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Airport pavements evaluation

Dissertação para obtenção do Grau de Mestre em Estruturas e Geotecnia

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Abstract

The airport pavement deteriorates during service due to traffic and climate effects therefore systematic monitoring is required in order to assess their structural and functional condition.
The aim of this work is to present the methodologies used nowadays for airport pavement evaluation and to contribute to their improvement in structural analysis area

The main aspects that are addressed are the application of the Ground Penetrating Radar (GPR) and the use of the Falling Weight Deflectometer (FWD) tests, for structural evaluation, and the use of the GRIP tester and the measurement of texture depth of the wearing course layer, for the functional evaluation of the runway.

Also, freeware computer softwares used to design new runways (FAARFIELD and COMFAA) are presented and examples are given.
Case studies are described both for structural and functional evaluation.

Avaliação de pavimentos aeroportuários

Resumo

Os pavimentos aeroportuários sofrem ao longo do tempo degradações das suas características funcionais e estruturais. A monitorização destas características e o planeamento das medidas de reabilitação requerem campanhas periódicas de auscultação.

O objetivo do presente trabalho foi de analisar as metodologias utilizadas atualmente para a caracterização funcional e estrutural de pavimentos aeroportuários.

Procurou-se ainda avaliar a eficiência da utilização de equipamentos de auscultação de alto rendimento desenvolvidos nos últimos anos, aperfeiçoar as técnicas de interpretação e as metodologias de ensaio e elaborar recomendações relativas a utilização destes métodos. Assim para avaliação estrutural são apresentados os equipamentos Radar de Prospeção e Defletómetro de Impacto e para avaliação de características funcionais o GRIP tester e a medição da profundidade de textura.

Dois programas livre FAARFIELD e COMFAA, desenvolvidos recentemente, para avaliação de pavimentos são também apresentados, junto com exemplos para a sua melhor utilização.

São apresentados exemplos de casos de estudo, para uma melhor exemplificação tanto para a avaliação estrutural como para funcional.

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The author wishes to dedicate this work to his grandparents, Steliana and Dumitru.

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Abbreviations

GPR – Ground Penetrating Radar

FWD – Falling Weight Deflectometer

CBR – California Bearing Ratio

LNEC – Laboratório Nacional de Engenharia Civil

NDT – Non-destructive Tests

ACN – Aircraft Classification Number

PCN – Pavement Classification Number

FAA – Federal Aviation Administration

CDF – Cumulative Damage Factor

AC – Advisory Circular

P/C – Pass to Coverage Ratio

TC – Traffic Cycle

P/TC – Pass to Cycle Ratio

HWD - Heavy Weight Deflectometer

PCI – Pavement Condition Index

ICAO – International Civil Aviation Organization

PCC – Portland Cement Concrete

HMA – Hot Mix Asphalt

1 Introduction

1.1 Background

Nowadays, roads and airfields are the most used communication means to connect people around the world. The infrastructure of a country is an important parameter for the economic and social health of a country. The number of people using this mean of communication is growing every year. Better and long lasting pavements are needed to meet the present requirements for transport infrastructure (Fontul, 2004).

As the road traffic volumes and vehicle loads are growing, the need for maintaining the pavement`s characteristics are also growing. More efficient methods for pavement monitoring and structural evaluation are required in order to ensure a good serviceability and to provide adequate maintenance solutions for the pavements.

The structural condition is one of the main factors to be taken into consideration for pavement maintenance planning. In order to evaluate the bearing capacity of a pavement, two solutions can be adopted, destructive tests (core drillings and pits) and non-destructive tests (NDT), such as Falling Weight Deflectometer (FWD) and Ground Penetrating Radar (GPR). Using the non-destructive tests, a mechanistic approach can be used, a structural model of the pavement is required for the estimation of its residual life. Using layer thickness data as input, the elasticity moduli of the pavement`s layers are "back calculated" from the deflection basin measured with non-destructive load testing equipment. In this way, the pavement bearing capacity is evaluated, and the remaining pavement life can be estimated, taking into consideration the future traffic (Irwin, 2002) (Fontul, 2004).

With the drastic increase in traffic over the last years, the stationary or slow moving and destructive tests procedures have become, not only dangerous for operators and difficult to perform but also with a significant impact on the traffic flow, mainly on road pavements.

It is a general concern nowadays to provide means that can improve safety of road workers and users, during testing and road maintenance works. For this purpose, the GPR and the FWD combined can provide a more accurate picture of the pavement`s structural condition, without conditioning the safety or the traffic or delaying the airport operation.

1.2 Objectives

The main purpose of this work is to show the importance of all steps in order to evaluate properly the pavements of an airport, not only the runway.

This study incorporates two main evaluation methods, functional and structural. The structural evaluation regards the use of the GPR and the use of the FWD with the corresponding interpretation of the results. The interpretation of FWD data together with layer thickness data obtained from GPR contributes to the improvement of the methodology for structural pavement evaluation.

As for the functional evaluation, the use of the GRIP Tester is a very important test, performed to find the friction coefficient of the runway. The safety of the aircrafts depends on this functional characteristic. Also the “sand patch” method used to find the depth texture of the surface layer of the runway is critical, as influences the safety of the aircrafts, the operation costs, the comfort, as well as the ambient is a very important evaluation characteristic.

An improved methodology for pavement evaluation represents an important tool for maintenance and rehabilitation of pavements.

1.3 Outline of the dissertation

The dissertation is organised in 7 chapters, including the Introduction presented in Chapter 1 and the Conclusions presented in Chapter 7.

Chapter 2 presents the main aspects to follow when an airport evaluation is required, as well as the main reasons for doing that. Starting with the classification of the pavements and continuing with the design considerations, the reader can make an idea of what is an evaluation and why is necessary.

Chapter 3 presents the functional characteristics and an example of determining them. The visual inspection is incorporated here; including the use of the GRIP tester for finding the friction coefficient of the surface layer as well as finding the depth texture using the “sand patch” method is shown in an example.

Chapter 4 consists of a state-of the art review of techniques for structural pavement evaluation of the flexible pavement (runway) using mechanistic approach, with special attention given to the procedures and interpretation of the non-destructive tests. These tests are the ones obtained using the GPR and the FWD equipment. Considerations are made on the influence of external factors such as the air and pavement temperature. The methodology for dividing the pavement into homogeneous sub-sections is mentioned here as well as the procedures for selection of a structural model for pavement evaluation.

The rigid pavement evaluation is addressed in Chapter 5. The respective tests are made in order to find a structural model for the pavement evaluation. The GPR and the FWD are also used for the characterization of the rigid pavements. The load transfer efficiency between the concrete slabs is calculated in this chapter.

In the next chapter, Chapter 6, the classification ACN-PCN is presented. All the steps in order to classify the runway are presented in detail in this chapter. The ACN-PCN methodology, the ACN classification as well as the PCN classification are carefully explained. After the ACN-PCN classification is done, a structural life is calculated, in order to find a remaining life of the runway. Conclusions are presented in the end of this chapter (AC:150/5320-6E, 2009)

2 Airport pavement evaluation

2.1 Airport Pavements – Functions and purposes

Airport pavements are constructed to provide adequate support for the loads imposed by airplanes and to produce a firm, stable, smooth, all-year, all-weather surface free of debris or other particles that may be blown or picked up by propeller wash or jet blast. In order to satisfactorily fulfil these requirements, the pavement must be of such quality and thickness that it will not fail under the load imposed. In addition, it must possess sufficient inherent stability to withstand, without damage, the abrasive action of traffic, adverse weather conditions, and other deteriorating influences. To produce such pavements requires a coordination of many factors of design, construction, and inspection to assure the best possible combination of available materials and a high standard of workmanship (AC:150/5320-6E, 2009)

These pavements can be flexible, rigid, or semi rigid (composite), depending on the type of materials that constitute them. The most important factor is the economic analysis along the life cycle, taking into account the traffic and climate. Besides the economical factor, there can be other factors like, operational constraints, funding limitation or future expansion.

The main materials of the different pavement layer are briefly described herein:

Wearing course

The materials that constitute the wearing surface courses include Portland cement concrete (PCC) and hot mix asphalt (HMA). Generally, the sand-bituminous mixture and bituminous surface treatments are forbidden for airport pavements due to the risk of debris.

Base course

Base courses consist of a variety of different materials, which generally fall into two main classes, treated and untreated. An untreated base consists of crushed or uncrushed aggregates.

A treated base normally consists of a crushed or uncrushed aggregate mixed with a stabilizer such as cement, bitumen, etc.

Subgrade course

Subgrade courses consist of granular material, stabilized granular material, or stabilized soil (AC:150/5320-6E, 2009).

2.2 Soil investigation and evaluation

Soil strength tests

Soil classification for engineering purposes provides an indication of the expected behaviour of the soil as a pavement subgrade. This indication of behaviour is, however, approximate. Performance different from that expected can occur due to a variety of reasons such as degree of compaction, degree of saturation, height of over layers, etc. The possibility of incorrectly predicting subgrade behaviour can be significantly reduced by measuring the soil strength. The strength of materials intended for use in flexible pavement substructures is measured by the CBR tests. Materials intended for use in rigid pavement structures are tested by the plate bearing method. Each of these tests is discussed below. Resilient modulus is used for rigid pavement design because of the variable stress states. Elastic modulus is estimated from CBR and k using the correlations E = 1500 x CBR and $E = 26 \times k^{1.284}$ (AC:150/5320-6E, 2009).

California Bearing Ratio (CBR)

The CBR test is basically a penetration test conducted at a uniform rate of strain. The force required to produce a given penetration in the material under test is compared to the force required to produce the same penetration in a standard crushed limestone. The result is expressed as a ratio of the two forces. Thus a material with a CBR value of 15 means the material in question offers 15 per cent of the resistance to penetration that the standard crushed stone offers. Laboratory CBR tests should be performed in accordance with ASTM D 1883, Bearing Ratio of Laboratory-Compacted Soils. Field CBR tests should be conducted in accordance with the ASTM D 4429, Standard Test Method for Bearing Ratio of Soils in Place (AC:150/5320-6E, 2009).

This method is generally used in order to determine the foundation class of flexible pavements, information needed for ACN/PCN classification (Fontul and Antunes, 2006).

Plate Bearing Test

As the name indicates the plate bearing test measures the bearing capacity of the pavement foundation. It is a test that it is performed in site (*in situ*). The result, *k* value, can be envisioned as the pressure required to produce a unit deflection of the pavement foundation. The plate bearing test result, *k* value, has the units of Mega-Newton per cubic meter. Plate bearing tests should be performed in accordance with the procedures contained in AASHTO T 222. For the ACN/PCN values determination for rigid pavements it is used to classify the subgrade category (AC:150/5320-6E, 2009).

Table 2.1 shows the pertinent types of soil suitable as foundations soils.

