\beta} \text{ where } K = 10^{6/\beta} k_{\theta f}.k_s.k_r.k_c.\bar{\varepsilon}_6$$

With:

- Temperature and frequency correction coefficient:

$$k_{\theta f} = \sqrt{\frac{E(10°C, 10Hz)}{E(\theta_{eq}, f)}} = \sqrt{\frac{1.32}{0.70}} = 1.37 \quad \text{for} \quad \theta_{eq} = 15°C \text{ and } f = 1Hz$$

- Risk coefficient:

$$k_r = 10^{-ub\delta} \text{ and } \delta = \sqrt{S_N{}^2 + \left(\frac{c.S_h}{b}\right)^2} \text{ where } \begin{cases} S_N = 0.3 \\ c = 2m^{-1} \\ b = -0.2 \\ u = -1.645 \end{cases}$$

Since the thickness of the EB-GB 3 is 13 cm in this example, the value of S_h is 0.019 m. Therefore, k_r = 0.764.

- The calibration coefficients (table 50), determined for each structure using the evolution curve of k_c:

Structure	*1*	*2*	*3*
ESWL (t)	*16.1*	*15.8*	*15.9*
k_c	*1.58*	*1.57*	*1.58*

Table 50: values of k_c according to the ESWL (3).

- Foundation coefficient: k_s = 1 (modulus of the material under the EB-GB 3 layer > 120 MPa)

◆ **Calculation of the damage**

The damage calculations are made using software that applies the principles laid down in this guide. They take the lateral wander of the different aircraft into consideration. The damage values associated with each aircraft on takeoff and landing, and the cumulative damage are given for each structure in Table 51.

Note (valid for all the damage calculations): *in the tables below, the damage corresponding to each aircraft is not the maximum value of the transversal profiles of individual damage (of each aircraft). These individual damage profiles are accumulated and form a global cumulative transversal damage profile, from which the maximum cumulative damage is deduced. The contributions of each aircraft to this maximum damage are read on the individual profiles at the transversal position corresponding to the maximum cumulative damage. These are the values shown in the damage tables.*

Project traffic aircraft		Structure 1	Structure 2	Structure 3
TU 134A	Takeoff	< 0.001	< 0.001	< 0.001
	Landing	< 0.001	< 0.001	< 0.001
FOKKER F100 STD	Takeoff	0.985	0.348	0.500
	Landing	0.985	0.348	0.500
Cumulative damage		**1.970**	**0.695**	**1.000**

Table 51: calculation of cumulative damage and the contribution of each aircraft in the material EB-GB 3

The structure is suitable if the damage is less than 1, while remaining as close as possible to 1. Therefore, regarding the damage to the EB-GB 3 material, the first iteration (structure 1) produces a damage value that is too high, which means that the thickness of the untreated graded aggregate must be increased. Structures 2 and 3 are suitable, because the cumulative damages with lateral wander are respectively 0.695 and 1.00. Structure 1 is eliminated.

Now, we must check that the other criterion applying to the permanent deformation of the soil is met.

8.4.2.2. Damage at the top of the subgrade

The elementary damage is defined by the following formula:

$$\Delta D = \frac{1}{N} = \left(\frac{\varepsilon_{zz\max}}{K}\right)^{\beta} \text{ where } K = 16000 \text{ and } \beta = 4.5$$

The damage calculation takes the lateral wander of the different aircraft (S_{bal} in this example) into consideration. The damage at the summit of the subgrade by type of aircraft, on takeoff and on landing, and the cumulative damage are given for each structure in Table 52 (the damage for structure 1 is for reference only).

Project traffic aircraft		Structure 1	Structure 2	Structure 3
TU 134A	Takeoff	< 0.001	< 0.001	< 0.001
	Landing	< 0.001	< 0.001	< 0.001
FOKKER F100 STD	Takeoff	0.153	0.056	0.087
	Landing	0.153	0.056	0.087
Cumulative damage		**0.305**	**0.112**	**0.174**

Table 52: calculation of the cumulative damage and the contribution of each aircraft at the summit of the subgrade (3)

Structures 2 and 3 meet the two design criteria and they are both acceptable from a mechanical perspective. However, the second iteration that produced structure 3 optimized the thickness of the materials to be used by reducing the thickness of the untreated graded aggregate, compared with structure 2. The structure with damage values closest to 1 is selected. Consequently, structure 3 is considered to be the result of the mechanical design process, provided that the static verification is validated and the base layer thickness is sufficient.

Figure 34 shows the curves of the variation in damage along the transversal profile of the runway for this structure.

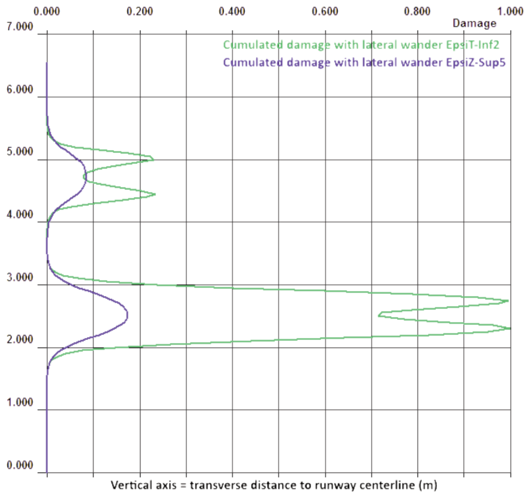

Figure 34: Structure 3 – transverse profiles of fatigue damage at the base of the base asphalt concrete (green) and of permanent deformation at the summit of the foundation (blue) (2)

8.4.3. Verification specific to aprons

Aprons undergo an additional verification for their mechanical design in order to take account of the static character of the loadings (see paragraph 3.2.4).

To perform this verification, the base course and the surface layer of the structure are modeled as untreated graded aggregates with a modulus of 800 MPa. For the foundation, only the value of the parameter K, is modified to K = 24 000.

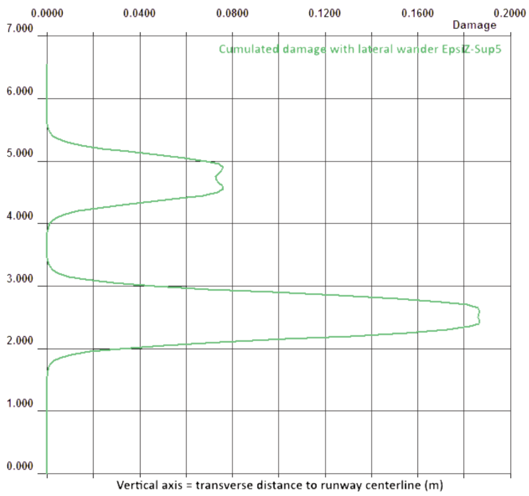

Figure 35: transversal profile of permanent deformation damage at the summit of the foundation for the verification specific low-speed sections

The verification consists of calculating the damage by permanent deformation at the summit of the foundation with this modified structure and the initial traffic. The damage must be less than 1.

Figure 35 shows the transversal profile determined using calculation software and following the principles laid down in this guide.

Since the maximum value is 0.19, this figure shows that the damage remains well below 1.

The static verification is validated for structure 3.

8.4.4. Conclusion

Note that, in this example, the fatigue criterion of the asphalt concrete is decisive for the design.

The selected structure is the one that meets the two design criteria, while optimizing the thicknesses of the layers: traction at the base of the EB-GB3 material and vertical deformation at the summit of the subgrade. This structure also passes the verification specific to aprons.

Structure 3 is selected, with **6 cm of EB-BBA 2 + 13 cm of EB-GB 3 + 34 cm of category 2 untreated graded aggregate.**

It should be checked that the base course of this structure is thick enough using Figure 36 below.

