Structural Evaluation of a Pre-stressed Concrete Airport Pavement

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ABSTRACT: This paper concerns the assessment of the bearing capacity of a pre-stressed concrete airport pavement built in the 60's, using non-destructive tests complemented with a number of other *in-situ* and laboratory tests aiming at the characterisation of the stress state in the concrete slab 40 years after construction.
The work performed comprised visual inspection, FWD tests, core drilling and site investigation, "small flat jack" tests (normally used in rock mechanics), as well as the characterisation of the *in situ* stress condition of the pre-stressing wires. A finite element computer program was used, together with multi-layer computer programs, for modelling the pavement, taking into account different friction conditions at the interface between the concrete slab and the soil-cement sub-base.
The paper describes the methodology used in the study and presents some of the results achieved.

KEY WORDS: Airport pavement evaluation, Pre-stressed concrete, Pavement condition evaluation.

1 INTRODUCTION

Beja military airfield was built in the 1960's and comprises, among other facilities, two runways with pre-stressed concrete pavements: the main runway is 60 m wide and 4000 m long, and the secondary runway is 30 m wide and 3200 long. According to the available construction records the runway pavements are composed of 160 mm thick pre-stressed Portland Cement Concrete (PCC) slabs, placed over a 150 mm soil-cement sub-base.

After 40 years of service life the runways will be used in the future also by civil aircraft, according to the development plans for the future Beja civil airfield. Therefore, it was questioned whether the pavements would have adequate bearing capacity for the most common commercial aircrafts, and what would be its Pavement Classification Number (PCN), according to ICAO's classification system (ICAO 1983).

Given the specificity of this type of pavement, the work programme for the evaluation included a number of specific tests aiming at the characterization of the tension in the pre-stressing cables and the resulting compression in the concrete due to its action (Antunes, M.L. *et al,* 2003). In fact, 40 years after construction of the runway, it was questionable whether the pre-stressing system would still be working as originally planned. The main activities performed for the pavement evaluation were the following:

a) Analysis of construction records;
b) Pavement visual condition assessment;
c) FWD testing;
d) Core drilling and concrete testing;
e) Evaluation of in-situ tensile stresses in the cables in the longitudinal and transverse directions;
f) Evaluation of in situ compressive stresses in the concrete, also in the longitudinal and transverse directions;
g) Interpretation of the test results, in order to set up a model for the pavement structure;
h) Determination of the maximum allowable single wheel load.

2 DESCRIPTION OF THE PAVEMENTS

The pavement is composed of 160 mm thick pre-stressed PCC slabs, placed over a 150 mm soil-cement layer (Figure 1). A “kraft” paper sheet was placed between the concrete and the soil cement, in order to prevent bond between the two layers. In the cores extracted from the pavement, evidence of the “kraft” paper was still visible. A regulating cement mortar between the soil-cement and the concrete slab was also present (Figure 2).