Table 2.1 – Soil characteristics pertinent to pavement foundations

Major Divisions		Letter	Name	Value as Foundation When Not Subject to Frost Action	Value as Base Directly under Wearing Surface	Potential Frost Action	Compressibility and Expansion	Drainage Characteristic	Unit Dry Weight (pcf)	CBR	Subgrade Modulus k (pci)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Coarse-gravelly soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	125-140	60-80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good	Poor to fair	None to very slight	Almost none	Excellent	120-130	35-60	300 or more
		GU	Gravel or sandy gravel, uniformly graded	Good to excellent	Poor	None to very slight	Almost none	Excellent	115-125	25-50	300 or more
		GM	Silty gravel or silty sandy gravel	Good	Fair to good	Slight to medium	Very slight	Fair to poor	130-145	40-80	300 or more
		GC	Clayey gravel or clayey sandy gravel	Good to excellent	Poor	Slight to medium	Slight	Poor to practically impervious	120-140	20-40	200-300
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor to not suitable	None to very slight	Almost none	Excellent	110-130	20-40	200-300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	105-120	15-25	200-300
		SU	Sand or gravelly sand, Poor uniformly Not suitablegraded	Fair to good	Poor	None to very slight	Almost none	Excellent	100-115	10-20	200-300
		SM	Silty sand or silty gravelly sand	Good	Not suitable	Slight to high	Very slight	Fair to poor	120-135	20-40	200-300
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	105-130	10-20	200-300
Fine grained soils	Low compress ibility LL<50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to good	Not suitable	Medium to very high	Slight to medium	Fair to poor	100-125	5-15	100-200
		CL	Lean clays, sandy clays, or gravelly clays	Fair to good	Not suitable	Medium to very high	Medium	Practically impervious	100-125	5-15	100-200
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to very high	Medium to high	Poor	90-105	4-8	100-200
	High compress ibility LL<50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	80-100	4-8	100-200
		CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	90-110	3-5	50-100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	80-105	3-5	50-100
Peat and other fibrous organic soils		Pt	Peat, humus and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	-	-	-

2.3 Design considerations. FAARFIELD software presentation

The main objective of this work is the airfield evaluation. But, in order to understand the evaluation, which is essentially the reverse of the design, here are the main ideas of an airfield`s design.

The design of airport pavements is a complex engineering problem that involves a large number of interacting variables.

The design method of an airfield pavement is computationally intense, so the FAA developed a computer program called FAARFIELD (Federal Aviation Administration Rigid and Flexible Iterative Elastic Layered Design) to help pavement engineers implement it.

FAARFIELD is a software free to use.
The design procedure provides a method of design based on layered elastic and three-dimensional finite element-based structural analysis developed to calculate design thicknesses for airfield pavements. Layered elastic and three-dimensional finite element-based design theories were adopted to address the impact of new complex gear and wheel arrangements.

Details on the development of the FAA method of design are as follows:

a. Flexible Pavements

For flexible pavement design, FAARFIELD uses the maximum vertical strain at the top of the subgrade and the maximum horizontal strain at the bottom of the asphalt layer for the predictors of pavement structural life. FAARFIELD provides the required thickness for all individual layers of flexible pavement (surface, base, and subgrade) needed to support a given airplanes traffic over a particular subgrade (ICAO, 1999).

b. Rigid Pavements

For rigid pavement design, FAARFIELD uses the maximum horizontal stress at the bottom of the PCC slab for the predictor of pavement structural life. The maximum horizontal stress for design is determined using an edge loading condition. FAARFIELD provides the required thickness of the rigid pavement slab needed to support a given airplane traffic mix (combination) over a particular subgrade.

An airfield pavement and the airplanes that operate on it represent an interactive system that must be studied in the pavement design process. Design considerations associated with both the airplanes and the pavement must be recognized in order to produce a satisfactory design. Producing a pavement that will achieve the intended design life will require careful construction control and proper maintenance. Pavements are designed to provide a finite life and fatigue limits are anticipated. Poor construction and a lack of maintenance reduce the service life of even the best-designed pavement (AC:150/5320-6E, 2009).

FAARFIELD is based on the cumulative damage factor (CDF) concept, in which the contribution of each airplane in a given traffic mix to total damage is separately analysed. Therefore, the FAARFIELD program should not be used to compare individual airplane pavement thickness requirements with the design methods contained in previous versions of the AC that are based on the “design aircraft” concept. Likewise, due care should be used when using FAARFIELD to evaluate pavement structures originally designed with the thickness design curves in previous versions of this AC. Any comparison between FAARFIELD and the design curve methodology from previous versions of this AC must be performed using the entire traffic mix.

Airplane Traffic Mixture

FAARFIELD was developed and calibrated specifically to produce pavement thickness designs consistent with previous methods based on a mixture (combination) of different airplanes rather than an individual airplane. If a single airplane is used for design, a warning will appear in the Airplane Window indicating a non-standard airplane list is used in the design. This warning is intended to alert the user that the program was intended for use with a mixture of different airplane types. Nearly any traffic mix can be developed from the airplanes in the program library. Solution times are a function of the number of airplanes in the mix. The FAARFIELD design procedure deals with mixed traffic differently than did previous design methods. Determination of a design aircraft is not required to operate FAARFIELD. Instead, the program calculates the damaging effects of each airplane in the traffic mix. The damaging effects of all airplanes are summed in accordance with Miner’s law. When the cumulative damage factor (CDF) sums to a value of 1.0, the design conditions have been satisfied (AC:150/5320-6E, 2009).

Cumulative Damage Factor

In FAARFIELD, the “design aircraft” concept has been replaced by design for fatigue failure expressed in terms of a cumulative damage factor (CDF) using Miner’s rule, CDF is the amount of the structural fatigue life of a pavement that has been used up. It is expressed as the ratio of applied load repetitions to allowable load repetitions to failure. For a single airplane and constant annual departures, CDF is expressed as:

$$CDF = \frac{number\ of\ applied\ load\ repetitions}{number\ of\ allowable\ repetitions\ to\ failure}$$

Or,

$$CDF = \frac{(annual\ departures) x\ (life\ in\ years)}{\left({}^{pass}/_{coverage\ ratio}\right) x\ (coverages\ to\ failure)}$$

Or,

$$CDF = \frac{applied\ coverages}{coverages\ to\ failure}$$

Table 2.2 – CDF classification

CDF value	Pavement remaining life
1	The pavement has used up all its fatigue life
<1	The pavement has some life remaining, and the value of CDF gives the fraction of the life used
>1	The pavement has exceeded its fatigue life

In the program implementation, CDF is calculated for each 254 mm wide strip along the pavement over a total width of 20 828 mm. Pass-to-coverage ratio is computed for each strip based on a normally distributed airplane wander pattern with standard deviation of 773 mm (equivalent to airplane operation on a taxiway) and used in the above equation for Miner’s rule.

The CDF for design is taken to be the maximum over all 82 strips. Even with the same gear geometry, therefore, airplanes with different main gear track widths will have different pass-to-coverage ratios in each of the 254 mm strips and may show little cumulative effect on the maximum CDF (AC:150/5320-6E, 2009).

Removing the airplanes with the lowest stress or strain may then have little effect on the design thickness, depending on how close the gear tracks are to each other and the number of departures.

Pass-to-Coverage Ratio

As an airplane moves along a pavement section it seldom travels in a perfectly straight path or along the exact same path as before. This lateral movement is known as airplane wander and is modelled by a statistically normal distribution. As an airplane moves along a taxiway or runway, it may take several trips or passes along the pavement for a specific point on the pavement to receive a full-load application. The ratio of the number of passes required to apply one full load application to a unit area of the pavement is expressed by the pass-to-coverage (P/C) ratio. It is easy to observe the number of passes an airplane may make on a given pavement, but the number of coverages must be mathematically derived based upon the established P/C ratio for each airplane. By definition, one coverage occurs when a unit area of the pavement experiences the maximum response (stress for rigid pavement, strain for flexible pavement) induced by a given airplane. For flexible pavements, coverages are a measure of the number of repetitions of the maximum strain occurring at the top of subgrade.

For rigid pavements, coverages are a measure of repetitions of the maximum stress occurring at the bottom of the PCC layer.

Coverages resulting from operations of a particular airplane type are a function of the number of airplane passes, the number and spacing of wheels on the airplane main landing gear, the width of the tire-contact area and the lateral distribution of the wheel-paths relative to the pavement centreline or guideline markings.

In calculating the P/C ratio, FAARFIELD uses the concept of effective tire width. For rigid pavements, the effective tire width is defined at the surface of the pavement and is equal to a nominal tire contact patch width. For flexible pavements, for the failure mode of shear in the subgrade layer, the effective tire width is defined at the top of the subgrade. “Response lines” are drawn at 1:2 slope from the edges of the contact patches to the top of the subgrade. Tires are considered to be either separate or combined, depending on whether the response lines overlap. All effective tire width and P/C ratio calculations are performed internally within the FAARFIELD program.

Annual Departures and Traffic Cycles

Airport pavement design using FAARFIELD considers only departures and ignores the arrival traffic when determining the number of airplane passes. This is because in most cases airplanes arrive at an airport at a significantly lower weight than at take-off due to fuel consumption.

During touchdown impact, remaining lift on the wings further alleviates the dynamic vertical force that is actually transmitted to the pavement through the landing gears.

The FAA has defined a standard traffic cycle (TC) as one take-off and one landing of the same airplane. In the situation described above, one traffic cycle produces one pass of the airplane which results in a pass-to-traffic cycle ratio (P/TC (Alves, 2007)) of 1.

To determine annual departures for pavement design purposes multiply the number of departing airplanes by the P/TC. For most airport pavement design purposes, a P/TC of 1 may be used.

2.4 Purposes of pavement evaluation

Airport pavement evaluation is necessary to assess the ability of an existing pavement to support different types, weights or volumes of airplane traffic. The load carrying capacity of existing bridges, culverts, rain drains, and other structures should also be considered in these evaluations.

Evaluations may be also necessary to determine the condition of existing pavements for use in the planning or design of improvements to the airport. Evaluation procedures are essentially the reverse of design procedures, so called back-calculation.

Evaluation process

The evaluation of airport pavements should be a methodical step-by-step process. The recommended steps in the evaluation process described below should be used regardless of the type of pavement.

a. Records Research

A thorough review of construction data and history, design considerations, specifications, testing methods and results, as-built drawings, and maintenance history should be performed. Weather records and the most complete traffic history available are also parts of a usable records file.

b. Site Inspection. Functional evaluation

The site in question should be visited and the condition of the pavements noted by visual inspection. This should include, in addition to the inspection of the pavements, an examination of the existing drainage conditions and drainage structures at the site. Evidence of the adverse effects of frost action, swelling soils, reactive aggregates, etc., should also be noted.

Beside the visual inspection, there are evaluated the following surface characteristics of the pavement:

1. Friction coefficient
2. Longitudinal regularity
3. Transversal regularity
4. Depth of texture

The main function of a pavement is to create a free and plane surface, designed for aircraft traffic in adequate safety, economy and comfort conditions. Therefore, the pavement surface must have certain characteristics, such as geometrical regularity, adherence and the capacity to drain surface waters. These characteristics, that affect directly the user of the pavement, are called functional characteristics. Also, more and more importance is given to the environmental issues that affect not only the user but also the surrounding, such as: mitigate the traffic noise and the landscape aspects.

1. Friction coefficient

The safety depends on the geometrical characteristics of the pavement, as well as on the friction coefficient between the tire and the pavement. The friction coefficient is affected by a number of factors such as the texture depth (ASTM 1996) and the drainage, which in its turn is directly related with the geometrical characteristics, like the longitudinal and transversal profile of the pavement. The longitudinal irregularity has a great influence on the user`s comfort and safety.

2. Longitudinal irregularity

The longitudinal irregularity of a pavement can be defined by the variation in depth of its surface, compared to an ideal profile. The wavelengths associated to the longitudinal irregularity are generally comprised between 0.5 m and 40 m. The vehicle driving conditions are more affected by the longitudinal irregularity as the vehicle velocity is higher.

3. Transversal irregularity

It is an essential characteristic used to ensure a good performance of the road. It affects the conditions of comfort, safety and it is a degradation factor when the surface of the pavement is wet. This parameter is, in general, a good indicator of the superficial degradation of the pavement.

4. Depth of texture

The texture depth of the surface layer of a runway plays a decisive role for its functional quality, as it is related to the following aspects: development of friction forces at the tire/pavement contact in adverse conditions – wet surface, resistance to the aircraft motion (high consumption of fuel), wear of the tires at the contact tire/pavement, low frequency noise (inside and outside of the aircraft).