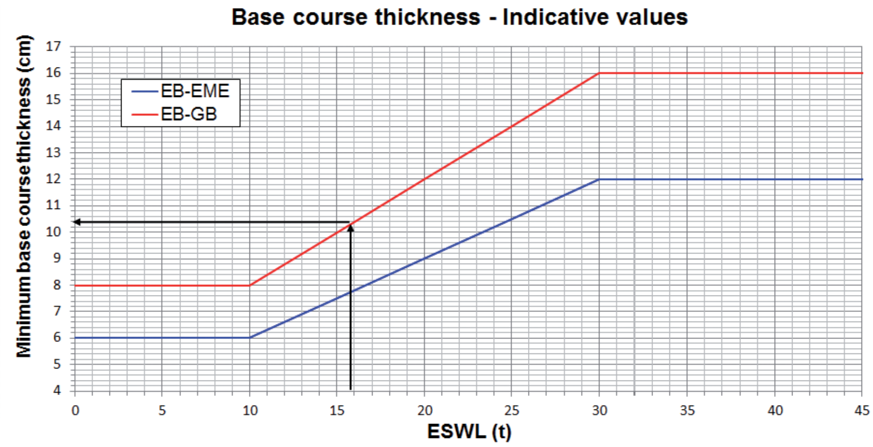

Figure 36 : verification of the thickness of the asphalt concrete for the chosen structure (3)

The thickness of the base course in the selected structure is 13 cm, which is sufficient in relation to the indicative value in the above figure when ESWL=15.9 t, i.e., 10.4 cm. Therefore, the design calculation is confirmed.

8.5. Example of freeze-thaw verification

8.5.1. Data

A freeze-thaw verification is performed in this part for the structure designed in paragraph 8.2. This is the design of a taxiway in a zone with a continental climate (type 3) with an equivalent temperature θ_{eq} = 15°C.

In this example, the structure is verified for **total protection**, meaning that the depth of frost corresponding to the exceptionally harsh winter must not reach the frost-susceptible layers of the subgrade (Q_g = 0).

In this example, the following structure is considered, which is correctly mechanically designed:

- 6 cm of EB-BBA 2,
- 11 cm of EB-GB 3,
- 36 cm of category 1 untreated graded aggregate.

The pavement structure rests on a PF2 foundation made up of clay loam covered with a 50 cm thick capping layer of untreated alluvial graded aggregate (classified D21) that is not frost-susceptible (SGn).

The frost swell test of the clay loam produces a gradient of p = 0.3 mm/(°C.h)½.

Therefore, this soil is classified as **slightly frost-susceptible** (SGp).

The verification is conducted using the method described in Chapter 7.

8.5.2. Frost-susceptibility of the pavement foundation

$$Q_{ng} = \frac{A_n \times h_n^{\,2}}{h_n + 10} \quad \text{where } A_n = 0.12\,(°C.\,day)^{1/2}\,cm^{-1} \text{ and } h_n = 50\,cm$$

i.e. $$Q_{ng} = \frac{0.12 \times 50^2}{50+10} = 5(°C.\ day)^{1/2}$$

The thermal protection Q_{ng} provided by the non frost-susceptible material in the capping layer is:

$$Q_{PF} = Q_{ng} + Q_g = 5(°C.\ day)^{1/2}$$

Therefore, the quantity of permissible frost at the pavement foundation is:

8.5.3. Thermal protection provided by the structure of the pavement

In this case, a reference frost index is taken into consideration that corresponds to the exceptionally harsh winter I_{HRE} = 200°C.day.

Consequently, the quantity of frost transmitted to the surface of the pavement can be deduced:

$$Q_s = \sqrt{0.7 \times (I_{HRE} - 10)} = 11.5(°C.\ day)^{1/2}$$

The next step consists of determining the quantity of frost transmitted in the foundation. The calculation uses the simplified method:

$$Q_s = (1 + a \times h) \times Q_t + b \times h$$

We have :

- $h = \sum h_i = 6 + 11 + 36 = 53\,cm$

- $a = \frac{1}{h}\sum(a_i \times h_i) = \frac{1}{53}\left(0.008 \times 6 + 0.008 \times 11 + 0.008 \times 36\right) = 0.008\,(°C.\ day)^{1/2}\,cm^{-1}$

- $b = \frac{1}{h}\sum(b_i \times h_i) = \frac{1}{53}\left(0.06 \times 6 + 0.06 \times 11 + 0.10 \times 36\right) = 0.099\,(°C.\ day)^{1/2}\,cm^{-1}$

And, therefore : $Q_t = \frac{Q_s - b \times h}{1 + a \times h} = \frac{11.5 - 0.099 \times 53}{1 + 0.008 \times 53} = 4.41\ (°C.\ day)^{1/2}$

8.5.4. Conclusion

The following condition must be verified to achieve total protection : $Q_t < Q_{PF}$

In this example, we have: $\begin{cases} Q_t = 4.41\ (°C.\ day)^{1/2} \\ Q_{PF} = 5(°C.\ day)^{1/2} \end{cases}$, so the inequality is verified.

Therefore, the pavement structure **passes the freeze-thaw** verification for total protection.

GLOSSARY

I. Airfield infrastructures

Aerodrome

An aerodrome is a surface, on land or water, intended to be used, in part or in full, for the arrival, departure and maneuver of aircraft.

Aircraft

An aircraft is a means of transport capable of operating in the Earth's atmosphere.

Aircraft stand taxilane

A taxiway next to or crossing the traffic areas.

Airplane

An airplane is an aircraft heavier than air that is maintained in flight by an aerodynamic force known as lift, created by the movement of the wings in the mass of air, and driven by an engine or a jet engine.

Connections (intersection strips / extra width in bends)

A connecting surface at the intersections between taxiways that allows an aircraft to change direction with a sufficient margin of clearance.

Displaced threshold

A portion of the runway between the offset threshold and the runway end that can be used on takeoff.

Helicopter

When the aircraft is heavier than air, driven by an engine and maintained in flight mainly by aerodynamic reactions on mobile surfaces, it is known as a helicopter.

Maneuvering area

Part of an aerodrome used for takeoffs, landings and surface maneuvers, but excluding traffic areas.

Movement

A movement is either a takeoff or a landing.

Movement area

Part of an aerodrome used for takeoffs, landings and surface maneuvers, including traffic areas (see Figure 1).

Rotation

A return flight of an aircraft, equivalent to two movements (takeoff and landing) in terms of traffic.

Runway

A rectangular area on land aerodromes used by aircraft to take off and land. The long sides of this rectangle are known as the edges, while the short sides as called ends and the longitudinal axis is called the runway axis.

Runway exit

A surface next to a taxiway that allows aircraft to enter or leave the runway.

Runway turn pad

An area at the end of a runway intended to make it easier for airplanes to turn round.

Shoulder

A strip of terrain bordering a pavement, built to provide a surface that connects this pavement and the surrounding terrain, such that an aircraft that accidentally leaves the pavement does not suffer and structural damage and no foreign bodies are ingested or projected by the engines.

Taxiway

A road on a land aerodrome used for the surface traffic of aircraft and intended to connect the different parts of the movement area.

Traffic area or apron

A surface on an aerodrome on land, intended for use during the boarding and unboarding of passengers, the loading and unloading of mail or freight, the supply or recovery of fuel, parking or maintenance.

Waiting area

A delimited area, where aircraft can be held or overtaken to facilitate surface traffic. Defrosting and washing areas and run-up areas can be considered as aprons

II. Pavement structures

Base course

A layer of the pavement between the sub-base and the surfacing.

Binder course

A pavement layer between the wearing course and the pavement base. Binder courses are not always used in airfield pavements.

Bonded interface

The link between two layers, for which all the displacements and strains are assumed to be continuous in the plane of the interface.

Capping layer

A layer between the top of the earthworks and the pavement layers, intended to homogenize the characteristics and reach and maintain the geometric, mechanical, hydraulic and thermal performances taken as hypotheses in the design and calculation of the pavement. The capping layer can be made up of materials already in place or added materials, treated or otherwise.

Elementary layer

A part of the pavement installed in a single operation.

Flexible structures

Pavement structures made up of an asphalt concrete wearing course and base course. The sub-base is made of untreated graded aggregate.

Interface

The contact surface between two pavement layers of the same or different natures. In the design method, the mechanical operation of interfaces can be bonded, sliding or semi-bonded, depending on the materials in contact. In the design of new pavements, the interfaces are assumed to be bonded. The interfaces are also assumed to be closed, i.e., with bilateral contact.

Layer

A structural element of a pavement, made up of a single standardized product. A layer can be spread in one or more elementary layers.

Level surface of earthwork

The surface of the upper part of the earthworks that supports the capping layer (if there is one). The classes of level surfaces (PSTi / ARi) are defined in the GTR according to the nature of the materials making up the upper part of the earthworks and the hydrous environment.