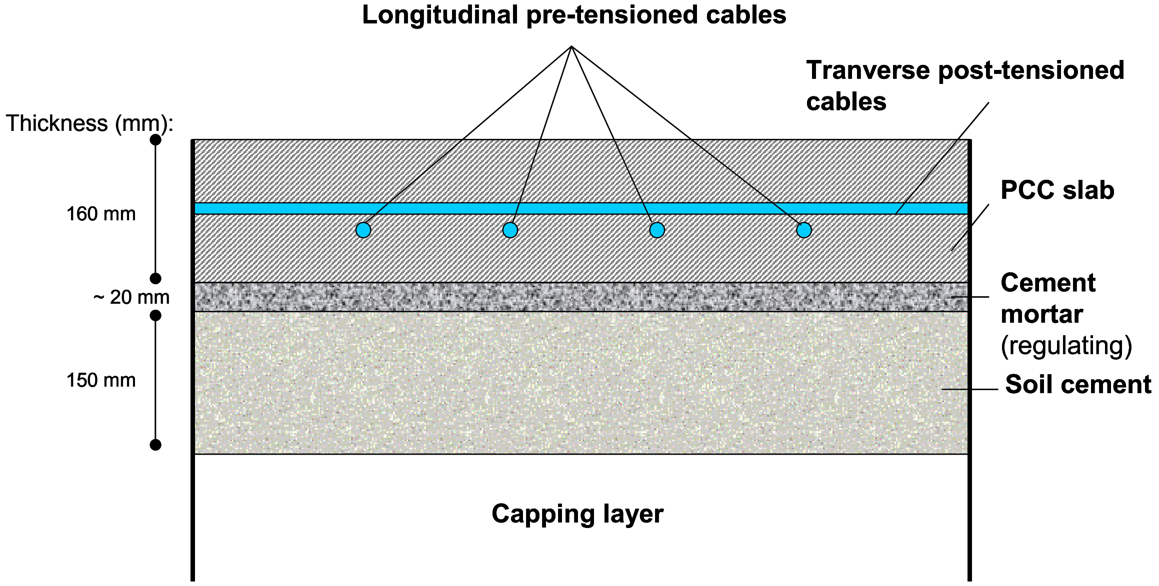

Figure 1: Pavement section

In the longitudinal direction, the PCC slabs are 95 to 100 long, with transverse expansion joints between two consecutive slabs. Although it can be seen that the placing of the concrete was performed in 7.5 m wide strips, in the longitudinal direction, the construction joints are tightly held together by the tensioned wires.

The longitudinal cables consist of two layers of 3 wires each, and they were pre-tensioned before placing the concrete. The transverse cables were placed over the longitudinal cables, inside a metal duct. These cables were post-tensioned after hardening of the concrete and the duct was filled with cement mortar. Each wire has a 40 mm^2 section. The cables are located 50 m apart form each other.

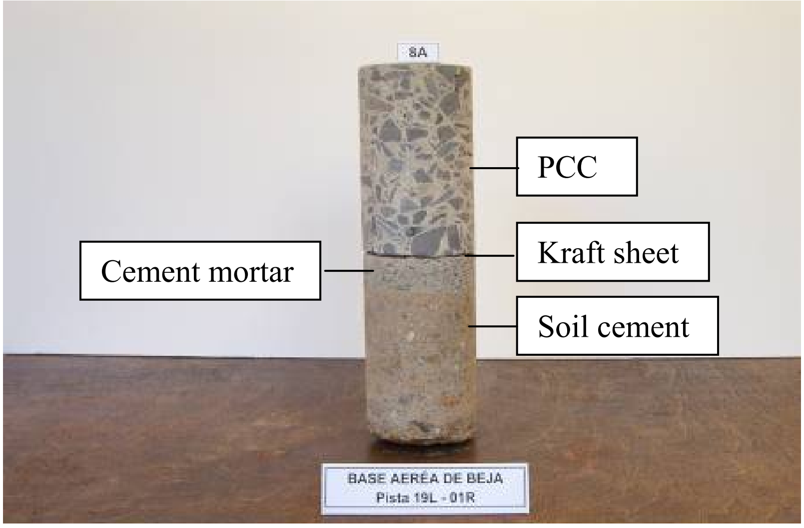

Figure 2: Pavement core

The natural subgrade soil under the pavement is primarily clayey (CL/CH), with the exception of a localized area, where, according to the construction records, there was bedrock (diorite) close to the surface. A capping layer of variable thickness, with a maximum of 420 mm, was placed on the natural soil.

The pavement surface was generally in a good condition, with no visible cracks or any other structural damage. Some of the joints, specially the joints with adjacent pavements, were in need of maintenance.

3 FIELD AND LABORATORY TESTING

3.1 Falling Weight Deflectometer tests

Falling Weight Deflectometer (FWD) tests were performed in points located along lines parallel to the runway center lines: 7 in the main runway and 5 in the secondary runway. Excluding the first impact, two more 150 kN impact loads were applied in each test point. The pavement deflections were measured in 7 points at the surface, located at 0, 300 mm, 450 mm, 600 mm, 900 mm, 1200 mm and 1800 mm offset from the center of the loaded area. The loading plate was a flexible plate, 450 mm diameter.

One of the primary objectives of these tests was the definition of homogeneous zones. The results obtained allowed for a clear identification of the areas where there was bedrock close to the surface. Figure 3 shows and example of the deflection plot obtained along the center line of the main runway, where zone 2P is clearly identified as having a stronger subgrade, then zone 1P.