Thus, the texture depth influences the safety of the aircrafts, the operation costs, the comfort, as well as the ambient, making it a very important evaluation characteristic (Alves, 2007)

The texture of a pavement is determined by its superficial irregularities, which goes from the finer details of the micro texture, through the particularities of macro texture, until the largest undulations of the mega texture.

The distinction between various areas or texture scales is in function of the considered wavelength. In the next table is shown the approximate texture size range for pavements.

Table 2.3 – Texture size range

Designation	Approximate dimensions range	
	Wavelength	Amplitudes
Micro texture	0 - 0.5 mm	0 - 0.2 mm
Macro texture	0.5 - 50 mm	0.2 - 10 mm
Mega texture	50 -500 mm	1.0 - 50 mm

c. Sampling and Testing. Structural evaluation

The need of physical test and materials analyses is based on the findings made from the site inspection, records research, and type of evaluation. Herein are presented the main methods of evaluating the runway through sampling and testing.

1. Direct Sampling Procedures

The basic evaluation procedure for planning and design are visual inspection and reference to the FAA design criteria, supplemented by the additional sampling, testing and research.

2. Non-destructive Testing

Several methods of non-destructive testing (NDT) of pavements are available. For purposes of this work, NDT means observing pavement response to a controlled dynamic load, as in the case of the Falling Weight Deflectometer (FWD), or other physical stimulus such as a mechanical wave. NDT provides a means of evaluating pavements that tends to remove some of the subjective judgment used during empirical evaluation procedures.

The main advantages of non-destructive testing are: the pavement is tested in place under actual conditions of moisture, density, etc.; the disruption of traffic is minimal; and the need for destructive tests is minimized. Research efforts are on-going in the area of non-destructive testing to broaden its application.

The common NDT tools available for pavement evaluation include: FWD, Ground Penetrating Radar (GPR), infrared thermography, which are presented herein (AC:150/5320-6E, 2009) :

a. Falling Weight Deflectometer (FWD)

The FWD was first build in France in the early 60`s, but its development was interrupted due to difficulties in achieving adequate deflection measurements at that time. Presently, the most used models are produced by DYNATEST, CARL-BRO (Denmark) and KUAB (Sweden).

In the earlier versions, the load pulse rose difficulties due to the internal oscillation of the spring system and its sensibility to the pavement`s deflection effects. The “spring” systems in the current machines consist of a set of rubber buffers, whose characteristics were designed to minimize these effects (COST 2002) (Fontul, 2004).

This system provides not only a drastic reduction of internal oscillation but also linearity between the peak force and the pavement`s deflection (Tholen, 1980).

The load pulse generated by the FWD during testing is different for each drop. The peak load values are not very different from the target value. However, in order to compare the results obtained in different locations, it is necessary to transform, through a simple mathematic operation, the deflections measured into “normalised” deflections corresponding to the target load. This process is called “normalisation”. In this way, the results in different test points can be compared and statistically analysed (Fontul, 2004).

Falling Weight Deflectometer apply an impulse load to the pavement with a free-falling weight. The magnitude of the dynamic load depends on the mass of the weight and the height from which it is dropped. The resulting deflections of the pavement surface are typically measured using an array of sensors. The Heavy Falling Weight Deflectometer (HWD) uses a greater dynamic load than FWD and may be more suitable for some airport applications. FWD and HWD can be used in conjunction with appropriate software to estimate pavement layer properties. AC 150/5370-11 gives guidance for the use of FWD and HWD equipment (AC:150/5320-6E, 2009).

The main issues taken into account in order to ensure the measurement accuracy are the stability of the signal to environmental effects and to pulse duration. Even more important is to be sure that the maximum value of the deflection is picked and recorded.

In the later versions of FWD (see Figure 2.1), the deflection measuring system is isolated, as much as possible, from the loading system, in order to avoid the influence of the dropping weight in the deflections measured.

There are two main types of deflection transducers used in the current FWD devices (Sorensen, 2004):

- Geophones (seismic velocity transducers), which measure velocities of the pavement`s surface and convert them into deflections, by integrating the signal;
- Seismometers (seismic displacement transducers), which measure directly the deflections of the pavement`s surface.

The measured values of the load and deflections are automatically recorded for each impact. Using a laptop, the user can control the load level, drop sequence, distance between testing points, etc. The output files are easy to import to Excel for processing. Although there are FWDs installed on dedicated vehicles, the majority are still mounted on trailers. (Fontul, 2004).

LNEC has a “KUAB 150” FWD since the early 80`s and a Carl Bro Pri 2100 since 2007. The deflection transducers` location can be modified, for a better adaptability to the condition of the pavement section under study.

b. Ground Penetrating Radar

Ground penetrating radar can be useful in studying subsurface conditions non-destructively. Ground penetrating radar depends on differences in dielectric constants to discriminate between materials. The technique is sometimes used to locate voids or foreign objects, such as, abandoned fuel tanks, tree stumps, etc. (AC:150/5320-6E, 2009).

GPR is non-destructive equipment; performing continuous assessment of pavement structure and giving information about layer thickness and structure changes.

The GPR was developed in the late 1920`s , by the military, for use in detecting subsurface non-metallic mines, although successful measurements applied to earth science problems were performed only in the late 1950`s. Geotechnical applications of ground penetrating radar to rock and soil did not occur until 1970`s (Ulriksen, 1982).

There are several GPR manufacturers, such as Geophysical Survey System Inc. (*GSSI*), Pulse Radar Inc. (*Pulse Radar*), Penetradar Corporation (*IRIS*) [U.S.A], Road Radar Inc., Sensor & Software Inc. (*Pulse EKKO*) [Canada], Auscult` (*EURADAR*, *Scanroad*) [France].

Each equipment has its own software for data processing and there are also soft wares developed by GPR users, such as ROADDOCTOR (Finland) (FORMAT 2004).

LNEC`s equipment has two pairs of air-launched antennas (1000 MHz and 1800 MHz). In the same figure (right hand side picture) four air-coupled antennas are seen suspended above the pavement (Fontul and Antunes, 2000) (Fontul, 2004). The two in line antennas on the right side of the trailer are a pair of transmitter-receiver having a frequency of 1 GHz (1000 MHz), while those on the left side are another pair of antennas of 1.8 GHz (1800 MHz).

The actual GPR system of LNEC is shown in the figure 2.1 :

Figure 2.1 – LNEC`s FWD (on the left) and GPR (on the right) equipment

c. Infrared Thermography

Infrared thermography is a non-destructive testing procedure whereby differences in infrared emissions are observed allowing certain physical properties of the pavement to be determined. Infrared thermography is purportedly capable of detecting delamination in bonded rigid overlay pavements and in reinforced rigid pavements.

3. Pavement Condition Index

The determination of the Pavement Condition Index (PCI) is often a useful tool in the evaluation of airport pavements. The PCI is a numerical rating of the surface condition of a pavement and is a measure of functional performance with implications of structural performance. PCI values range from 100 for a pavement with no defects to 0 for a pavement with no remaining functional life. The index is useful in describing distress and comparing pavements on an equal basis.

4. Evaluation Report

The analyses, findings, and test results should be incorporated in an evaluation report, which becomes a permanent record for future reference. While evaluation reports need not be in any particular form, it is recommended as a drawing identifying limit of the evaluation is included. Analysis of information gained in the above steps should culminate in the assignment of load carrying capacity to the pavement sections under consideration. When soil, moisture, and weather conditions conductive to detrimental frost action exist, an adjustment to the evaluation may be required (AC:150/5320-6E, 2009).

2.4.1 Flexible pavement evaluation

Evaluation of flexible pavements requires, as a minimum, the determination of the thickness of the component layers, and the CBR of the subgrade.

a. Layer Thicknesses

The thickness of the various layers in the flexible pavement structure must be known in order to evaluate the pavement. Thicknesses may be determined from borings, test pits or NDT. As-built drawings and records can also be used to determine thicknesses if the records are sufficiently complete and accurate.

b. Subgrade CBR

Laboratory CBR tests should be performed on soaked specimens in accordance with ASTM D 1883, Bearing Ratio of Laboratory-Compacted Soils. Field CBRs should be performed in accordance with the procedure given in The Asphalt Institute Manual Series 10 (MS-10), Soils Manual. Field CBR tests on existing pavements less than 3 years old may not be representative unless the subgrade moisture content has stabilized.

The evaluation process assumes a soaked CBR is and will not give reliable results if the subgrade moisture content has not reached the ultimate in situ condition.

In situations where it is impractical to perform laboratory or field CBR tests, a back calculated subgrade elastic modulus value may be obtained from NDT test results. The FAARFIELD program assumes that CBR is related to the subgrade modulus as E = 1500xCBR (E in psi), so that the back calculated modulus value can be input directly into FAARFIELD without manually converting to CBR.

2.4.2 Rigid pavement evaluation

Evaluation of rigid pavements requires, as a minimum, the determination of the thickness of the component layers, the flexural strength of the concrete, and the subgrade modulus.

a. Layer Thicknesses

The thickness of the component layers is sometimes available from construction records. Where information is not available or of questionable accuracy, thicknesses may be determined by borings or test pits in the pavement.

b. Concrete Flexural Strength

The flexural strength of the concrete is most accurately determined from test beams extracted from the existing pavement and tested. Quite often this method is impractical as extracted beams are expensive to obtain and costs incurred in obtaining sufficient numbers of beams to establish a representative sample is prohibitive.
Construction records, if available, may be used as a source of concrete flexural strength data. The construction data may require adjustment due to the age of the concrete.

c. Subgrade Modulus

The modulus of subgrade reaction, *k*, is ideally determined by plate bearing tests performed on the subgrade. These tests should be made in accordance with the procedures established in AASHTO T 222. An important part of the test procedure for determining the subgrade reaction modulus is the correction for soil saturation, which is contained in the prescribed standard.
The normal application utilizes a correction factor determined by the consolidation testing of samples at *in situ* and saturated moisture content. For evaluation of older pavement, where evidence exists that the subgrade moisture has stabilized or varies through a limited range, the correction for saturation is not necessary.
If a field plate bearing test is not practical, a back calculated subgrade elastic modulus value may be obtained from NDT test results.

2.5 Synopsis

The soil investigation and evaluation is a very important part of the runway evaluation, as we seen in this chapter 2.2. The purpose of this evaluation is very clearly stated in 2.4, as this evaluation is enabling a classification in function of the load carrying capacity.
In order to effectuate a thoroughly pavement evaluation, the following steps must be followed:

- Records research;
- Site inspection;
- Sampling and testing.

3 Functional evaluation

3.1 General presentation of the case study

The pavements studied are the ones shown in the Figure 3.1:

- The runway;
- The rigid aprons: Charlie R., Delta 1, Delta 2, Echo 1, Echo 2.