Pavement

A structure made up of one or several layers intended to convey traffic on land without damaging the subgrade.

Pavement base

The main structural element of a pavement. The base can be made up of one or more layers, called the base course and the sub-base

Pavement foundation or foundation

The surface of the capping layer that supports the layers of the pavements. If there is no capping layer, the foundation and the level surface of earthwork are one and the same

Pavement structure

All the layers of materials resting on the pavement foundation, intended to distribute the loads produced by aircraft traffic onto the natural soil.

Permissible stress in a pavement layer

The intensity of the stress or elastic strain in a pavement layer that cannot be exceeded.

Semi-bonded interface

Link between two layers, conventionally based on the calculation hypothesis corresponding to one half of the sum total of the results achieved successively with a bonded interface and a sliding interface.

Sliding interface

The link between two layers, for which the horizontal shear stress is assumed to be zero. In this case, the strains in the plane of the interface is discontinuous

Strain

Stress or elastic deformation resulting from the calculation of the pavement structure.

Sub-base

A layer of the pavement resting on the subgrade or the capping layer, when there is one..

Surfacing
The surfacing may comprise one or more layers, called the wearing course and the binder course.

Wearing course
The top layer of the pavement in contact with the aircraft tires.

III. Parameters used in the design of pavements

Cumulative traffic
The number of passages of a type of aircraft maneuvering on the movement area during the design period.

Decisive traffic
The part of the traffic, of which the aircraft directly influence the design of the pavement structure due to their heavy weight and/or the number of cumulative passages in the design period.

Design period - Initial calculation duration
The duration or period chosen for the design calculation of the pavement structure.

Equivalent Single Wheel Load (ESWL)
The ESWL associated with aircraft traffic and a pavement structure is the simple, non-swept load (in tons) applied 10,000 times to the structure, with a footprint with a radius of 0.20 m, which produces the same value of fatigue damage of the asphalt concrete as the complete traffic.

Life time
The duration between the construction of the pavement (new pavements) and its destruction (the pavement is in a state that prevents it from being used for its intended purpose).

Risk
The expectancy, in the sense of probabilities, of the length of pavement *r* (%) to be rebuilt at the end of the design period, in the absence of any structural maintenance operations.

Traffic
The traffic is the number of movements and the types and weights of the aircraft that may use the airfield pavement during the design period.

IV. Parameters used in the freeze-thaw verification

Frost index

For a given time and period, an index linked to the absolute value of the sum total of the average daily negative temperatures. It is expressed in Celsius per day. The method used to calculate the frost index is described in NF P 98-080-1 [34].

Frost susceptibility

A quantity that characterizes the behavior of soil or granular material in response to the effects of freeze/thaw. It is assessed using the frost swell test, as per NF P 98-234-2 [22].

Quantity of frost

The square root of the frost index.

Quantity of permissible frost at the foundation

The sum total of the thermal protection provided by the non frost-susceptible materials in the subgrade and by the capping layer, when there is one, situated above the first frost-susceptible layer of the subgrade, and of the permissible quantity of frost at the surface of the first frost-susceptible layer.

V. Concepts used in the design of pavements

Damage laws of bituminous materials and unbonded materials

The damage laws are used to express the evolution of the state of the materials according to their loading history, and to determine the design criteria. The phenomenon of traction fatigue cracking is the mechanism taken into consideration for bituminous materials. The phenomenon of cumulative permanent deformation under repeated compression is the mechanism taken into consideration for unbonded materials. In both cases, the damage laws are translated by the combination of Wöhler's laws (damage under repeated loadings of a constant amplitude) and Miner's law (cumulative damage in the case of loadings of a variable amplitude). Only the nature of the loadings (stress or reversible strain) controlling the phenomena and the numerical values of the parameters of these laws are subject to change.

Service level

All the requirements defined by the contracting authority to guarantee certain conditions of access to users. These requirements are closely linked to the importance of the aerodrome, the socio-economic policy, etc.

VI. Parameters taken from the gan [3]

Aircraft group

This notion is based on two variables that are representative of the impact of an aircraft on a pavement: the tire pressure (P) and the number wheels (R) in the main landing gear. The groups are defined as follows:

- Group 1: No value. Includes all so-called light aviation aircraft, with a total weight of less than 5700 kg a tire pressure lower than 0.9 MPa.
- Group 2: (P xR) < 2 MPa
- Group 3: 2 MPa ≤ (PxR) < 4.1 MPa
- Group 4: 4.1 MPa ≤ (PxR) < 5.5 MPa
- Group 5: 5.5 MPa ≤ (PxR)

Stress level NSi

A parameter characterizing the degree of stress of an aeronautical zone, according to the type of climate and the traffic class.

<table>
<tr><th>Traffic class / Climate</th><th>CT1</th><th>CT2</th><th>CT3</th><th>CT4</th><th>CT5</th></tr>
<tr><td>Oceanic</td><td rowspan="4">NS1</td><td rowspan="2">NS1</td><td rowspan="2">NS2</td><td rowspan="2">NS3</td><td rowspan="4">NS4</td></tr>
<tr><td>Continental</td></tr>
<tr><td>Mediterranean</td><td rowspan="2">NS2</td><td rowspan="2">NS3</td><td rowspan="2">NS4</td></tr>
<tr><td>Tropical</td></tr>
</table>

Table 53: definition of the levels of stress

Traffic class

A parameter that characterizes an aeronautical zone according to the most constrictive aircraft that uses the zone, and the frequency at which it uses it (daily number of passages of the aircraft).

Tire pressure x Wheels (MPa) / Frequency (F)*	Light aircraft Total aircraft weight < 5 700 kg	$P \times R < 2$	$2 \leq P \times R < 4{,}1$	$4{,}1 \leq P \times R < 5{,}5$	$5{,}5 \leq P \times R$
	Group 1	Group 2	Group 3	Group 4	Group 5
F < 10 mvts/d**	CT1	CT2	CT2	CT3	CT4
10 mvts/d ≤ F ≤ 100 mvts/d	CT1	CT2	CT3	CT4	CT5
F > 100 mvts/d	CT1	CT2	CT4	CT5	CT5

**** one passage is one movement, i.e., takeoff or landing.***
***** if F > 1 movement per day of passage, the determined traffic class is applied to all areas (maneuvering and traffic).***
If F ≤ 1 movement per day, the traffic class only applies to the traffic area, and the traffic class of the movement area is determined by the other aircraft using the platform.

Table 54: definition of the traffic classes

Bibliography

General documentation

- **[1]** « Conception et dimensionnement des structures de chaussées » (Structural design of road pavements SETRA technical guide - LCPC, December 1994.
- **[3]** Guide to the application of standards (GAN), « Enrobés hydrocarbonés et enduits superficiels pour chaussées aéronautiques » (Bituminous mixtures and surface dressings for airport pavements), STAC, April 2009.
- **[4]** Technical guide, « Dimensionnement des chaussées, volume 1 » (Pavement design, volume 1), STBA, 1983.
- **[5]** TAC decree: decree published on 10 July 2006 relating to the technical characteristics of certain land aerodromes used by fixed-wing aircraft, and its Annex 1: Technical Annex N°1 relating to the physical characteristics of civil aerodromes used by fixed-wing aircraft.
- **[6]** Technical guide, « Chaussées aéronautiques en béton hydraulique » (Hydraulic concrete airfield pavements), STBA, 2000.
- **[7]** « Etude sur la capacité portante des accotements pour avions gros porteurs » (A study of the bearing capacity of shoulders for wide-body aircraft), STAC, 2006. This document can be downloaded from the Civil Aviation authority's Technical department site (*www.stac.aviation-civile.gouv.fr*)
- **[10]** « Note sur l'organisation des reconnaissances géotechniques des tracés routiers et autoroutiers » (Note on the organization of geotechnical investigations of road and motorway routes) (1981), French Infrastructure Ministry.
- **[11]** « Réalisation des remblais et des couches de forme » (Production of backfill and capping layers) (GTR) – Technical guide, LCPC - SETRA, 2000 (2nd edition).
- **[12]** « Traitement des sols à la chaux et/ou aux liants hydrauliques (GTS) - Application à la réalisation des remblais et des couches de forme » (Treatment of soils with lime and/or hydraulic binders (GTS) – Application to the production of backfill and capping layers) – Technical guide, January 2000.
- **[14]** Technical guide, « Conception et dimensionnement des réseaux de drainage des aérodromes » (Design of aerodrome drainage networks), STBA, 2000.
- **[15]** Guide to « Drainage routier » (Road drainage), 2006, LCPC-SETRA
- **[16]** SETRA information memo N°120, « Apport du drainage dans la conception des plates-formes support de chaussées, complément au guide drainage routier et aux guides relatifs aux terrassements » (The contribution of drainage to the design of pavement foundations, supplement to the road drainage guide and the guides to earthworks), 2009.
- **[21]** « A380 – Pavement Experimental Program », technical report, LCPC/Airbus/STBA, 2001.
- **[22]** « High Tire Pressure Test », technical report, Airbus, 2010.
- **[23]** OACI Annexe 14 Volume I, « Conception et exploitation technique des aérodromes », Convention de Chicago 1982.
- **[27]** SETRA information memo N° 114, « Eléments techniques pour la conception et la réalisation de planches d'essais de compactage dans les chantiers de terrassements » (Technical elements for the design and execution of plate compacting tests on earthworks sites), 2005