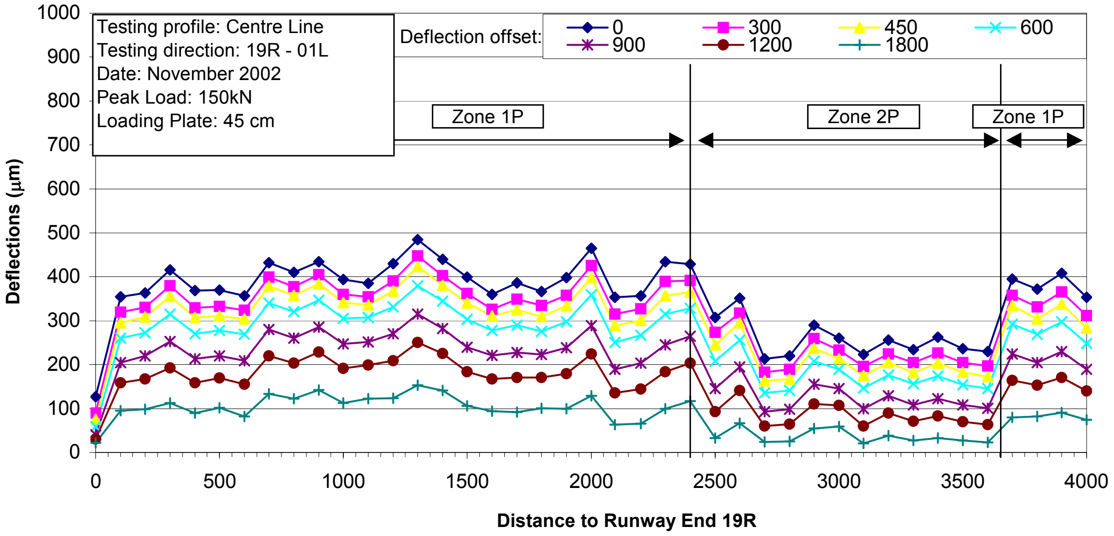

Figure 3 – Centre line deflection plot

3.2 Mechanical characteristics of the concrete

Core drilling was performed at several locations in the two runways, in order to confirm the information gathered from construction records and to collect samples for the following laboratory tests performed on PCC:

- Determination of compressive strength;
- Determination of indirect tensile strength;
- Determination of elastic modulus.

Two of the cores drilled from the pavements also included the soil cement layer (one of them is shown in Figure 2). These soil-cement samples were also tested. The results from the laboratory tests are summarized in Table 1.

Table 1 – Summary of results from laboratory tests

Material	**Compressive strength** (MPa)			**Indirect tensile strength** (MPa)			**Elastic Modulus** (GPa)		
	N	$\overline{X}$	STDEV	N	$\overline{X}$	STDEV	N	$\overline{X}$	STDEV
PCC	10	83.4	11.1	10	6.4	0.6	10	41.8	1.7
Soil-cement	1	13.4	-	-	-	-	1	6.0	-

N - number of samples tested

$\overline{X}$ - average

STDEV - standard deviation

The results summarized in Table 1 show that the concrete modulus is rather high, around 42 GPa. The strength is also high, with an average compressive strength of 83 MPa and an average indirect tensile strength of 6.4 MPa. Using correlations between these properties and the flexural tensile strength (Machado, A.; Neves, R., 2003), a value of 7.5 MPa for this property is considered to be on the safe side. This was the reference strength used for the assessment of the maximum allowable equivalent single wheel load.

3.3 *In-situ* pre-stresses in the concrete slabs

The assessment of in-situ concrete pre-stresses was essential for the study under consideration, since these stresses would condition the maximum allowable stresses induced in the PCC slab by the wheel loads. In order to assess these stresses, the first action was to detect the position of the cables, by using a metal detector and also based on the knowledge of the distance between successive cables.

One of the possible methods for this assessment is to measure the *in-situ* tensile stresses in the cables (Pompeu Santos, S., 2002). The method used for this comprised the following steps:

- To open a "narrow" trench in the concrete pavement until the steel wires are uncovered and to measure the *in-situ* length of a certain segment of the wire;
- To cut the wire segment (Figure 4), bring it to the laboratory and to stretch it again until it reaches the same length as measured in situ; the value of the load that is necessary to restore the wire length will be equal to the load resulting from the in situ tension in the wires;
- To calculate the concrete stresses by dividing the total load in each cable by the pavement section corresponding to the area of influence of the cable.