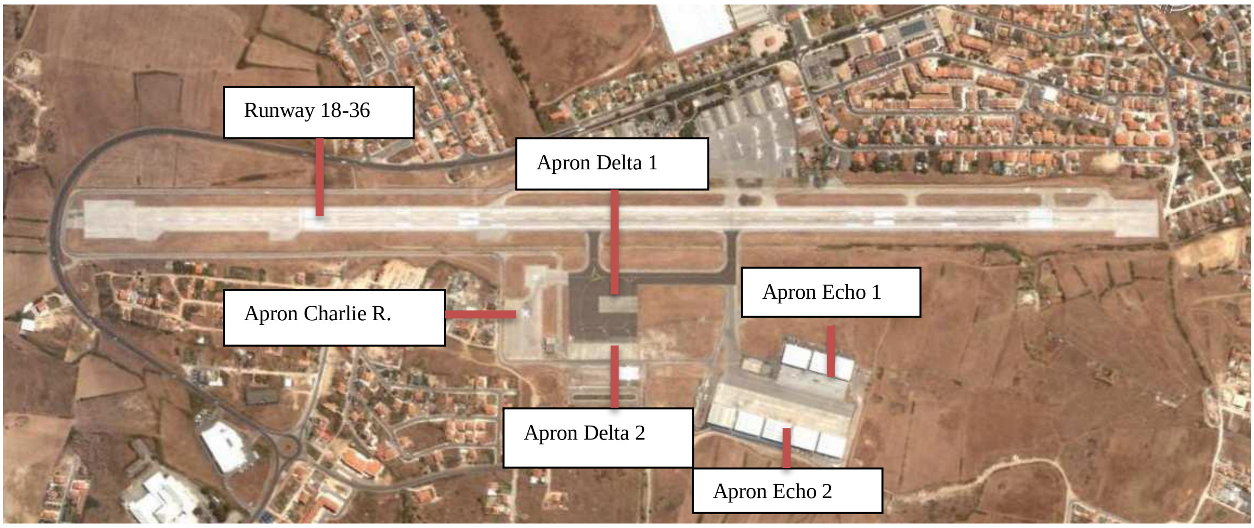

Figure 3.1 – Location of the runway and the main aprons

The methodology used for the study of superficial pavements characteristics evaluation consisted of:

- visual inspection of the superficial layer:
- measuring the continuous friction coefficient, with GRIP TESTER (FAA 1997);
- measuring the texture depth of the superficial layer with the “sand patch” test method;
- evaluating the state of the superficial layer of the runway based on the obtained results, applying the recommendations of the International Civil Aviation Organization (ICAO) and Federal Aviation Administration (FAA) for the active runways.

3.2 Brief description of the flexible runway

The runway with an orientation of 18-36 has the following geometrical dimensions:

- Total length: 1700 m;
- length between thresholds 1190 m;
- width 30 m;
- width of thresholds 2.5 m.

3.3 Visual inspection of the runway

The visual inspection was done in October and was found in good conditions. The next problems were found:

- The existence of slight disintegration phenomena usually associated with finishing deficiencies in some of the construction joints.
- Rubber deposits on the surface layer; this phenomenon was not creating any difficulties because the deposits are not significant

Figure 3.2 - Rubber deposits on the contact area near the threshold 36

The anomalies found do not interfere with the runways functionality. During the visual inspection, effectuated after a rainy period, was possible to note that the superficial layer has some water accumulation areas because of the superficial irregularity. This phenomenon was already noted in the previous visual inspection, performed 4 years before.

3.4 Friction coefficient measurement

3.4.1 Methodology

The survey for measuring the friction coefficient was done in October, during the day, using the "Grip-Tester" equipment from LNEC. This equipment allows the friction coefficient measurement between the runway and an especial tire, that is, partial blocked. To allow a standard conditions test, the equipment has a watering system, which distributes a water flow directly in the front of the measuring wheel. The friction coefficient it is measured from 10 to 10 m, on the entire length of the runway, along several parallel alignments.
The survey was done in 12 alignments parallel to the axis, at a distance of 1.5; 3.0; 4.5; 6.0; 9.0 and 10.0 m on each side. The survey conditions were in conformity with the ones specified by ICAO, as follows:

- test speed: 65 km/h;
- wet surface simulation with 1 mm of water.

For these conditions, ICAO recommends the following limit values of the friction coefficient:

- new runways 0.74;
- planning threshold of rehabilitation measures for operating runways 0.53;
- minimum acceptable limit for operating runways 0.43.

3.4.2 Results

For each alignment, the friction coefficient was registered for sections of 10 m.

According to ICAO, for a proper analysis of the friction coefficient of airport runways, they shall be divided in three sections, two landing zones and one central zone. Thus, the following areas were considered:

- Zone A, from 0 to 400 m starting from threshold 18, corresponding to the landing area beside 18;
- Zone B, from 400 to 800 m starting from threshold 18, corresponding to the central landing area;
- Zone C, from 800 to 1200 m starting from threshold 18, corresponding to the landing area beside 36.

The results revealed that the average values of the friction coefficient for 100 m are above the ICAO recommended value as threshold planning rehabilitation measures for active runways.

Table 3.1 - Average friction coefficient values by alignments and by zones

Zones	Alignment											
	12.0 m	9.0 m	6.0 m	4.5 m	3.0 m	1.5 m	1.5 m	3.0 m	4.5 m	6.0 m	9.0 m	12.0 m
A	0.82	0.67	0.74	0.83	0.77	0.78	0.77	0.72	0.77	0.67	0.67	0.76
B	0.84	0.85	0.84	0.83	0.77	0.8	0.74	0.77	0.86	0.79	0.78	0.82
C	0.84	0.81	0.8	0.76	0.7	0.75	0.77	0.71	0.84	0.76	0.76	0.7
Media	0.83	0.76	0.79	0.8	0.75	0.78	0.76	0.73	0.82	0.74	0.74	0.76
	Left side of the axis						Right side of the axis					

3.5 Measurement of runways texture depth

3.5.1 Methodology

The measurements were made along 4 alignments parallel to the axis, at 1.5 m and 3.0 m on both sides of the axis, at 100 m apart.

Although there are usually requirements for the texture depth values for paving works, ICAO does not establishes any recommendations whatsoever regarding active runways.

The FAA recommendations for the texture depth of active runways are as follows:

- Periodic measurements 0.76 to 1.14 mm;
- Scheduled rehabilitation measurement in one year 0.40 to 0.76 mm;

- Rehabilitation measurements in maximum two months under 0.25 mm.

3.5.2 Results

The depth texture obtained values are shown in the table below, for each alignment. Also the standard deviation of the values per alignment is shown in the table. From these results, we can note that the values of medium depth texture are between the planning rehabilitation measures range values (0.40 mm to 0.76mm) in conformity with FAA recommendations (1997). This can affect its drainage capacity in intense rain periods and favour the accumulation of water on the surface of the pavement.

Table 3.2 - Depth texture values obtained by "sand patch" method

RUNWAY 18-36				
Distance from threshold 17 (m)	Alignment			
	3.0 m left	1.5 m left	1.5 m right	3.0 m right
0		0.6		0.6
50	0.33		0.53	
100		0.45		0.55
150	0.55		0.47	
200		0.6		0.6
250	0.66		0.66	
300		0.6		0.58
350	0.55		0.66	
400		0.72		0.51
450	0.51		0.63	
500		0.72		0.53
550	0.66		0.51	
600		0.88		0.63
650	0.58		0.66	
700		0.69		0.6
750	0.55		0.63	
800		0.6		0.51
850	0.53		0.63	
900		0.58		0.55
950	0.55		0.55	
1000		0.51		0.55
1050	0.55		0.58	
1100		0.58		0.6
1150	0.6		0.72	
1200		0.8		0.69

Average values	0.55	0.64	0.6	0.58
Standard deviation	0.08	0.12	0.07	0.04

3.6 Summary of results

This study analysed the visual inspection of the runway, the friction coefficient measurement and the measurement of runways texture depth. Summarizing the main results obtained in this study:

- The visual inspection of the runway, permitted to find that it is in a good general condition, presenting however some phenomena of surface breakdown, mainly along the construction joints. Still, there were verified the water accumulation on the runway after rain periods.
- The values obtained for the friction coefficient of the runway, measured with GRIP-TESTER, at a velocity of 65 km/h, with 1 mm of water, were superior to the planning threshold rehabilitation measures to active runways (>0.53), being generally above the recommended value of ICAO for new runways (>0.74).
- The values obtained for the depth texture are, in general, between 0.40 and 0.76 mm.

According to the criteria established by FAA, it is recommended the planning of rehabilitation measures when the depth texture lies in that interval. However, according to the same institution, when the friction coefficient is high, the measurement of the depth texture of runways can be dispensed. Thus, taking into account the results obtained of the friction coefficient of this runway, it is considered that the runway`s surface is in adequate conditions. It is recommended that visual inspection will be made after rain periods, in order to identify any water accumulation areas.

As a final conclusion, it can be stated that the runway 18-36 it has satisfactory anti-skip characteristics even if it presents a few deficiencies at surface drainage.

4 Structural evaluation of flexible pavements

4.1 Load tests using the Falling Weight Deflectometer

In this case was used the FWD (Falling Weight Deflectometer) in points from 50 to 50 m, in 5 alignments parallels with the centre line of the runway, including the centre line.

In each test point were made 3 impacts, that corresponds to a maximum force of 150 kN, using the load plate of 0.45 m diameter. During this process the air and soil temperatures have been recorded. The deflections from the last fall were recorded at 7 seismometers located as follows: D_0 centre of the load plate, D_1 at 0.30 m from the centre, D_2 at 0.45 m, D_3 at 0.60 m, D_4 at 0.90 m, D_5 at 1.20 m and D_6 at 1.80 m, respectively.

Example of using the program BISAR 3.0:

Zone: z1

Table 4.1 – Layer characteristics

Layer number	Thickness (m)	Modulus of Elasticity (MPa)	Poisson's ratio
1	0.19	5000.00	0.4
2	0.35	220.00	0.35
3	1.20	120.00	0.35
4		1000.00	0.35

Here are introduced in the program the characteristics of the layers: thickness, elasticity modulus and Poisson`s ratio.

Table 4.2 – Vertical load

Load number	Vertical load (kN)	Vertical Stress (MPa)	Horz. (Shear) Load (kN)	Horz. (Shear)		X-Coordinate	Y-Coordinate	Shear Angle (Degrees)
				Stress (MPa)	Radius (m)			
1	150	0.943	0.00	0.00	0.225	0.00	0.00	0.00

The load, 150 kN, is applied right in the centre of the plate (X, Y coordinates are 0, 0).

Table 4.3 – Top layer displacements

Position Number	Layer Number	X-Coordinate (m)	Y-Coordinate (m)	Depth (m)	Displacement UX (µm)	Displacement UY (µm)	Displacement UZ (µm)
1	1	0.00	0.00	0.00	0.00	0.00	690
2	1	0.00	0.300	0.00	0.00	-56.6	531
3	1	0.00	0.450	0.00	0.00	-57.0	426
4	1	0.00	0.600	0.00	0.00	-50.7	336
5	1	0.00	0.900	0.00	0.00	-34.7	202
6	1	0.00	1.20	0.00	0.00	-21.8	119
7	1	0.00	1.80	0.00	0.00	-74.3	40.6

Here are presented the values, resulted from the application of the load. The displacements of the top layer in UZ direction are conditioning.

The same steps are done for all zones of the runway so that a graphic of the deflections can be realized, and therefore see the real differences between the runway`s zones characteristics.