- [30] SETRA information memo N°118, « Eléments clés de l'assurance qualité des couches de formes traitées » (Key aspects of the quality assurance of treated capping layers), 2009
- [38] Technical guide, « Organization de l'assurance qualité dans les terrassements » (Organizing quality assurance in earthworks), LCPC-SETRA, 2000.

Standards

- [2] NF P 98-086: Road pavement structural design - Application to new pavements, October 2010.
- [8] NF P 94-500: Geotechnical engineering missions - Classification and specifications, 2000.
- [9] EN 1997-2: Eurocode 7 – Geotechnical design – Part 2: Ground investigation and testing, 2007.
- [13] NF P11-300: Earthworks. Classification of materials for use in the construction of embankments and capping layers of road infrastructures, 199.
- [17] EN 13108 (series of standards).
- [18] EN 13-285: Untreated graded aggregates – Specifications, 2010.
- [19] EN 12697-24: Bituminous mixtures. Test methods for hot mix asphalt - Part 24: Resistance to fatigue, 2007.
- [20] EN 12697-26: Bituminous mixtures. Test methods for hot mix asphalt - Part 26: Stiffness, 2004
- [24] NF P 98-234-2: Test relative to pavements – Frost behavior - Part 2: Frost heaving test of soils or treated or non-treated granular materials with D less than or equal to 20 mm, 1996.
- [25] NF P 18-545: Aggregates - Definition, compliance and codification, 2011.
- [26] EN 13-242: Aggregates for materials treated with hydraulic binders and untreated materials used in civil engineering works and in the construction of pavements, 2008.
- [28] NF P 94-117-1: Soils: Investigation And Testing - Formation Level Bearing Capacity - Part 1: Plate Test Static Deformation Module (ev2), 2000.
- [29] NF P 94-117-2: Soils: Investigation And Testing - Formation Level Bearing Capacity - Part 2: Modulus under dynamic loads, 2004.
- [31] EN 12697-25: Bituminous mixtures. Test methods for hot mix asphalt - Part 25: cyclic compression test, 2005..
- [32] EN 13108-20: Bituminous materials – Materials specifications - Part 20: type testing, 2006.
- [33] EN 13108-1: Bituminous materials – Materials specifications - Part 1: Asphalt concrete, 2007.
- [34] EN 13108-2: Bituminous materials – Materials specifications - Part 2: Asphalt Concrete for very thin layers, 2006.
- [35] EN 13286-7: Unbound And Hydraulically Bound Mixtures - Part 7: Cyclic load triaxial test for unbound mixtures, 2004.
- [36] NF P 98-080-1: Pavements - Earthworks - Terminology - Part 1: terminology relating to the design of pavements, 1992.
- [37] NF P 98-114-3: Road Foundations - Methodology for laboratory study of materials treated with hydraulic binders - Part 3: Soils treated with hydraulic binders, possibly associated with lime, 2001.
- [39] NF P98-200-1: Road testing - Measurement of a rolling load deflection - Part 1: definitions, measurements, specific values, 1991.

Symbols and abreviations

The symbols and abbreviations are classified by theme:

- General
- Mechanical design
- Traffic
- Pavement subgrade
- Materials
- Freeze-thaw

General

- ACN aircraft classification number
- CBR california bearing ratio
- CCTP special technical clauses
- GAN guide to the application of standards
- GTR guide to earthworks for roads
- GTS guide to soil treatment
- HTPT high tire pressure test
- IFSTTAR institut français des sciences et technologies des transports, de l'aménagement et des réseaux (formerly LCPC)
- LCPC laboratoire central des ponts et chaussée
- ICAO international civil aviation organization
- PCA portland cement association
- PCN pavement classification number
- PEP pavement experimental program
- RESA runway end safety area
- SETRA service d'étude des transports, des routes et de leurs aménagements
- STAC service technique de l'aviation civile (formerly STBA)
- STBA service technique des bases aériennes

Mechanical design

b	damage law gradient
β	damage law parameter ($\beta = -1/b$)
c	coefficient associating the variation in strain with the random variation of the thickness of the pavement Δh (m^{-1})
ΔD	elementary damage created by a loading cycle
ΔD_{bal}	elementary damage with integration of the lateral wander created by a loading cycle
$\Delta D_{bal,\ cumulated}$	elementary damage with integration of the lateral wander created by a loading cycle

$\Delta D_{max,\ cumulated}$	maximum of the cumulative damage of all the criteria taken into consideration for the design
e_{mini}	minimum thickness of the base course (m)
E	Modulus of elasticity (MPa)
$E\ (\theta, f)$	norm of the complex modulus at the temperatureand the frequency (MPa)
ε_t	amplitude of the traction/compression horizontal strain
ε_{zz}	amplitude of the vertical compression strain
$\varepsilon_6\ (\theta, f)$	strain at which the conventional failure by bending on a specimen occurs after 10^6 cycles with a 50% probability, at the temperature θ and the frequency f
$\overline{\varepsilon_6}$	strain at which the conventional failure by bending on a specimen occurs after 10^6 cycles with a 50% probability, at the 10°C and 25 Hz
f	frequency (Hz)
h_i	thickness of the layer of materials i (m)
h_{sub}	depth at which the rigid substratum is situated (counting from the level surface of earthwork) (m)
k	multiplication coefficient of the modulus of untreated graded aggregates
$k_{\theta f}$	temperature and frequency correction coefficient
k_r	correction coefficient integrating the risk and dispersion factors
k_s	coefficient to integrate local inconsistencies in the bearing capacity of the unbound layer underlying the bituminous layers
k_c	calibration coefficient
K	parameter of the damage law
N	number of cycles to failure
$N_{aircraft}$	number of passages of an aircraft
n_b	number of trajectories taken into consideration to calculate damage with lateral wander
ν	Poisson's ratio
P	point of the coordinate calculation grid (x_i, y_j, z_k)
$(P_j)_b$	proportion of the traffic moving on the ordinate trajectory $(y_j)_b$
P_m	probability of the passage of the aircraft in question, associated with the temperature profile $\theta_m(z)$
r	design risk (%)
$R(\theta, f)$	ratio indicating the sensitivity to temperature and frequency of an asphalt concrete
s_{max}	maximum amplitude of strain or stress
S_N	standard deviation on the logarithm of the number of cycles to failure by fatigue
S_h	standard deviation on the thickness of the material layer used (m)
S_{bal}	standard deviation of the lateral distribution of the traffic (lateral wander)
θ	design temperature (°C)
θ_{eq}	equivalent temperature (°C)
$\theta_m(z)$	vertical temperature profile (°C)
u	random variable of the reduced centered normal law associated with the risk r
$(y_j)_b$	ordinates of the trajectories taken into consideration to calculate damage with lateral wander (m)