The results obtained in the tests are summarized in Table 2. This Table also shows the ration between the *in-situ* tensile stress in the steel and the corresponding strength. It can be observed that this ratio is on average 0.46 for the longitudinal wires and 0.53 for the transverse wires, which means that the steel is not working too close to its strength. Based on the results obtained for the estimated concrete stresses, a value of 2.25 MPa was retained.

Table 2 – Summary of test results concerning *in-situ* tensile stresses in the steel wires

Direction of cables	**Number of samples**	**Load** (kN)		σ / R		**Estimated concrete stress** (MPa)
		$\overline{X}$	STDEV	$\overline{X}$	STDEV	
Longitudinal	12	28.9	6.1	0.46	0.10	2.16
Transversal	11	32.3	4.6	0.53	0.08	2.42

$\overline{X}$ - average
STDEV - standard deviation
σ **-** tensile stress in the steel wire
R - steel tensile strength

Figure 4: Assessment of in-situ pre-sresses in the concrete

The stresses in the concrete slabs were also measured by another process, using small flat jacks (SFJ), applying a technique that is frequently used in rock mechanics (Rocha, M.C.; Lopes, J.N.; Silva, J.N.; 1966). The procedure is the following:

- Two points are fixed and marked in the surface; these points are located in each side of the section where the stresses are to be measured;
- The distance between these points is measured and recorded;
- A saw-cut is performed in the section mentioned above (Figure 4);
- The flat jack is introduced in the saw-cut and a load is applied on it, until the distance between the two points is the same as the original; this load will be equal to the concrete pre-stresses + thermal stresses.

The SFJ tests were performed at ten locations for each of the directions, and the results were quite variable, with an average value of 3.5 MPa for the stresses, both in the longitudinal and in the transverse directions. It is not surprising that these are higher then the values derived from the *in-situ* tensile stresses in the cables, since the SFJ tests involve mainly the upper part of the concrete slab, which is more compressed due to thermal actions, since the tests were performed during day time.

4 INTERPRETATION OF TEST RESULTS

4.1 Pavement response model

The results from the FWD tests, combined with the information given by construction records and coring, were used to set up a response model for the pavement structure. The pavement was idealized as a 160 mm PCC layer, with infinite dimensions in the horizontal plane, resting on a soil cement layer, 170 mm thick (corresponding to the soil-cement + cement mortar). The subgrade soil was split into two layers: an upper layer 2 m thick, and a *rigid* layer underneath, which is a common assumption for FWD test result interpretation.

Taking into consideration that the conditions at the interface between the concrete slab and the underlying layer will strongly influence the results, the following types of models were tested for the interpretation of results in one of the most critical zones (zone 1P):

- The Westergaard model, where the pavement is modeled as a slab resting on a Winkler foundation (Westergaard, H.M., 1947).
- A multi-layer linear elastic model (BISAR 3, developed by SHELL) with perfect bond at the layer interfaces.
- Multi-layer linear elastic layers with partial slip at the interface between the first two layers. In this case, the interface has a linear elastic behaviour, with a shear stiffness $K_T = 8333$ MPa/m. Two types of software were used for this case: BISAR 3, where the interface conditions will correspond to a shear compliance $1/K_T = 1.2 \times 10^{-10}$ m^3/N, and; FLAC 4.0 (Finite Element Program). The two different tools provided similar results.
- Multi-layer linear elastic layers with non-linear slip at the interface between the first two layers, where the shear stress at the interface (σ_T) is limited by the Coulomb criteria, given by: $\sigma_T = \sigma_N \times tg\ \phi$, where σ_N is the normal stress and $\phi = 40°$. In this case, only FLAC 4.0 was used.

Table 3 summarizes the assumptions made for each type of model. In this Table, E1, E2 and E3 represent the concrete, the soil-cement and the subgrade moduli (upper part of the subgrade), and k represents the modulus of reaction of the Winkler foundation. The Poisson coefficients were 0.20, for the concrete, 0.25 for the soil-cement and 0.35 for the soil.