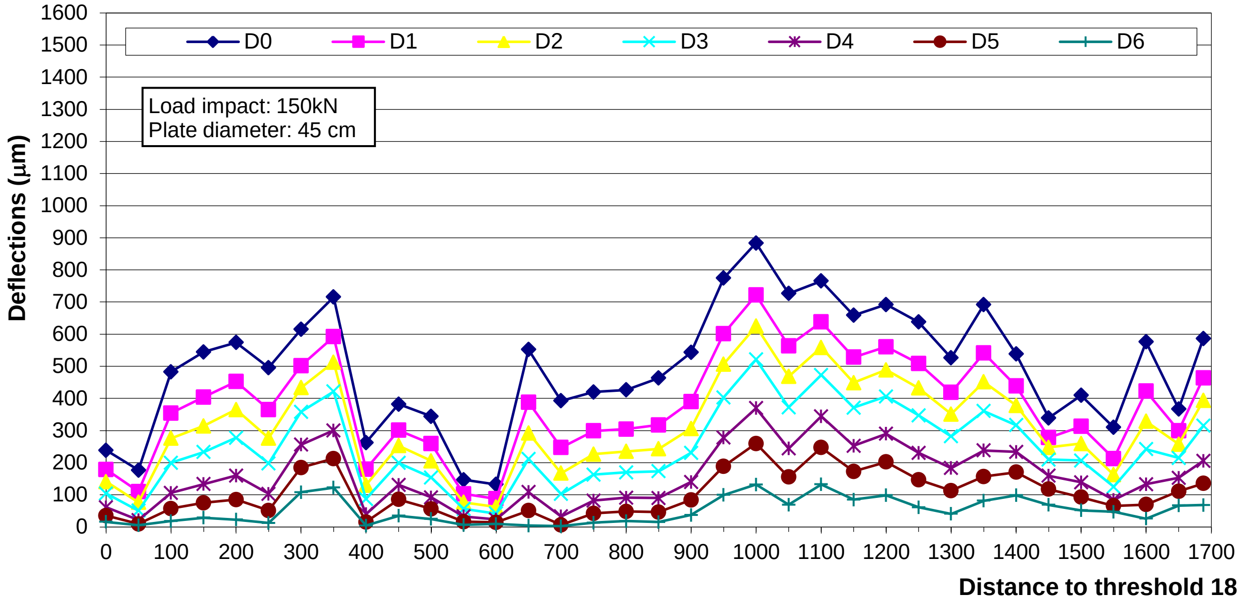

Figure 4.1 - Load test using the FWD - Measured deflections - Center Line Runway 18-36

4.2 Tests using the Ground Penetrating Radar

The radar was used to perform tests on the same path as the falling weight deflectometer. The sampling interval was 0.25 m, so that the obtained results will provide a continuous survey of the interfaces between layers. The tests were made with an antenna of 1 GHz, which due to its greater depth of penetration allowed detecting the granular layers thickness.

4.3 Division in subsections

Dividing a pavement into numerous subsections should take in consideration the following parameters:

- Surface distress
- Subgrade type, earthworks
- Drainage condition
- Layer thicknesses
- Traffic volumes
- Construction records

- Measured deflections and deflection bowl parameters
- Number of measuring points of the subsection.

Besides the above factors, other more elaborate parameters may be used for sub-division:

- Surface modulus plots
- Layer moduli
- Residual pavement life
- Overlay requirement, if the method used calculates the overlay needed at every test point of the road.

The division in subsections can be performed, either by engineering judgment using visual assessment of the variation of parameters or by statistical methods or by combination of both. "Visual assessment" is used for analysis of construction records, subgrade type, and drainage condition and traffic volumes. A "visual assessment" for other variables such as deflections and layer thickness is very useful when a statistical method is used, as a complement to this one. For large databases the visual assessment delineation can be time-consuming and confusing. In this case, there are statistical methods that can be used, allowing for a better interpretation of the variability of several parameters along the pavement (Fontul, 2004).

Cumulative difference method

Using this method we can divide our pavement into subsections. It is widely used for identification and delimitation of statistically homogeneous sub-sections along the pavement, and can be applied for a variety of pavement parameters or response variables such as: deflections, layer thickness, serviceability, surface distresses, etc. The cumulative difference slope changes whenever there is a change in pavements characteristics. Those points are called "borders" between two consecutive subsections. The division into subsections was made as follows:

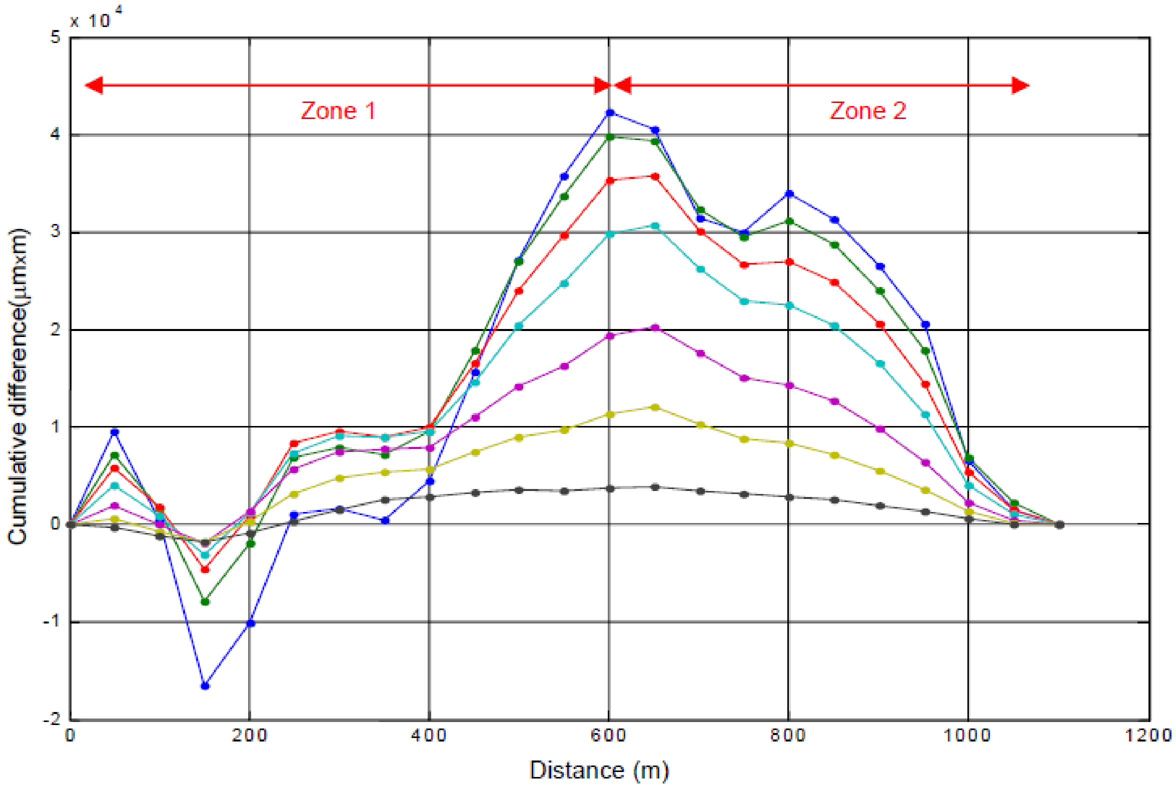

Figure 4.2 – Example of Cumulative Difference Method

Table 4.4 - Standard deviation and average values of deflections on the runway 18-36

Zone	Location	Normalized deflections for 150 kN (μm)													
		D0		D1		D2		D3		D4		D5		D6	
		M	σ	M	σ	M	σ	M	σ	M	σ	M	σ	M	σ
z1	80-250 m	504	155	372	133	290	114	211	92	113	57	58	32	17	11
z2	250-350 m e 900-1400 m	653	103	522	85	442	77	357	69	240	56	159	44	77	28
z3	350-600 m	275	102	200	81	156	69	114	55	65	36	38	25	16	13
z4	600-900 m	418	95	286	70	210	58	144	46	69	29	32	18	9	7
z5	1400-1630 m	400	89	304	54	246	38	192	32	125	31	85	29	49	19

Legend:

M – average value of deflections

σ - standard deviation

4.4 Asphalt Boring tests

In each zone of homogenous behaviour that have already been divided there have been made a boring test so that we can compare with the data collected with the GPR.

Table 4.5 - Bituminous layers thicknesses of the flexible pavement

Sampling	Location	Thickness of the bituminous layer (m)	Average value obtained with GPR (m)
P1	Pista 18-36	0,19	0,21
P2		0,19	0,2
P3		0,12	0,12
P4		0,165	0,17
P5		0,18	0,18

4.5 Defining the structural behaviour model

After obtaining the results with the FWD in each defined zone, there were selected a representative point for each one of them. The points that were approximately close to D_{85} were the points selected.

$D_{85} = M + 1.04 * \sigma$

Where:

M – deflection value

σ – standard deviation

The results of the FWD tests, allowed establishing a structural behaviour model of the runway in each representative zone.

In order to do that, using the methodology usually used in evaluating the bearing capacity of runways studies, proceeded to estimate the deformability layers modules. It was used the program BISAR 3.0 to calculate the deflections. The layers thicknesses used were in conformity with the results obtained with the ground penetrating radar and the test pits.

Regarding the layers Poisson ratio, were used typically values for the respective material: 0.40 for the asphaltic concrete layer and 0.35 for the other layers (Antunes, 1993). As usual, the foundation layer was divided in two layers: one upper layer, whose thickness was determined in function of the tests results, and a bottom layer, semi-infinite, which is called "rigid layer" (Fontul, 2004).

Table 4.6 – Structural behaviour model

Zone	BM			GM		FS		
	E1	T(°C)	h1	E2	h2	E3	h3	E4
z1	5000	19	0.19	220	0.35	120	1.2	1000
z2	5000	19	0.20	240	0.20	100	1.5	1000
z3	5700	19	0.20	400	0.35	180	0.5	1000
z4	5700	19	0.17	420	0.45	160	0.4	1000
z5	5400	19	0.19	420	0.27	160	0.8	1000

Where:

BM - layers of bituminous mixtures;

GM – layers of granular material;

FS – foundation soil;

T (°C) – recorded temperature in the bituminous layer, at 2.5 cm depth;

E_1, E_2, E_3, E_4 – deformability modulus (MPa) of pavement layers;

h_1, h_2, h_3 – layer thicknesses.

The bituminous layers moduli of each zone were corrected to take into account the project temperature of bituminous layers for the region where the airport is located. This temperature is calculated using the SHELL methodology based on the air`s monthly averages temperatures values published by the *Instituto Nacional de Meteorologia e Geofisica*, having obtained an annual air temperature in this region of 17°C. To this temperature corresponds a project temperature of the bituminous layers of 24°C.

The project values of the bituminous layers moduli were found using the ratio proposed in a study conducted in Lisbon airport (Antunes, 1993):

$$Et = (1.635 - 0.0317 * T) * E20$$

E_t – deformability modulus at temperature T (°C)

E_{20} – deformability modulus at temperature of 20 °C.

The following table shows the deformability modulus values of the bituminous layers, for the respective tests temperature, and the correspondent deformability modulus values for the project temperature.

Table 4.7 - Deformability modulus of bituminous layers corrected for the project temperature

Zone	FWD Tests		Project values	
	E1	T	T	E1
z1	5000	19	24	4230
z2	5000	19	24	4230
z3	5700	19	24	4820
z4	5700	19	24	4820
z5	5400	21	24	4870

5 Structural evaluation of rigid pavements

5.1 Brief description of the studied rigid pavements

The case study presents only platforms with rigid pavement, as the runway and taxiway consist of flexible pavement. For runways or taxiways the evaluation procedure is similar to the one presented herein for the apron, in terms of location of tests per slabs, namely centre and joints while the alignments tested are longitudinal to runway centre, similar to flexible pavements evaluation. In this case study there are three rigid pavements on aprons designated as “Delta”, “Echo” and “Charlie”. The rigid pavement of Charlie apron consists of concrete slabs with 4.60 m width and 4.25 m length, with a total area of 2557 m^2. According to the design data, the slab thickness is 0.15 m, with a base layer of 0.10 m thickness in lean concrete.

Delta apron is divided into two areas, with different rigid pavement constitutions, designated as Delta 1 and Delta 2.

The apron Delta 1 it consists of concrete slabs with 5.0 m width and a length that varies between 2.80 m and 5.30 m, with a total area of 60 m x 25 m. According to the design data, the pavement consists of concrete slabs with a thickness of 0.25 m, based on a subgrade on lean concrete with a thickness of 0.10 m. The Delta 2 apron it consists of concrete slabs of 5 m x 5 m, with a total area of 110 m x 25 m, having the same subgrade characteristics as Delta 1.

The Echo apron is also divided in two rigid pavement areas with different characteristics, designated as Echo 1 and Echo 2. The apron Echo 1 it consists of slabs of 3.7 m x 4.0 m, with a total area of 110 m x 34 m. Design data not available for this pavement.