Traffic

- *CTi* traffic class *i* (see GAN)
- *NSi* stress level *i* (see GAN)
- *V* speed of aircraft movement (kph)
- *Mrw* maximum ramp weight (t)
- *Mtow* maximum takeoff weight (t)
- *Mlw* maximum landing weight (t)
- *Mzfw* maximum zero fuel weight (t)
- Pr_j highest load at the wheel for the aircraft *j* (t)
- Pr_{max} the highest load at the of the entire traffic (t)
- n_j number of passages of the aircraft *j*
- n_{tot} total number of passages of all the aircraft
- p_j percentage of the accumulated traffic represented by the aircraft , relative to the total accumulated traffic
- r_j ratio determining the decisive character of an aircraft for design purposes
- *ESWL* equivalent single wheel load (t)

Pavement subgrade

- *ARi* class of long-term bearing capacity of the level surface of earthwork
- *PFi* long-term bearing capacity of the pavement foundation
- *PST* upper part of the earthworks

Materials-tests

- *BBA* aeronautical asphalt concrete
- *BBSG* semi-coarse asphalt concrete
- *BBME* high-modulus asphalt concrete
- *BBM* thin asphalt concrete
- *BBTM* very thin asphalt concrete
- *EB* asphalt concrete
- *ECF* cold asphalt concrete
- *EP* grouted macadam
- *ESU* surface dressing
- *EME* high modulus asphalt concrete
- *GB* base asphalt concrete
- *GNT* untreated graded aggregate
- *GRH* humidified graded aggregate

- I_p plasticity index
- IPI immediate bearing index
- I_{CBR} CBR index after immersion
- $MAER$ slaved rheology test machine
- OPM optimum proctor modified
- OPN optimum proctor normal
- PCG gyratory shear compactor
- R_c compression resistance
- R_t direct traction resistance
- $\rho_{d_{OPN}}$ density of the dry soil at optimum proctor normal
- SMA stone mastic asphalt
- TCR repeated load triaxial
- VBS blue methylene value of a soil
- w_{OPN} water content at optimum proctor normal
- w_{OPM} water content at optimum proctor modified

Freeze-thaw

- a_i characteristic coefficient of the material i
- A_i coefficient associated with non frost-susceptible material
- b_i characteristic coefficient of the material i
- HRE exceptionally harsh winter
- $HRNE$ unexceptionally harsh winter
- SGn non frost-susceptible material class
- SGp slightly frost-susceptible material class
- SGt very frost-susceptible material class
- p gradient of the frost swell test (mm/(°C.hour)$^{1/2}$)
- h thickness of the body of the pavement (cm)
- h_i thickness of the pavement layer i (cm)
- h_n thickness of the upper stratum of the non frost-susceptible soil (cm)
- I atmospheric frost index (°C.day)$^{1/2}$
- Q_s quantity of frost transmitted to the surface of the pavement (°C.day)$^{1/2}$
- Q_t quantity of permissible frost at the foundation (°C.day)$^{1/2}$
- Q_g quantity of permissible frost on the surface of a frost-susceptible layer (°C.day)$^{1/2}$
- Q_{ng} thermal protection provided by the non frost-susceptible materials in the subgrade and the capping layer (°C.day)$^{1/2}$

Annex A : Wöhler-Miner's law

The continuous Wöhler-Miner law allows for the integration of groups of rolling loads (bogies) that produce complex histories of strains (often with multiple peaks, with no return to zero between the peaks), to which the notion of loading cycles cannot really be applied.

This continuous law of the evolution of the damage is expressed as follow :

$$\dot{D} = \beta \frac{< \varepsilon(t) >^{\beta-1}}{K^{\beta}} < \dot{\varepsilon}(t) >$$

where :

$\dot{D}$ = speed of increase of the damage

$\varepsilon(t)$ = intensity, at the instant, of the strain of the afferent Wöhler's law

$\dot{\varepsilon}(t)$ = temporal derivative $\varepsilon(t)$

$< x >$ = the positive part of the scalar x; $< x > = x$ si $x \geq 0$, $< x > = 0$ otherwise.

Comments :

i) the consideration of the positive part of $\varepsilon(t)$ gives a mathematical meaning to the term $<\varepsilon(t)>^{\beta-1}$ (the condition $\beta > 1$ also defines this term for $\varepsilon(t) \leq 0$) and, for asphalt concretes, demonstrates the physical fact that their damage occurs essentially in extension. For unbonded granular materials, the calculations of the fields of strain will probably result in $\varepsilon_{zz\ max}$ values in contraction everywhere, therefore positive.

ii) the consideration of the positive part of $\dot{\varepsilon}(t)$ guarantees the continually increasing character of the damage[13] and links its increase to the increases in the intensity of the strain.

iii) the physical time t does not play an explicit role in this equation, which can be rewritten in a differential form :

Any other monotonous increasing parameterization of the time would, after integration, produce the same damage values.

$$dD = \beta \frac{< \varepsilon >^{\beta-1}}{K^{\beta}} < d\varepsilon >$$

iv) it is demonstrated that, in the event of strain increasing from 0 to ε_{max}, then decreasing, the damage predicted by this evolution law coincides with the expression of the elementary damage of Wöhler-Miner's law: :

$$\Delta D = \frac{1}{N(\varepsilon_{max})} = \left(\frac{\varepsilon_{max}}{K} \right)^{\beta}$$

Therefore, the continuous law is indeed an extension of the initial « discrete » law.

[13] *The design method does not explicitly take account of any effects of healing in the asphalt concrete. However, these effects may form part of the adjustment coefficient kc.*

Annex B : calculation of the traffic distribution at a given point

Let y be the perpendicular to the taxiway. The normal centered distribution of the lateral wander is discretized by the finite sequence of trajectories $(y_j)_b$ with $b \in \{1..n_b\}$ where n_b designates the total number of trajectories considered for the calculation (transversal discretization pitch $\Delta y = 5\ cm$).

The percentage of the traffic $(P_j)_b$ represented by the aircraft in question, associated with the position $(y_j)_b$ according to the normal distribution of standard deviation S_{bal} which is supposed to be known, is calculated as follows :

$$(P_j)_b\left((y_j)_b\right) = \frac{1}{\sqrt{2\pi}} \int_{y_1}^{y_2} e^{-y^2/2}\, dy = \phi(y_2) - \phi(y_1)$$

With the distribution function of the reduced normal centered law : $\phi(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{x} e^{-y^2/2}\, dy$

Where $y_1 = \frac{(y_j)_b - \Delta y/2}{S_{bal}}$ and $y_2 = \frac{(y_j)_b + \Delta y/2}{S_{bal}}$

S_{bal} is the standard deviation and Δy is the pitch of discretation,

The figure below represents the distribution of the traffic, according to a normal centered Gaussian distribution, standard deviation S_{bal}). The percentage of traffic $(P_j)_b$ represented by the aircraft in question, associated with the position $(y_j)_b$ is shown by the section in blue.

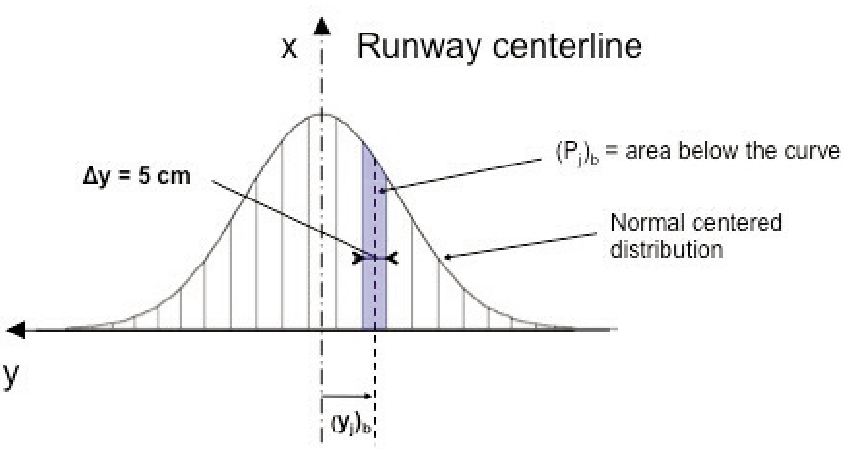

Figure B1 : lateral distribution of traffic

Annex C: parameters S_N, S_h, r and risk coefficient k_r

C1. Parameter S_N

While the production process of the pavement materials and precautions taken in their use can limit certain variations in their mechanical characteristics, the development of fatigue damage remains random in nature, as shown by the dispersion of the results of the laboratory fatigue tests on specimens. This dispersion is assumed to follow a normal probability law with a standard deviation S_N the values of which are given in Chapter 6.

