INNOVATIVE ASPECTS OF THE CONCRETE PAVEMENT DESIGN STANDARD FOR SÃO PAULO HIGHWAYS

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Abstract

During the last decades there were no official national standards for the design of concrete pavements for highways and streets in Brazil. Due to technical differences between each region, it is necessary to develop consistent research to establish design tools for pavements. Researches carried out last five years could immediately be used to assure highway officials that both rupture criteria and climate-traffic effect models for concrete slabs are reliable and sound for a specific project condition. This paper presents the calibration of stress equations for the design of concrete slabs based on field measurements, models for defining thermal differentials for tropical conditions, and the main results for calibration of a fatigue model for concrete, all in an attempt to determine appropriate rules for the local design of concrete pavements.

Research Significance

The most important result achieved from a set of academic research was the technology transfer for the society. Moreover, the findings are useful to emphasize the need to seek specific design tools for concrete pavements for local highway agencies.

Background

The use of portland cement concrete for road construction in São Paulo began early in the first decade of the twentieth century. Several main roads and streets were then paved using this material up to the early 1950s, when the first petroleum refining plant was opened in the country, allowing the production of asphalt at low costs. Due to the impact of such a competing paving binder, in successive years concrete paving construction for highways has almost stopped.

Although international prices for crude oils have increased in the last decade, self-independence concerning crude oil sources in Brazil will likely avoid excessive prices for asphalt binders. For this reason, price competitiveness of concrete pavement construction has not been significantly improved in relation to asphalt pavements. In addition, the lack of creativeness in concrete pavement

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designs has been a motivation for criticism from official and private road administrations, resulting sometimes in unreal and anti-economical thickness design.

Since there is not an official design method for concrete pavements in the country, the routine option for designers is to request that the cement industry technical association define both slab thickness and concrete properties for a specific project. Moreover, designers are aware that it is not suitable to use AASHTO design criteria (a method developed under a climate regime very different from the prevalent tropical conditions in the country). Therefore, pure mechanistic-forecast approaches should be imported for design, and a fatigue criterion design has prevailed as systematic design fundamentals. During the 1970s and the 1980s, the PCA´66 method was used, and during the 1990s, designers used the PCA´84 method.

In 1998 a major research project began in the country, supported by the State of São Paulo Research Foundation, aiming to establish empirical models for the prediction of thermal gradients in concrete pavements and mechanistic approaches for the prediction of realistic stresses due to the simultaneous action of trucks and environmental loads.

There are about 150 km of the Metropolitan Ring to be built in the coming years, as well as 500 km of new constructions and rehabilitation of pavements using JPCP, and these facts require the systematization of appropriate knowledge for the structural design of such roads and urban streets. Starting from this point, the São Paulo Infrastructure Department (formerly Public Roads Department) decided to begin a full revision of the recently acquired national technology for concrete pavements, and in December 2004 launched its new design standard for concrete pavements, with an emphasis on the employment of stiff structures with unbonded roller compacted concrete base courses. The main features of this new standard, with a major focus on the assembly of new models, are presented and discussed in this paper.

The Basis for Developing the New Standard

All the new developments, comprising field, laboratory and theoretical studies, were developed under the coordination of the Pavement Mechanics Laboratory of the University of São Paulo, from 1999 to 2004, beginning with the construction of two experimental areas of JPCP, the first on the University campus (Site I) and the second one on the SP-79 road (Site II), 100 km from the State Capital.

Monitoring of temperatures on concrete slabs was carried out along eighteen months for Site I (September 1999 to February 2001) and one year for Site II (July 2000 to June 2001). Strain measurements during dynamic load tests were carried out in November 2000 for Site I and in October 2000 and December 2002 for Site II.

At the experimental Site I there were 250 mm thick, 3.5 by 5.5 m slabs supported by a 100 mm well-graded crushed stone base compacted over a lateritic clay subgrade (Balbo and Severi, 2002). Controls of concrete during construction allowed the definition of its average flexural strength and elasticity modulus, respectively 5.3 MPa and 25.76 GPa.

At Site II the concrete pavement was built with slab thicknesses of 230 mm, 3.6 by 5 m slab over a 100 mm RCC base ensuring bond between both layers (composite pavement). The subgrade was a clay-sand soil. Elasticity moduli for such materials were 28.9 MPa and 19.2 MPa, respectively. Both sections were instrumented with strain gages and thermal resistor.