Table 3 – Assumptions for different response models

Identification	Type of model	PCC / soil-cement interface	Layer characteristics
a) Westergaard	Slab on Winkler foundation	-	E1 = 42 GPa; k = 105 MN/m^3
b.1) BISAR	Multi-layer linear elastic	Perfect bond	E1 = 42 GPa E2 = 6 GPa E3 = 75 MPa
b.2) BISAR	Multi-layer linear elastic	Linear elastic interface K_T = 8333 MPa/m	E1 = 42 GPa E2 = 6 GPa E3 = 75 MPa
b.3-1) FLAC	Finite element Linear elastic layers	Non-linear interface $\phi = 40°$	E1 = 42 GPa E2 = 6 GPa E3 = 75 MPa
b.3-2) FLAC	Finite element Linear elastic layers	Non-linear interface $\phi = 40°$	E1 = 42 GPa E2 = 6 GPa E3 = 100 MPa

Figure 4 shows the comparison between the deflections obtained with the different moduli. It can be observed that a partial slip linear elastic interface will give the best fit between measured and calculated deflection bowls, therefore the BISAR 3 computer program was used for the interpretation of the tests performed in the other zones and for subsequent calculations.

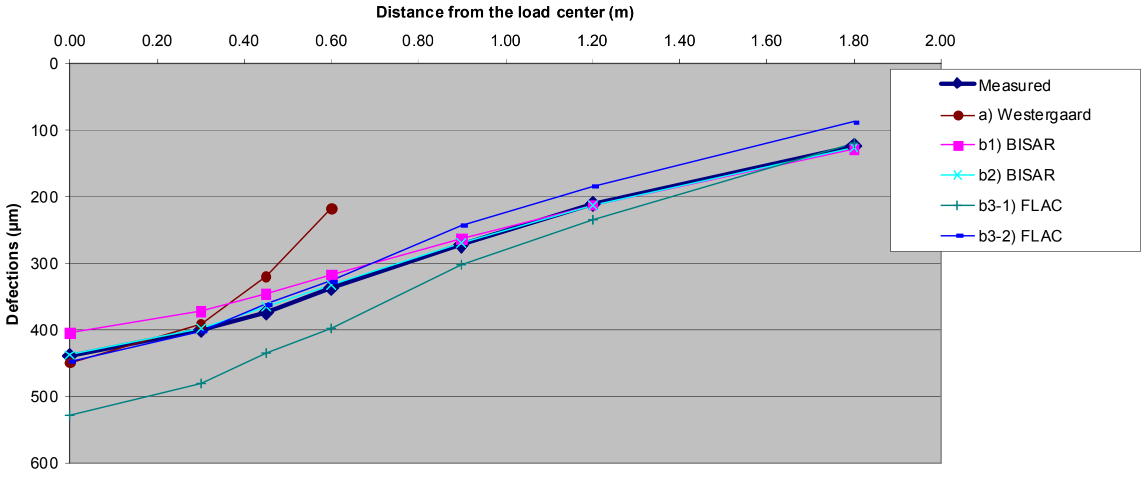

Figure 4 – Comparison between measured and calculated deflections for different types of pavement models

4.2 Calculation of critical stresses in the concrete

The pavement bearing capacity, will be determined by the limitation of tensile stresses induced in the concrete by wheel loads, combined with stresses induced by other loads, such as the pre-stressing and the thermal stresses. The maximum tensile stress is given as a function of the concrete flexural strength. In this case, the following maximum tensile stress was considered, for a number of load applications over 10 000:

$$\sigma_{\max} = 0.64 \times 7.5 \cong 4.8 MPa$$

Two types of thermal stresses were estimated from climatic historical data for the Beja region: stresses induced by a thermal gradient with depth and stresses induced by a uniform cooling of the slab. In order to calculate these stresses, the concrete elastic modulus was considered as 2/3 of the value obtained for instant loads. In the case of uniform cooling, the interface conditions were also modified to take into account the time effects: a non-linear behaviour with $\phi = 45^{\circ}$ was considered for the interface. Table 4 summarises the calculations performed.

Table 4 – Combination of stresses in the concrete slab

Effect	**Stress in the concrete** (MPa)
Pre-stressing system	-2.25
Temperature gradient	1.40
Uniform cooling	1.00
Single wheel load	4.65
Total	4.80

5 FINAL REMARKS

This paper presents the methodology used in the structural evaluation of a pre-stressed concrete airport pavement. This methodology was specially adapted to this specific type of pavement, since the assessment of compressions induced in the concrete by the pre-stressing system is essential. Thermal stresses are also a key issue in this case, due to the slabs dimensions.

Another aspect that is highlighted in this paper is the interface conditions between the concrete and the underlying layers. A good fit was obtained between the measured and the calculated FWD deflections using the BISAR computer program, for a partial slip condition.

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