C2. Parameter S_h

The coefficient S_h takes account of the variability of the thickness and of the degree of compaction of the layers of the pavement at the time of construction. This dispersion, which depends on the methods and the quality of the works, is considered to follow a normal probability law with a standard deviation S_h , the values of which are given in Chapter 6.

C3. Design risk r

The design risk r represents the expectancy, in the sense of probabilities, of the pavement length to be rebuilt after the design period, in the absence of any overlay works.

For the purposes of the design calculations, the value of the risk r is translated into normal law fractals u, considering r as a random variable to the reduced centered normal law:

$$r = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{u} e^{\frac{-x^2}{2}} dx$$

The main values of u associated with the risk r are defined in Table C1.

r (%)	u
1	- 2.326
1.5	- 2.170
2	- 2.054
2.5	- 1.960
2.8	- 1.911
3	- 1.881
5	- 1.645

r (%)	u
5.6	- 1.590
7.5	- 1.439
10	- 1.282
11.5	- 1.200
12	- 1.175
15	- 1.036
20	- 0.842

r (%)	u
23	- 0.739
24	- 0.706
25	- 0.674
30	- 0.524
35	- 0.385
40	- 0.253
50	0

Tableau C1: values of u associated with the risk r

C4. Risk coefficient k_r

The risk coefficient k_r is defined according to the standard deviations S_N and S_h and the design risk r associated with the value of the reduced centered variable, u.

Since the dispersions on the mechanical properties of the materials (standard deviation S_N) and on the thickness of the pavement layers (standard deviation S_h) are supposed to follow independent normal laws, the resulting law is a normal law.

The adjustment coefficient k_r is defined by:

$$k_r = 10^{-ub\delta}$$

The design risk r is the integral of the density of probability of the reduced variable log(N). The standard deviation, δ, associated with the variable log(N), is deduced from the combination of the dispersion factors on the fatigue law and on the thickness of the layers, by the relation:

$$\delta = \sqrt{S_N{}^2 + (\frac{c\,S_h}{b})^2}$$

where

- u: the value of the random variable of the reduced centered normal law associated with the risk r. The relation between u and r and the table mapping one to the other are given in paragraph C3,
- b: the gradient of the fatigue line of the material in the layer in question ($b = {}^{-1}\!/_{\beta}$), the values of which are given in Chapter 6 for each class of materials ($-1 < b > 0$),
- S_N: the standard deviation on the decimal logarithm of the number of cycles causing failure by fatigue, the values of which are given for each class of materials in Chapter 6,
- S_h: the standard deviation on the thickness of the base layers material, expressed in meters, the values of which are given for each class of materials in Chapter 6,
- c: he coefficient associating the variation in strain with the variation in the thickness ΔH of the asphalt concrete layer (in m^{-1}) defined by the relation:

$$\log_{10}\left(\frac{\varepsilon_t(\Delta H + H)}{\varepsilon_t(H)}\right) = -c\,\Delta H$$

The value c set on the basis of the usual structural analyses is 2 m^{-1}.

Annex D : calculation of the equivalent temperature for bituminous materials

Since the stiffness and the fatigue behavior of the bituminous materials vary with the temperature, the levels of loading and damage fluctuate in the course of the year, according to the temperature cycles. The concept of the equivalent temperature simplifies the integration of these variable situations by using a single temperature in the determination of the modulus E^* and of the coefficient $k_{\theta f}$.

In the most common situations, a single temperature of between 15°C and 28°C is used, irrespective of the aircraft, and according to the type of climate.

In special cases, where these conditions do not apply, the equivalent temperature can be calculated on the basis of the histograms of the temperature in the pavements, by aircraft type and for each criterion, using the method described below.

The equivalent temperature θ_{eq} is determined so that the sum total of the damage suffered by the pavement used by the aircraft in question, and for the annual distribution of the temperature on the site in question, is equal to the damage that the pavement would suffer with the same traffic, but for a constant temperature θ_{eq}.

The equivalent temperature is determined by applying Miner's law.

The calculation demands a histogram for each aircraft $[\theta_{\mathrm{m}}(z), p_m]$ where $\theta_{\mathrm{m}}(z)$ is a vertical temperature profile and p_m is the probability that the aircraft in question will use the pavement, associated with $\theta_{\mathrm{m}}(z)$.

The elementary damage with lateral wander ΔD_{bal} can be calculated for each thermal situation m according to paragraph 2.5.4, and the affected values of the coefficient p_m can be totaled:

$$\Delta D_{bal}(y_j, z_k) = \sum_m p_m \times \Delta D_{bal}(y_j, z_k, \theta_m(z))$$

In this case, we look for the homogeneous temperature θ_{eq} which is the solution of the equation:

$$\max_{y_j}\left\{\Delta D_{bal}(y_j, z_k, \theta_{eq})\right\} = \max_{y_j}\left\{\sum_m p_m \times \Delta D_{bal}(y_j, z_k, \theta_m(z))\right\}$$

Annex E : susceptibilities of asphalt concrete to the temperature and the frequency

The following curves represent the typical behavior of changes in the stiffness of different types of asphalt concretes according to the temperature and the frequency of the loading. They are derived from the summary of numerous test results of complex moduli made in the laboratories of the French « Réseau Scientifique et Technique », which has compiled a significant database.

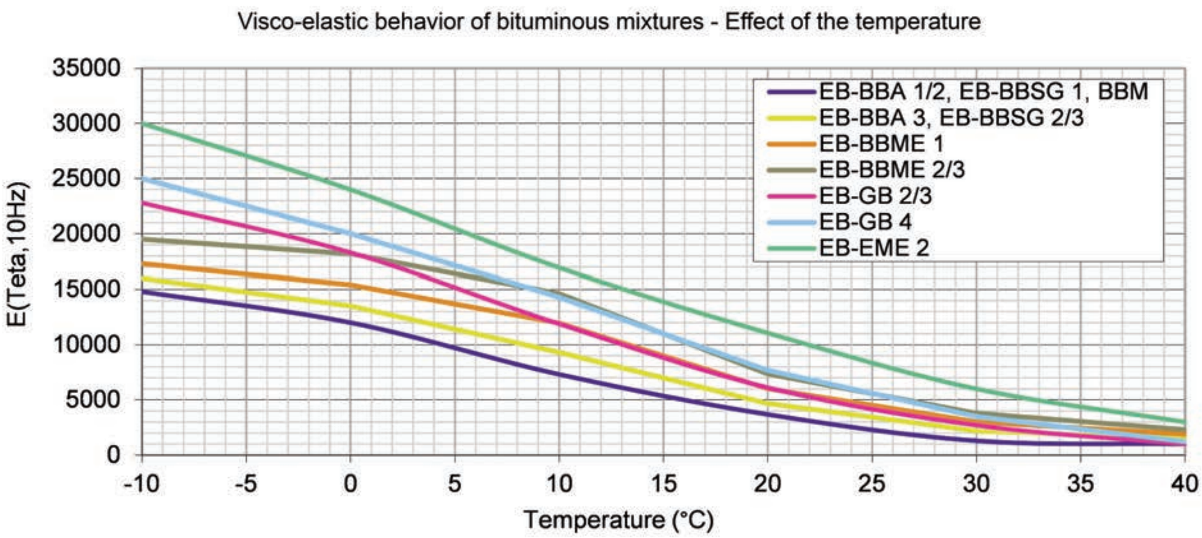

Figure E1 : effect of the temperature on the values of the moduli of different asphalt concretes