Thermal Differentials in Concrete Slabs

The main results for temperature analysis at Site I were previously published by Balbo and Severi (2002). In this study it was verified that, under tropical (wet-hot) conditions, the nighttime gradients are not significant in absolute values; on the other hand, positive daytime gradients resulted in values higher than those obtained in former studies carried out in temperate climate conditions. The common occurrence of positive gradients during the night was verified by these observations, leading to a frequency of positive gradients of 76 percent in spring, 89 percent in summer, 62 percent in autumn, and 65 percent in winter. As a major result of the temperature research, Severi and Balbo (2004) proposed prediction models for temperature at the top of a concrete slab and also for positive daytime temperature differentials described by the equations as follows.

For top temperature autumn/winter prediction model:

$$TTOP=14.3+0.2*IH+0.75*TAIR-0.07*MOI \quad (1)$$

For top temperature spring/summer prediction model:

$$TTOP=11.94+1.01*IH+0.92*TAIR-0.03*MOI \quad (2)$$

For thermal differential autumn/winter prediction model:

$$\Delta T+=-6.534+0.509*TTOP+0.0013*D \quad (3)$$

For thermal differential spring/summer prediction model:

$$\Delta T+=-18.83+0.542*TTOP+0.037*D+4.165*BMOI \quad (4)$$

For thermal differential alternative full-year prediction model:

$$\Delta T+=-7.883+0.379*TTOP+0.018*D+2.236*BMOI \quad (5)$$

In Eq.1 and Eq.2, TTOP is the maximum top temperature in degrees Celsius, IH is the solar radiation period (in hours) from sunrise to peak top temperature, TAIR is the air temperature (^{o}C) and MOI is the air moisture (in percent). From such models, it can be concluded that solar radiation is related to a top-of-slab temperature increase in warm periods, accounting for 26 percent of the observed variation. In cold periods, solar radiation is not as relevant, accounting for up to 6 percent of the temperature gain; air moisture offers only a weak deterrent to surface temperature increase.

In Eq.3 to Eq.5, ΔT+ is the daytime temperature differential in degrees Celsius, TTOP is the slab top temperature in degrees Celsius (which may be estimated using Eq.1 or Eq.2), D is the slab thickness in millimeters, and BMOI is a dummy variable equal to one when excessive moisture is possible underneath the slab and zero when not. From these models, it is inferred that slab thickness is a more relevant factor for daytime temperature differentials in warm periods than in cold ones. It should be clear from the models that consideration of saturated bases is a relevant concern for wet seasons in tropical environments and not for dry seasons. Such temperature models are employed in the new design method for concrete pavements as presented in this paper.

Field Calibration of Strain and Stresses

Another goal to be accomplished was the need for stress calibration, considering that, from earlier field experimentation in Arlington Experimental Farm, Teller and Sutherland (1943) concluded that field measured strains, for several loads, were systematically lower than the stresses forecasted through the Westergaard equations. Indeed, calibration for analytical or numerical models to permit the prediction of realistic design stresses has been sought even in recent times, as shown by Barenberg and Zollinger (1990); on the basis of the conclusion of this last study, during the research the ILSL2 computer program (Khazanovich & Ioannides, 1993) was chosen as numerical tool.

The comparison between numerical and field strains/stresses was sought through load tests carried out at concrete pavement experimental Sites I and II, as detailed above. Load tests were carried out at Site I on November 11, 2000, early in the morning (6 to 8 a.m.), when the temperature differential between top and bottom of the concrete slabs was null and the uniform temperature distribution was 24^{o}C (equal to curing temperature during construction in 1999). The truck used during the tests was provided with a front axle loaded by 28.06 kN and a rear axle (twin wheels) loaded by 83.58 kN, with wheel pressure of 638 kPa. The position of the truck was recorded by VHS camcorder, which allowed the determination of the exact situation of the truck during the passage on every slab in section E. During the test, the border of the external wheel of the rear axle was located 0.68 m from the longitudinal edge of the slabs, in a constant trajectory, and truck velocity during the test was around 3.3 km per hour.

At Site II, the tests were performed on October 30, 2000, in the morning, by means of measurements of strains generated by loaded trucks leaving the cement plant exit gate close to the site; on December 3, 2002, a new load test was carried out employing a loaded truck with 118.7 kN rear single axle and tire pressure of 633 kPa, passing 0.2 m from the longitudinal edge of the instrumented slab

The strain gage is sensitive to the loads and presents increasing values of deformation when the loads are approaching, and afterwards, decreasing as the load goes away. In Figure 1 an example of the strain-time records is presented as obtained during the tests.