The apron Echo 2 it consists of slabs of 3.8 m x 4.0 m, with a total area of 220 m x 32 m. According to the design data, the pavement is made of concrete slabs of a thickness of 0.15 m, based on a subgrade with a thickness of 0.10 m.

General state of runway surface

The aprons Delta 2 and Echo 2 are presenting cracking phenomena. Also, it was observed some joints with poor finishing and lack of sealing on the Charlie R. pavement, as well as, longitudinal cracks on Echo 1 and Delta 1 aprons.

5.2 Load tests using the Falling Weight Deflectometer

In each apron (Charlie R., Delta 1, Delta 2, Echo 1 and Echo 2), squared mesh was used to test the slabs, in order to cover all areas in question. On each slab three tests were made: one applying the load in the centre of the slab, and the other two close to a transversal or longitudinal joint, in order to characterize their respective efficiency of load transfer.

In the next table are presented the average values and the standard deviation obtained for the deflections in the centre of the slab in each one of the aprons.

Table 5.1 - Standard deviation and average values of deflection of the rigid pavements measured in the centre of the slab

Pavement	Deflections (µm)													
	D0		D1		D2		D3		D4		D5		D6	
	M	σ	M	σ	M	σ	M	σ	M	σ	M	σ	M	σ
Charlie R.	172	65	142	58	119	53	93	46	57	33	31	21	7	6
Delta 1	197	44	179	42	162	39	141	35	108	29	78	23	35	13
Delta 2	171	32	153	28	138	26	119	24	87	19	60	16	25	10
Echo 1	430	83	375	59	338	59	288	50	210	37	146	29	61	19
Echo 2	374	108	330	94	299	85	258	75	195	58	414	47	61	31

Legend:

D_0 to D_6 – measured deflections due to the load in the centre of the slab, normalized for 150 kN;

M – average deflection value;

σ – standard deviation of deflections.

5.3 Core samples and laboratory tests

In each apron were made boring tests to determine the layer thicknesses and samples were collected to determine the traction resistance in diametrical compression of concrete samples. The location of extracted cores and their thickness are presented in the next table.

Table 5.2 - Thicknesses of cores and test results in diametrical compression

Core	Apron	Location N° slab	Concrete layers thicknesses (m)	Subgrade layer thickness (m)	Breaking load (kN)	Breaking traction tension (MPa)
H15	Charlie R.	15	0.15	0.08	93.8	5.8
H18		18	0.13	0.08	87.2	5.4
D2	Delta 1	13	0.23	-	75.2	4.8
D1	Delta 2	9	0.235	-	74.7	4.8
E13	Echo 1	13	0.155	-	59	3.7
E15		15	0.155	-	79.1	4.9
E23*		23	0.125	-	-	-
E1	Echo 2	21	0.145	0.075	62.2	4

*** -** cracked core

The results presented above confirm that the slabs thicknesses are close to the values of the design data. It is observed that the slabs thicknesses of aprons Charlie R. and Echo are quite low comparing with the usual thicknesses in airport pavements.

Using the cores collected from the concrete slabs samples were prepared to determine the concrete strength, more specifically its tensile strength in diametrical compression. The respective results are also presented in the table above; it can be observed that the values for the respective resistance vary between 3.7 MPa and 5.8 MPa.

5.4 Layers deformability characteristics

Following the usual methodology for rigid pavements, a first attempt was made to determine the concrete elasticity moduli and the reaction moduli of the foundations of rigid pavements, using Westergaard model.

It is recalled that in the Westergaard model the pavement is treated as a slab on elastic supports, whereby the sub base layer, usually in soil-cement or in lean concrete is grouped together with the foundation soil layer. So, the reaction modulus refers to the assembly made of the sub-base and the soil layer, which is generally designates as *foundation.*

Given the peculiarity of the structure of rigid pavements Charlie R., Echo 1 and Echo 2, in particular because the cement concrete slabs are thin (around 0.15 m), the subjacent soil-cement layers have a significant contribution for the structural pavement behaviour, making the Westergaard model inadequate. Therefore, the first results obtained by direct application of the Westergaard model were not considered in this study.

So, it was first determined the deformability characteristics of the rigid layers, following the same methodology used for flexible pavements, using the program BISAR 3.0. This program uses an multilayer layers elastic model and allows considering various interface conditions between layers, being considered for this study, at the interface between concrete slab and soil cement layer, a tangential deformability (shear compliance) of:

$1/K_T = 12 \text{ x } 10^{-10} \text{ m}^3/ \text{ N}$

The layers deformability characteristics were determined based on the results of tests carried out in the centre of the slabs and that are presented in the next table.
As for the Poisson coefficient, it was adopted a value of 0.20 for concrete and 0.25 for the soil-cement.

Table 5.3 - Structural behaviour model deducted from load tests on rigid pavements – elastic layers model

Zone	CC		LC		FS		
	E1	h1	E2	h2	E3	h3	E4
Charlie R.	42000	0.150	7500	0.080	300	1.20	1000
Delta 1	40000	0.235	7000	0.080	105	1.10	1000
Delta 2	38000	0.230	6500	0.080	200	1.20	1000
Echo 1	32000	0.155	7500	0.080	60	1.00	1000
Echo 2	40000	0.145	7500	0.075	80	1.20	1000

Legend

CC – Concrete cement

LC – Lean concrete

FS – Foundation soil

E_1, E_2, E_3, E_4 – Deformability moduli

h_1, h_2, h_3, h_4 – Thicknesses

The obtained results, especially the deformability modulus, were used in a Westergaard model to estimate an equivalent reaction moduli of the subjacent layers in order to obtain a maximum tension equal to the one obtained previously with the elastic layers model. The equivalent reaction moduli are presented in the next table.

Table 5.4 - Equivalent reaction moduli for Westergaard application

Zone	Cement concrete		Foundation
	h (m)	E* (MPa)	k (MN/m^3)
Charlie R.	0,150	42000	1500
Delta 1	0,235	40000	570
Delta 2	0,230	38000	630
Echo 1	0,155	32000	530
Echo 2	0,145	40000	460

Legend

h – layers thickness;

E* - deformability modulus;

k - equivalent reaction modulus of the foundation.

5.5 Joint load transfer efficiency

In the next table are presented the average values, the standard deviation and the characteristics values of the joint load transfer, in terms of deflections, for the rigid pavements joints.

Table 5.5 – Joint load transfer (E_d)

Platforms	Joint type	E_d (%)			α
		M	σ	E^{85}_d	
Charlie R.	Transversal	64	23	40	86
	Longitudinal	80	15	64	-
Delta 1	Transversal	43	28	14	93
	Longitudinal	62	14	47	-
Delta 2	Transversal	43	18	24	91
	Longitudinal	65	12	53	-
Echo 1	Transversal	75	22	52	82
	Longitudinal	70	8	61	-
Echo 2	Transversal	68	18	49	83
	Longitudinal	77	10	67	-

Legend:

M – Average value of E_d;

σ – standard deviation of E_d;

E^{85}_d – Characterisic vlaue of joint load transfer in ters of deflections;

α - tension reduction factor.

Based on the results presented in the above table, it is observed that the transversal joints are, in a general way, in worse conditions, presenting variability in its behaviour. For calculus reasons, it was selected the most unfavourable values, for each platform.

According to Witczak`s methodology (1989), the joint load transfer in terms of tensions, E_σ, relates to the joint load transfer in terms of deflections, E_d, through the relation:

$$\log E\sigma\ (\%) = 1.611 + 0.0266\ x\ Ed\ (\%)$$

From this value, it can be calculated the tension reduction factor acting next to a free edge, by the next expression: $E\sigma = \frac{1}{\alpha} - 1$. The value of α, determined like this it is used to estimate the tension value induced by the aircraft's wheels, when these circulate close to the edge of the slabs, from the values calculated for the free edge joint load case using the Westergaard formulas. These values are presented in the table 5.5.

6 Classification ACN-PCN

6.1 Methodology ACN-PCN

6.1.1 Definition

The ACN-PCN method of classification was introduced by ICAO in the early eighties (ICAO, 1983). The system is used to classify the aircrafts and the bearing strength capacity of a pavement. It is possible to express the effect of an aircraft on a runway`s pavement, by a single numerical value.

6.1.2 System methodology

The ACN-PCN system is structured so a pavement with a particular PCN value can support, without weight restrictions, an airplane that has an ACN value equal to or less than the pavement's PCN value (Transport Canada, 2004). This is possible because ACN and PCN values are computed using the same technical basis.

6.1.3 Application

The use of the standardized method of reporting pavement strength applies only to pavements with bearing strengths of 5 700 kg or greater. The method of reporting pavement strength for pavements of less than 5 700 kg bearing strength remains unchanged (AC:150/5335-5C, 2014).

6.2 Determination of aircraft classification number

6.2.1 Definition

The airplane manufacturer provides the official computation of an ACN value. Computation of the ACN requires detailed information on the operational characteristics of the airplane such as maximum aft centre of gravity, maximum ramp weight, wheel spacing, tire pressure, and other factors (AC:150/5335-5C, 2014)

6.2.2 Methodology

For flexible pavements, airplane landing gear flotation requirements are determined by the California Bearing Ratio (CBR) method for each subgrade support category. The CBR method employs a Boussinesq solution for stresses and displacements in a homogeneous, isotropic elastic half-space. For rigid pavements, the airplane landing gear flotation requirements are determined by the Westergaard solution for a loaded elastic plate on a Winkler foundation (interior load case), assuming a concrete working stress of 2.75 MPa.
Using the parameters defined for each type of pavement section, a mathematically derived single wheel load is calculated to define the landing gear/pavement interaction. The derived single wheel load implies equal stress to the pavement structure and eliminates the need to specify pavement thickness for comparative purposes. This is achieved by equating the thickness derived for a given airplane landing gear to the thickness derived for a single wheel load at a standard tire pressure of 1.25 MPa. The ACN is defined as two times the derived single wheel load (expressed in thousands of kilograms) (AC:150/5335-5C, 2014).

6.2.3 Operational Frequency

Operational frequency is defined in terms of coverage that represents a full-load application on a point in the pavement. Coverage must not be confused with other common terminology used to reference movement of aircraft. As an aircraft moves along a pavement section it seldom travels in a perfectly straight path or along the exact same path as before. This movement is known as aircraft wander and is assumed to be modelled by a statistically normal distribution. As the aircraft moves along a taxiway or runway, it may take several trips or passes along the pavement for a specific point on the pavement to receive a full-load application. It is easy to observe the number of passes an aircraft may make on a given pavement, but the number of coverage must be mathematically derived based upon the established pass-to-coverage ratio for each aircraft.

6.2.4 Variables Involved in Determination of ACN Values

Because aircrafts can be operated at various weight and centre of gravity combinations, ICAO adopted standard operating conditions for determining ACN values. The ACN is to be determined at the weight and centre of gravity combination that creates the maximum ACN value. Tire pressures are assumed to be those recommended by the manufacturer for the noted conditions. Aircraft manufacturers publish maximum weight and centre of gravity information in their Aircraft Characteristics for Airport Planning (ACAP) manuals. To standardize the ACN calculation and to remove operational frequency from the relative rating scale, the ACN-PCN method specifies that ACN values be determined at a frequency of 10,000 coverages.

6.3 Determination of PCN numerical

6.3.1 Definition

The determination of a pavement rating in terms of PCN is a process of (1) determining the ACN for each aircraft considered to be significant to the traffic mixture operating of the subject pavement and (2) reporting the ACN value as the PCN for the pavement structure. Under these conditions, any aircraft with an ACN equal to or less than the reported PCN value can safely operate on the pavement subject to any limitations on tire pressure.