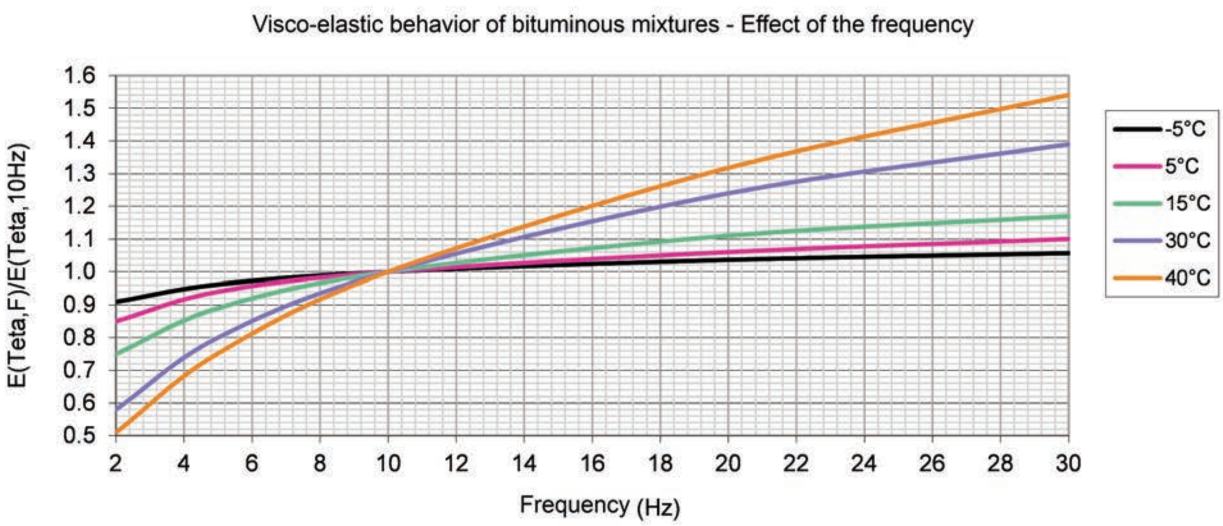

Figure E2 : effect of the frequency on the values of the moduli of different asphalt concretes

Annex F: design rules of capping layers

The thickness of material in the capping layer that is necessary to reach the required bearing capacity class of the pavement foundation is determined:

- by examining the impact of the thickness and the quality of the capping layer on the stresses and strains in the layers of the pavement and the subgrade,
- by verifying the mechanical strains and stresses in the capping layers made of treated materials.

This analysis of the workings of the operational pavement must be completed by the consideration of the behavior of the capping layer that is not protected by the body of the pavement beneath the site traffic.

The rules, which are adapted according to the soil classification in the GTR [11], are given below and extended to a broader selection of situations. The proposed thicknesses result from the mechanical calculations of the pavement to analyze the behavior of the operational pavement, and the site observations of the behavior of the capping layers under the site traffic.

F.1. Capping layer of untreated material

The GTR [11] proposes thicknesses of capping layers made of untreated materials (granular capping layers) according to the upper part of the earthworks and the classes of level surface of earthwork defined in the survey. As a general rule, on AR1 and AR2 level surface classes, these thicknesses allow for a PF2 foundation.

The over-grading rules that allow for a PF3 foundation on the basis of an AR1 or AR2 class level surface, which are thicker, are shown in Table F1[14].

Comment:

Class of the level surface	*Capping layer material*	*Thickness of the material in the capping layer*	*Resulting foundation class*
AR1	*B31, C1B31, C2B31, D21, D31, R21, R41, R61* *C1B11*, C2B11*, R11*, R42*, R62**	*0.80 m ***	*PF3***
AR2	*As above*	*0.50 m*	

** subject to a verification on the pavement foundation. The intrinsic modulus of certain granular materials cannot always guarantee a PF3 with the proposed thickness. In this case, a PF^{qs} solution is adopted.*
*** a reduction of the thickness by about 0.10 to 0.15 m can be admitted, if a suitable geotextile is inserted between the capping layer and the upper part of the earthworks.*

Table F1: conditions of over-grading of the bearing capacity of the foundations with an untreated capping layer

Over-grading to PF3 requires a verification of the target performance on the pavement foundation. The intrinsic modulus of certain granular materials cannot always guarantee a PF3 with the proposed thickness. In this case, a PF2^{qs} solution is adopted.

In the absence of experience with the selected material, it is strongly advisable to perform a plate test to verify that the modulus is effectively achieved with the material and under the conditions of use by the contractor.

[14] Refer to the GTR [11] for the thicknesses of capping layers that allow for PF2 on the basis of an AR1 level surface class. The thickness varies according to the material of the capping layer and the upper part of the earthworks.

F.2. Capping layers treated with only lime

This type of treatment is only possible with fine and moderately to highly clayey soils (measurable I_p mesurable ou $VBS > 0.5$) and in regions that are not, or only slightly, affected by frost. Usually, this type of treatment cannot be used to exceed the foundation class PF3.

This type of solution demands a specific mechanical performance analysis, or level 1 type test in the GTS [12]:

- To analyze the behavior under traffic, the values of IPI and I_{CBR} should be reached after 4 days of immersion, as defined in Chapter C1-3,4 of the GTS [12].
  This will simultaneously verify that the following two conditions are met:

$$I_{CBR} \geq 20 \text{ and } I_{CBR} / IPI \geq 1$$

- Where appropriate, for the analysis of behavior in frost, simple compression tests are performed on specimens compacted to 98.5% of the $\rho_{d\,OPN}$, fabricated at w_{OPN} and preserved for a period that is representative of the time separating the end of the works from the probable occurrence of frost on the site. The resistance to compression of the treated soil must then demonstrate that:

$$R_c \geq 2.5\,MPa$$

Note that, in most cases, treatment with lime alone does not make frost-susceptible soils insensitive to frost. Moreover, if the results of a specific frost swell tests, performed in accordance with NF P 98-234-2 [24], are available, then these results prevail over the indicated resistance value and can optimize the solution.

Finally, special attention must be paid to the representativeness of the test specimen, in relation to the pavement section concerned by the response given. It is necessary to recommend a population of measurements that is coherent with the variability of the soils and their behavior in frost.

F.3. Capping layers made of materials treated with hydraulic binders, possibly associated with lime

The hydraulic binder treatment techniques, which may be associated with lime, can achieve high levels of mechanical performance and class PF3 or PF4 foundations, provided that the conditions of execution and the dosage of binder are correct. The thickness of the capping layer and the classification of the foundation depend on:

- the bearing capacity class of the subgrade,
- the mechanical characteristics of the treated material,
- the mode of treatment (in a plant or on the site).

To this end, the behavior of the treated soil must be assessed in relation to:

- the age that permits movements on the treated layer,
- resistance to immersion at a young age,
- resistance to frost,
- the foreseeable long-term performances.

These parameters must undergo a level 1 type study, as defined in the GTS [12], which specifies the performance objectives for each of them:

1) The material of the capping layer is thus qualified by a mechanical class that is determined on the basis of:

- the chart in Figure F1, which defines the zones according to the 90-day values (or the 180-day values, for slow-setting binders) of the modulus of elasticity E and the direct tensile resistance $R_t = 0.8R_{th'}$, corresponding to the tensile mechanical resistance of the base of the capping layer (the analysis is made on specimens compacted to 96% of the OPN density, which corresponds to the degree of compaction at the base of the layer of a material compacted by a compacting energy q_3, as per the GTS [12]),
- Table F2, which shows the treatment mode, in order to take account of the differences in the homogeneity of the treated material.

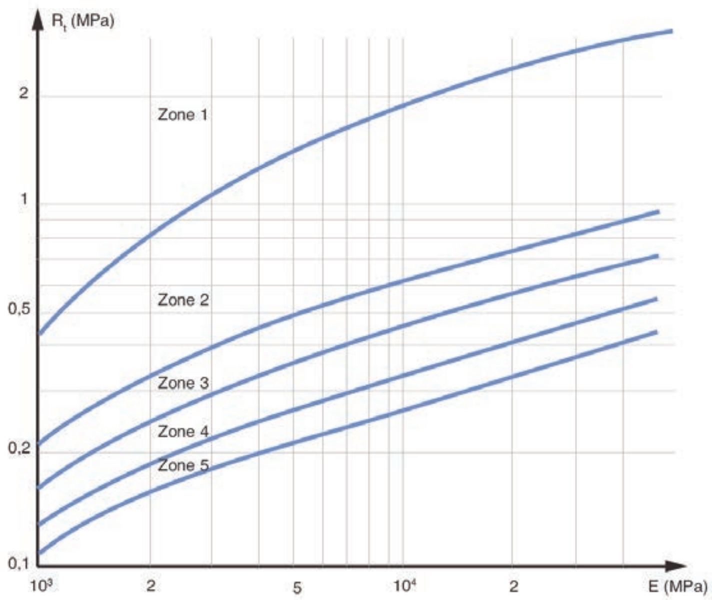

Figure F1: classification of capping layer materials treated with hydraulic binders according to their 90-day mechanical behavior

Treated in plant	*Treated on the site*	*Mechanical class*
Zone 1	-	*1*
Zone 2	-	*2*
Zone 3	*Zone 2*	*3*
Zone 4	*Zone 3*	*4*
Zone 5	*Zone 4*	*5*

Table F2: determination of the mechanical class of the sands and untreated graded aggregates treated with hydraulic binders, according to the mode of treatment

Table F3 gives the thicknesses of the capping layers to be used for the different mechanical classes of the treated material, with a view to obtaining the required foundation classes.

Mechanical class of the capping layer material		*Thickness of the capping layer material (in cm)*		
		Class AR1 level surface		*Class AR2 level surface*
	3	*(*)*	*30*	*25*
	4	*30*	*35*	*30*
	5	*35*	*50 (+)*	*35*
Foundation class obtained		*PF2*	*PF3*	*PF3*

(*) Due to the significant contrast between the moduli, the formation of a capping layer made of mechanical class 3 treated materials, on an AR1 level surface, is not permitted with thicknesses of less than 30 cm.
(+) The achievement of the required degree of compaction at the base of the layer usually leads to spreading in two layers.

Table F3 : PF class according to the AR class, the mechanical performances of the treated material and the thickness of the capping layer

Comments :

- a) The resulting foundation classes can be used to define a long-term modulus of elasticity, as defined in Table 9 in Chapter 4.4 (for example E = 120 MPa for a PF3).

- b) In view of the specifics and the severe constraints associated with loadings on airfields, PF4 class platforms with a capping layer made of materials treated with hydraulic binders are not selected.

2) In accordance with the GTS [12] and the standard NF P 98-114-3 « Road Foundations - Methodology for laboratory study of materials treated with hydraulic binders - Part 3 : Soils treated with hydraulic binders » [37], the type study of the planned treatment must be accompanied by :

- an indication of resistances at young ages (in particular in R_c), to define the curing period before the resumption of the traffic on the capping layer. The curing process guarantees hydraulic setting. During this period, any movements of heavy vehicles may be harmful to the setting process,
- an indication of frost resistance,
- an indication of the workability times,
- an indication of resistance to immersion.

The moduli and the design hypotheses of the foundation are defined during the detailed design analyses (level 2). The dosage and the nature of the binder are usually specified in the contract, because they will have already undergone level 1 and 2 analyses to optimize the solution. However, the contract may allow for adaptations in the choice of the binder and the dosage. In this case, the contractor will be required to conduct an execution analysis (type G3, as per NF P 94-500 [8]) to validate the selected design hypotheses. This analysis will be conducted during the project preparation phase, and no later than 4 months before the start of the operation. Remember that the design is validated on the basis of the performance of the treated materials after 90 days.

The performances specified in the contract must be verified by reference trials, in order to make sure that they are achieved by the contractor's own means, under normal site conditions and with the materials that have been approved for the capping layer. The acceptance of the reference trials is a milestone prior to the formal production of the capping layer, as per the guide to organizing the quality assurance of earthworks, LCPC-SETRA, 2000 [38]. It can be the first day of production in the workshop, if the results are satisfactory, and if the project manager accepts this condition.

Annex G : the works phase, acceptance criteria and constructional features relating to chipping and the surface protection of the capping layers

G.1. Acceptance criteria

In the case of criteria for construction, the threshold values depend on the importance of the project, the equipment, the methods and conditions of execution, the nature of the capping layer and the sub-base of the pavement, and on the required class of the foundation.

For regular projects, the following values generally apply (according to the GTS [12]) :

- for the rideability of the foundations required for the proper execution of the capping layer, a deformability modulus of at least EV2 > 30 MPa for the construction of granular capping layers and EV2 > 35 MPa for capping layers treated with hydraulic binders,
- a leveling of the foundation, with a tolerance of ±3 cm in relation to the red line of the earthworks for granular capping layers, and of ±2 cm for capping layers treated with hydraulic binders (or even ±1 cm, with effective adjustments, in particular if the tolerances of the upper pavement layers so require),
- maximum deformability of the foundation (upon construction), according to the nature of the capping layer and the required foundation class (Tables G1).

Note :

- *The lower values defining the foundation classes represent the long-term design characteristics, and must be adjusted by the project manager to the site conditions, for example by setting higher contractual levels.*
- *There is no direct linear relation or strict equivalence between the different acceptance methods (plate Dynaplaque, deflection, etc.) proposed hereafter. Therefore, the designer is strongly advised to use only one method per structure, in association with a minimum value and/or a characteristic statistical value (for example, 95 % of values higher than 60 MPa and 100 % higher than 50 Mpa, for a granular PF2).*

Untreated capping layer		
Target foundation	***Modulus as per NF P 94-117-1 [28] or 94-117-2 [29]***	***Characteristic deflection NF P 98-200-1 [39]***
PF2	***50 MPa***	***2 mm***
PF2qs	***80 MPa***	***1.5 mm***
PF3	***120 MPa***	***0.9 mm***
PF4	***200 MPa***	***0.6 mm***

Capping layer treated only with lime (28 days)		
Target foundation	***Modulus as per NF P 94-117-1 [28] ou 94-117-2 [29]***	***Characteristic deflection NF P 98-200-1 [39]***
PF2	***100 MPa***	***1.2 mm***
PF2^{qs}	***120 MPa***	***1.0 mm***
PF3	***150 MPa***	***0.8 mm***

Capping layer treated with a hydraulic binder, possibly associated with lime (28 days)		
Target foundation	***Modulus as per NF P 94-117-1 [28] ou 94-117-2 [29]***	***Characteristic deflection NF P 98-200-1 [39]***
PF2	-	-
PF2^{qs}	***Unsuitable test***	***0.8 mm***
PF3	***Unsuitable test***	***0.6 mm***

Table G1: required performances for the use of the pavements

Observations:

1) Class PF2^{qs}, or superior quality PF2, is neither defined nor quantified in the GTR. It is an intermediate design class that was introduced in NF P 98-086 [2], and can be used to assess foundations with acceptance characteristics of values between 80 and 120 MPa.

2) According to NF P 98-200-1 [39], the characteristic deflection is the mean value, plus two standard deviations of the deflection values measured on a given section. By default, a reference section length of 50 m is used, unless contractually stipulated otherwise.

3) For treated capping layers, acceptance is the result of a global quality assurance process that combines analyses and the various check points in the project (hydrous condition, dosage of the binder, treated thickness, milling, compacting, etc.). The process is described in the SETRA information memo N°118 SETRA (2009) [30].

4) It is advisable that the special technical clauses specify the acceptance procedure, a minimum value (all instances of non-compliance must be recorded and processed using anomaly forms) and, possibly, average values. These recommendations must be based on a certain number of tests, specifying the number of points per channel, per profile, etc.

G.2. Chipping and surface protection

Chipping

The capping layer foundations made of treated fine soils, that have to withstand intense traffic, must be protected by chipping. This operation consists of spreading and embedding crushed gravel, of a caliber of 14/20 mm or more, and with a Los Angeles coefficient equal to or lower than 35, on the foundation, after final grading and compacting. The gravel must be embedded before the end of the workability time, by two or three passes of a smooth steel roller that does not vibrate, or a rubber-tired roller.

Surface protection

The surface of capping layers made of treated soils must be protected, in addition to possible chipping. The nature of this protection depends on the role it is expected to play, the nature of the treated materials and the mechanical and climatic stress.

The protection must:

- maintain the hydrous condition of the treated material making up the capping layer (protection against ingress and evaporation) during the hydraulic setting time of the treated soil,
- favor adherence between the capping layer and the sub-base.

It may also have to withstand site traffic.

The surface protection is selected according to the role it is expected to play: maintenance of the hydrous condition, limitation or elimination of dust emissions, increased resistance of the foundation to tangential forces, etc., the climate and the length of exposure before the construction of the pavement structures.

It can range from a simple curing emulsion to a two-coat gravel dressing.

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