6.3.2 Methodology

Determination of the numerical PCN value for a particular pavement can be based upon one of two procedures: the “Using” aircraft method or the “Technical” evaluation method. ICAO procedures permit member states to determine how PCN values will be determined based upon internally developed pavement evaluation procedures. Either procedure may be used to determine a PCN, but the methodology used must be reported as part of the posted rating.
According to ICAO, a pavements PCN value represents the load capacity as the maximum allowable load per single wheel, with a tire pressure of 1.25 MPa, for 10000 coverages.

Using Aircraft Method to Determine PCN

The Using aircraft method is a simple procedure where ACN values for all aircraft currently permitted to use the pavement facility are determined and the largest ACN value is reported as the PCN. This method is easy to apply and does not require detailed knowledge of the pavement structure. The subgrade support category is not a critical input when reporting PCN based on the Using Aircraft Method. The recommended subgrade support category when information is not available should be Category B.

Technical Evaluation Method to Determine PCN

The strength of a pavement section is difficult to summarize in a precise manner and will vary depending on the unique combination of aircraft loading conditions, frequency of operation, and pavement support conditions. The technical evaluation method attempts to address these and other site-specific variables to determine reasonable pavement strength. In general terms, for a given pavement structure and given aircraft, the allowable number of operations (traffic) will decrease as the intensity of pavement loading increases (increase in aircraft weight). It is entirely possible that two pavement structures with different cross-sections will report similar strength. However, the permissible aircraft operations will be considerably different. This discrepancy must be acknowledged by the airport operator and may require operational limitations administered outside of the ACN-PCN system. All of the factors involved in determining a pavement rating are important, and it is for this reason that pavement ratings should not be viewed in absolute terms, but rather as estimations of a representative value. A successful pavement evaluation is one that assigns a pavement strength rating that considers the effects of all variables on the pavement.

The accuracy of a technical evaluation is better than that produced with the Using aircraft procedure but requires a considerable increase in time and resources. Pavement evaluation may require a combination of on-site inspections, load-bearing tests, and engineering judgment. It is common to think of pavement strength rating in terms of ultimate strength or immediate failure criteria. However, pavements are rarely removed from service due to instantaneous structural failure. A decrease in the serviceability of a pavement is commonly attributed to increases in surface roughness or localized distress, such as rutting or cracking. Determination of the adequacy of a pavement structure must not only consider the magnitude of pavement loads but the impact of the accumulated effect of traffic volume over the intended life of the pavement. The subgrade support category is a necessary input when reporting PCN based on the Technical Method.

6.3.3 PCN classification

The PCN system uses a coded format to maximize the amount of information contained in a minimum number of characters and to facilitate computerization. The PCN for a pavement is reported as a five-part number where the following codes are ordered and separated by forward slashes**: Numerical PCN value / Pavement type / Subgrade category / Allowable tire pressure / Method** used to determine the PCN.

Numerical PCN value

The PCN numerical value indicates the load-carrying capacity of a pavement in terms of a standard single wheel load at a tire pressure of 1.25 MPa (181 psi). The PCN value should be reported in whole numbers, rounding off any fractional parts to the nearest whole number.

Pavement type

For the purpose of reporting PCN values, pavement types are considered to function as either flexible or rigid structures.

Table 6.1 lists the pavement codes for the purposes of reporting PCN.

Table 6.1 - Pavement Codes for Reporting PCN

Pavement type	Pavement code
Flexible	F
Rigid	R

a. Flexible pavement

Flexible pavements support loads through bearing rather than flexural action. They comprise several layers of selected materials designed to gradually distribute from the surface to the layers beneath. The design ensures that load transmitted to each successive layer does not exceed the layer's load-bearing capacity (AC:150/5335-5C, 2014).

b. Rigid pavement

Rigid pavements employ a single structural layer, which is very stiff or rigid in nature, to support the pavement loads. The rigidity of the structural layer and resulting beam action enable rigid pavement to distribute loads over a large area of the subgrade. The load-carrying capacity of a rigid structure is highly dependent upon the strength of the structural layer, which relies on uniform support from the layers beneath (AC:150/5335-5C, 2014).

Subgrade category

The ACN-PCN method adopts four standard levels of subgrade strength for rigid pavements and four levels of subgrade strength for flexible pavements. These standard support conditions are used to represent a range of subgrade conditions as shown in the following table.

Table 6.2 - Standard Subgrade Support Conditions for Rigid and Flexible Pavement ACN Calculation

Subgrade Strength Catergory	CBR	k (MN/m3)	Code Designation
High	CBR≥13	k≥120	A
Medium	8≤CBR≤13	60<k<120	B
Low	4≤CBR≤8	25<k≤60	C
Ultra Low	CBR≤4	k<25	D

Allowable tire pressure

Table 6.3 lists the allowable tire pressure categories identified by the ACN-PCN system. The tire pressure codes apply equally to rigid or flexible pavement sections; however, the application of the allowable tire pressure differs substantially for rigid and flexible pavements.

Table 6.3 - Tire Pressure Codes for Reporting PCN

Category	Code	Tire pressure range
Unlimited	W	No pressure limit
High	X	Pressure limited to 1.75 Mpa
Medium	Y	Pressure limited to 1.25 Mpa
Low	Z	Pressure limited to 0.50 Mpa

a. Tire Pressures on Flexible Pavements

Tire pressures may be restricted on asphaltic concrete (asphalt), depending on the quality of the asphalt mixture and climatic conditions. Tire pressure effects on an asphalt layer relate to the stability of the mix in resisting shearing or densification. A poorly constructed asphalt pavement can be subject to rutting due to consolidation under load. The principal concern in resisting tire pressure effects is with stability or shear resistance of lower quality mixtures. A properly prepared and placed mixture that conforms to FAA specification can withstand substantial tire pressure in excess of 218 psi (1.5 Mpa). Improperly prepared and placed mixtures can show distress under tire pressures of 100 psi (0.7 MPa) or less. Although these effects are independent of the asphalt layer thickness, pavements with well-placed asphalt of 10.2 to 12.7 cm in thickness can generally be rated with code X or W, while thinner pavement of poorer quality asphalt should not be rated above code Y (AC:150/5335-5C, 2014).

b. Tire Pressures on Rigid Pavements

Aircraft tire pressure will have little effect on pavements with Portland cement concrete (concrete) surfaces. Rigid pavements are inherently strong enough to resist tire pressures higher than currently used by commercial aircraft and can usually be rated as code W (AC:150/5335-5C, 2014).

Method used to determine PCN

T – Technical evaluation method
U – Using aircraft method

The PCN system recognizes two pavement evaluation methods. If the evaluation represents the results of a technical study, the evaluation method should be coded T. If the evaluation is based on "Using aircraft" experience, the evaluation method should be coded U. Technical evaluation implies that some form of technical study and computation were involved in the determination of the PCN. Using aircraft evaluation means the PCN was determined by selecting the highest ACN among the aircraft currently using the facility and not causing pavement distress.

Example PCN reporting

An example of a PCN code is 80/R/B/W/T, with 80 expressing the PCN numerical value, R for rigid pavement, B for medium strength subgrade, W for high allowable tire pressure, and T for a PCN value obtained by a technical evaluation.

6.4 Case study

6.4.1 Subgrade classification

In conformity with the results obtained for in 3.5, the subgrade layer was classified as follows:

1. High Strength
    - Zones z3, z4 and z5 were classified as high subgrade strength (CBR>13%).
    - The rigid parking aprons Charlie R. and Delta 2;
2. Medium Strength
    - Zones z1 and z2 of the runway 18-36 (8 %< CBR<13%);
    - The remaining rigid parking aprons: Delta 1, Echo 1 and Echo 2;
3. Low Strength

6.4.2 Finding ACN values for the operating aircrafts with COMFAA 3.0

According to the information provided by the aerodrome management, the most frequently aircrafts that use the runway are ATP, ATR72, FOKKER 50 and FOKKER 100. In the next table the ACN values of the respective aircrafts are displayed, but also the ACN of the BOEING 737-100 aircraft, which is considered, besides FOKKER 100, the representative aircraft with a maximum take-off weight of 400 kN.

Steps to follow in order to determine ACN values using COMFAA 3.0:

1. Select aircraft group
2. Select aircraft from library
3. Confirm aircraft parameters
4. Click ACN calculation button
5. Click to calculate flexible ACN
6. Click to calculate rigid ACN

Table 6.4 - Aircrafts ACN that operate the runway and ACN of BOEING 737-100

Aircraft	Maximum take-off weight (kN)	ACN				
		Flexible pavement			Rigid pavement	
		A	B	C	A	B
FALCON50	196	*	*	*	*	*
ATP	229	*	*	*	*	*
ATR 72	211	11	12	14	13	14
FOKKER50	205	9	10	11.5	10	11
B 737-100	445	25	26	29	27	29
FOKKER100	452	25	27	30	28	30

6.4.3 Failure criteria

The methodology used to determine the pavement classification number is based on the structural analysis of the pavement, using the structural model presented in the former chapter.

Flexible pavements

For flexible pavements two criteria were used: the limitation of fatigue cracking on the base of the asphalt layer and the limitation for the formation of permanent deformation of the foundation soil. The limitation of fatigue cracking of the asphalt layers is made using Shell criteria:

$$\varepsilon_t = (0.856 \text{ x } V_b + 1.08)E^{-0.36}\ N^{-0.2}$$

V_b - volumetric proportion of bitumen;
E – deformability modulus of bituminous mixture, in Pa;
ε_t – maximum extension of traction induced in the layer;
N – admissible number of load applications.

It is considered that the percentage of volumetric proportion of bitumen in the layers of bituminous mixtures is around 9%, knowing that the maximum extension of traction was calculated at the base of the bitumen macadam layer.
Foundation contribution of limitation for the formation of excessive permanent deformation was made using the Chou criteria (1982) for airport pavements:

$$\varepsilon_c = 0.00539N^{-0.1436}$$

ε_c – the vertical extension of compression on top of the foundation soil;
N – corresponding number of admissible passes.

Rigid pavements

According to ICAO`s recommendations for ACN-PCN classification of rigid pavements, the PCN can be calculated by a equivalent load per single wheel that induces a traction tension in the concrete of 2.75 MPa. It is admitted that for this tension level, the pavement will be able to support an unlimited number of load applications.
The result value obtained of the diametrical compression tests performed on the samples of rigid pavements platforms was a minimum value of 3.70 MPa for the traction resistance in diametrical compression of the concrete.
According to the recommended expressions by *Cement and Concrete Association* (Australia), if the ratio between the installed tension in concrete and the traction resistance at flexion is 0.64, then the admissible load applications are above 10 000. This criterion is used, in PCN calculations, in order to limit the tensions due to the load applications in the centre of the slab.
In order to limit the tensions due to the load applications along the edges of the slabs, considering the load transference presented in TABLE 6.5, an additional calculation was made. The ratio between the installed tension in concrete and its tensile resistance is 0.67, corresponding to 5 000 coverages.

Table 6.5 - Maximum admissible tensions due to the load on the concrete slabs

	σ_{cd} (MPa)	σ_f (MPa)	$\sigma_{f,calculation}$ (MPa)	$\sigma_{center,max}$ (MPa)	$\sigma_{edge,max}$ (MPa)
Charlie R	5.4	8.1	5.5	0.64x5.50=3.53	0.67x5.50=3.67
Delta 1	3.96	5.94			
Delta 2	4.79	7.19			
Echo 1	3.7	5.55			
Echo 2	3.96	5.94			

Legend:

σ_{cd} – tensile resistance in diametrical compression of the concrete sample collected in situ;

σ_f – estimated tensile resistance at flexion based on the previous value;

$\sigma_{f,calculation}$ – tensile resistance at flexion adopted for calculation effects;

$\sigma_{center,max}$ – maximum admissible tension value due to the loads applied in the center of the slabs, corresponding to a number of load applications superior to 10000;

$\sigma_{bordo,max}$ – maximum admissible tension value due to the loads applied near the edge of the slabs, corresponding to a number of load applications superior to 5000.

6.4.4 Determination of PCN using BISAR 3.0

The runway was divided in 5 zones with similar characteristics. So, we are going to calculate the PCN for each zone and select the most conditioning one.

Flexible pavements

Using the behaviour model found in the previous chapter, were determined the strains made by a standard single wheel load at a tire pressure of 1.25 MPa (181psi) for 10000 coverages. The conditioning strains (ε_t – tensile strain on the bituminous layers and ε_c - compression strain on the top of the foundation soil), were compared with the respective maximum allowable values for 10000 coverages. The results are shown in the next table.

Table 6.6 - Load-bearing capacity of the runway

	Zone	P (kN)	Strains limit(10^{-6}) (10000 coverages)		Strains (10^{-6}) load induced		PCN
			ε_t	ε_c	ε_t	ε_c	
Pista 18-36	z1	230	477	1444	475	1120	42/F/B/W/T
	z2	209	477		454	1440	42/F/B/W/T
	z3	400	455		396	1280	>80/F/A/W/T
	z4	400	455		408	1330	>80/F/A/W/T
	z5	348	453		405	1440	70/F/A/W/T

Legend:

P – single wheel load at a tire pressure of 1.25 MPa

ε_t – tensile strain in the bituminous layer

ε_c – compression extension in the foundation soil

As can be seen, for each subsection with similar characteristics we have a similar PCN value. The most conditioning zones are z1 and z2, with a medium subgrade strength category, which leads to a smaller PCN of: 42/F/B/W/T. This PCN value is bigger than ACN of the aircrafts that use the runway.

Rigid pavements

In the next table are presented the load bearing capacity values for the rigid pavements of the aprons, in terms of a maximum admissible single wheel load, with a tire pressure of 1.25 MPa and the corresponding maximum induced tensile tensions in the slabs by applying the load in the middle and at the joints of the slabs. The limit values for these tensions and the corresponding PCN values are also presented in Table 6.7.

Table 6.7 - Load bearing capacity of the rigid pavements of the aprons

Zone	P (kN)	Limit tensions (MPa)		Load induced tensions (MPa)		PCN
		$\sigma^{centre(1)}$	$\sigma^{joint(2)}$	σ^{centre}	σ^{joint}	
Charlie	63	3.53	3.67	2.37	3.66	13/R/A/W/T
Delta 1	112	3.53	3.67	2.05	3.65	23/R/B/W/T
Delta 2	104	3.53	3.67	2.06	3.65	21/R/A/W/T
Echo 1	58	3.53	3.67	2.34	3.64	12/R/B/W/T
Echo 2	44	3.53	3.67	2.23	3.65	9/R/B/W/T

Legend:

P – single wheel load at a tire pressure of 1.25 MPa

σ^{centre} – tension due to the center slab load application

σ^{joint} – tension due to the joint slab load application

(1) – value corresponding to 10 000 coverages

(2) - value corresponding to 5 000 coverages

The PCN values of the rigid aprons are not above the ACN of the aircrafts. The biggest aircraft, FOKKER 100, has an ACN value of 28 and 30, for rigid pavements with subgrade category A and B, respectively. The rest of the fleet can use all the aprons except Echo 2, which has a PCN value smaller than the ACN value of all the aircrafts that usually use the aprons.

6.4.5 Structural life

In addition to the ACN-PCN classification, the runway was evaluated remaining structural life, in terms of an admissible numbers of passages, for aircrafts that have maximum take-off weight of 400 kN.

So, the runway`s ability to operate aircrafts with a maximum take-off weight of 400 kN and the potential reinforcement requirements. Fokker 100 and Boeing 737-100 were used for this study, both of them with a take-off weight superior than 400 kN.

The next table presents the structural life of the runway, in terms of admissible coverages number and the admissible passages number. The conversion between the coverages number, N_r, and the passages number, N_p, was made in conformity with FAA regulations.

$N_p = 3.48 \times N_r$

The calculation was made in BISAR 3.0, using the same deformability modulus and layer thicknesses as in the PCN determination. The same dimensioning criteria were used.

Table 6.8 - Load-bearing capacity of the runway – admissible passages of the conditioning aircraft

	Zone	Maximum strain		Maximum number of coverages		Maximum number of passages	
		Fokker100	B 737-100	Fokker 100	Boeing 737-100	Fokker 100	Boeing 737-100
Runway 18-36	z1	3.39E-04	3.14E-04	5.50E+04	8.00E+04	1.91E+05	2.78E+05
	z2	3.43E-04	3.17E-04	5.20E+04	7.70E+04	1.81E+05	2.68E+05
	z3	2.29E-04	2.29E-04	3.10E+05	4.50E+05	1.08E+06	1.57E+06
	z4	2.50E-04	2.37E-04	2.00E+05	2.60E+05	6.96E+05	9.05E+05
	z5	2.45E-04	2.29E-04	2.15E+05	3.00E+05	7.48E+05	1.04E+06

As the table shows, the worst-case corresponds to 1.8×10^5 passages of the Fokker 100 aircraft for a life of the runway of 20 years. So, the maximum number of passages per year by aircrafts with the take-off weight of 400kN is 9000 passages.

6.5 Final considerations

In this study was evaluated the load-bearing capacity of the flexible runway and rigid aprons.
There were made load tests using the falling weight to cover the whole area of study. After this the ground penetrating radar was used, combined with coring tests, we could divide our pavements into zones with homogeneous structural behaviour.
With this knowledge we could establish a structural behaviour model, which was used to calculate the PCN, using the structural analyse methodology.
For the runway 18-36 the PCN values are superior to the ACN values of the aircrafts that use the airfield, considering appropriate reporting a **PCN = 42/F/B/W/T.**
The rigid pavements of the parking aprons have PCN values inferior to the ACN of the heaviest aircraft (Fokker 100), but superior to the ACN of the rest of the fleet.

Besides the ACN-PCN classification, the assessment of the possibility of operation on the runway of aircrafts with the maximum take-off weight of 400 kN is presented in this chapter. Two aircrafts with a maximum take-off weight of 400 kN were chosen in order to calculate the load-bearing capacity of the runway in terms of admissible coverages number. It was concluded that the most unfavourable case corresponds to 1.8×10^5 movements of Fokker 100 for a structural life of 20 years, which transforms into 9000 coverages per year of the same aircraft. So the airport can operate aircrafts with maximum take-off weight of 400 kN without any reinforcement.

7 Conclusions

Nowadays, the need of air transportation is growing rapidly. The aircrafts are getting bigger and bigger, the aircraft traffic flow is higher than always and the number of passengers has reached an incredible high number. From commuters to around the globe passengers, from the smallest aircrafts to the biggest ones, the runways needs to be able to support as many types of aircrafts that are available now, but also the ones that will be developed in the future. Figure 7.1 shows the four biggest aircrafts available in this moment around the world.

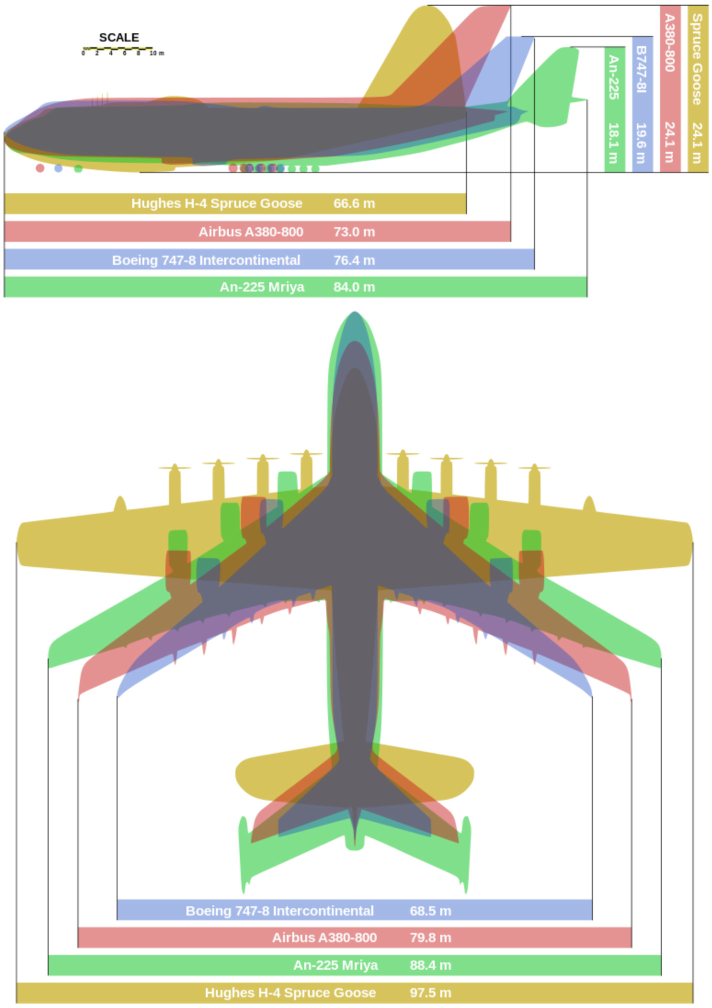

Figure 7.1 – The biggest world aircrafts (Daily Mail, 2015)

The evaluation of an airport runway is the most important process in the designing and execution of new runways, or in reinforcing an already existing runway in order to support new aircrafts.

This dissertation intended to explain the whole evaluation process of an airport runway. It is divided into two main evaluations, structural and functional. Both of them were explained for flexible and rigid pavements.
This study enables the reader to follow the steps presented, to calculate all the necessary mathematical expressions and to use the respective soft wares, including FAARFIELD, COMFAA or BISAR3.0.

The evaluation process is a combination of visual inspection, records research and non-destructive tests. Also, a computational process is required, this allows a greater precision and with a good interpretation and application of the results, the runway will be evaluated very close to its real characteristics.

The structural evaluation regards the use of the GPR and the use of the FWD with the corresponding interpretation of the results. The interpretation of FWD data together with layer thickness data obtained from GPR contributes to the improvement of the methodology for structural pavement evaluation.

As for the functional evaluation, the use of the GRIP Tester is a very important test, performed to find the friction coefficient of the runway. The safety of the aircrafts depends on this functional characteristic. Also the "sand patch" method was used to find the depth texture of the surface layer. There were analysed load tests performed with the falling weight deflectometer in flexible and rigid pavements. After this, the ground penetrating radar was used, combined with soil boring tests. In this way, it became possible to divide the pavements into zones with homogeneous structural behaviour.

With this knowledge structural behaviour models were established, which were used to calculate the PCN, based on structural analysis methodology.

This evaluation concludes into a PCN number, or even better into an ACN/PCN classification, which is the ratio between the Aircraft Classification Number and the Pavement Classification Number. This ratio is the most common condition that a specific aircraft may use a certain runway. It is expressed so that the aircraft pilots can relate their

Aircraft Classification Number with the runway's Pavement Classification Number. All that is required is to search for the ACN that is presented in the aircraft's manual and to compare it with the PCN of the airport's runway; this information is available and should be reported by the airport's authorities. So a pilot that compares these two numbers and concludes that the PCN is bigger than the ACN, he can land safely.

Besides this classification, a structural lifetime expectation of the runway is calculated. This is possible using the same computer program as before, BISAR3.0; but this time is calculated the number of passages of some specific aircrafts that can be operated safely on the respective runway. This way we can predict a possible reinforcement of the runway, or the construction of a new one. All of this is included in what is called, airport pavements evaluation.

The evaluation of airport pavements is critical, as influences the safety of the aircrafts, the operation costs, the comfort, as well as the enviroment.

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