Modeling through finite elements of the pavement load test was done with the ILSL2 computer program. The main criterion for refining the mesh (Bathe, 1982) was the truck loads, following the false ellipse graphical representation (Huang, 1993) formerly employed for the construction of the stress criteria defined by PCA (1984). Figure 2 presents the mesh for slab E1 when the load, during the dynamic test, passed over the slab. The dark areas represent the wheels (rear axle) and the little circles the strain gage positions; one must note that the meshes were defined in order to set a nodal point exactly at the center of each strain gage for each slab, thus allowing the numerical value of the strain to be received at any point of interest related to the field test. Hence, due to the mesh definition, it is possible to compare measured and calculated strains/stresses. The values of strains/stresses computed through ILSL2 were adjusted for the actual position of the strain gages in the slabs.

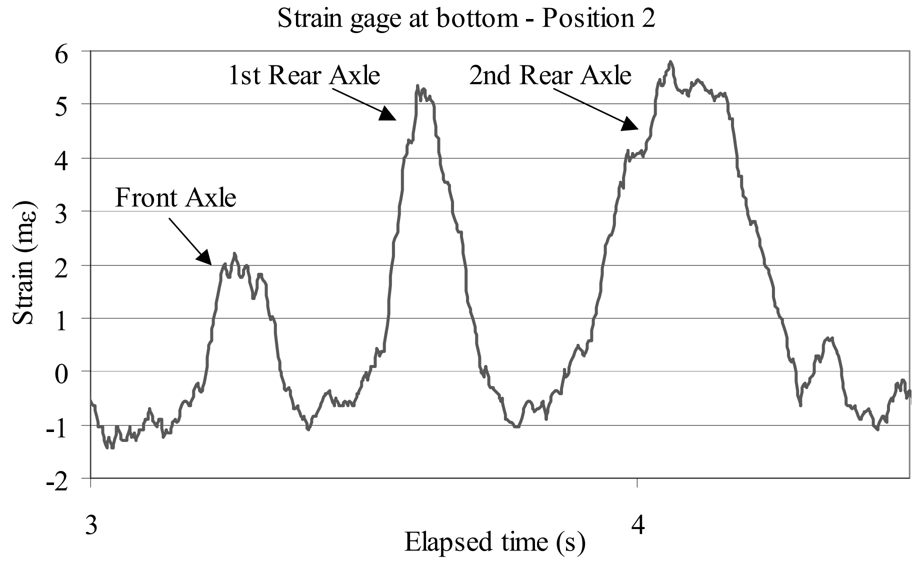

Figure 1. Typical records for strains during the truckload passage (Site II).

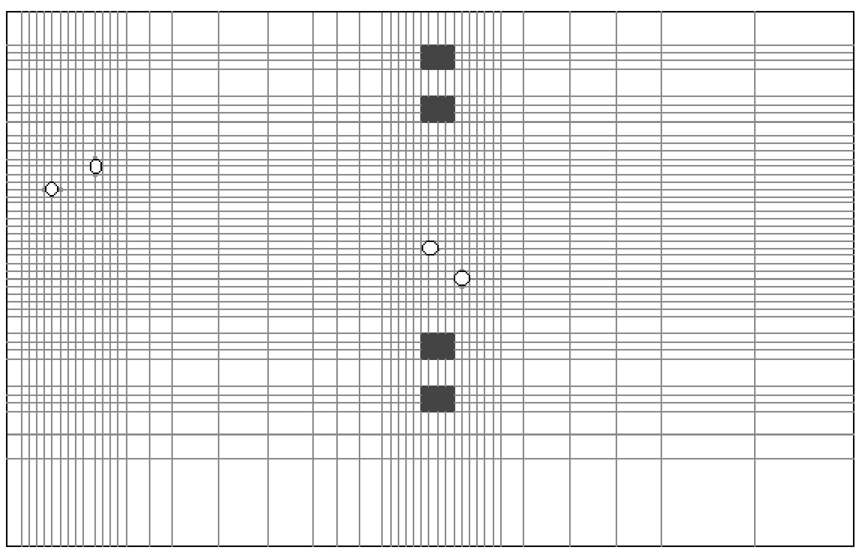

Figure 2. Mesh for analysis of slab E1 with load at the center of slab (Site I).

Stresses on concrete slabs due to strain measured during load passage were calculated by multiplying the specific deformation recorded at one strain gage by the elasticity modulus of the concrete. The total strain due to the maximum effect of the load in each strain gage was obtained by subtraction of the peak of strain minus the initial strain as recorded (see record in Figure 1). It is supposed, for those results, that one measured stress is the total amount of stress in the considered direction added to the stress component of the perpendicular direction due to Poisson effect; therefore, Hooke´s Law was taken into account.

Table 1 shows the results for field and ILSL2 computed strains and their individual ratios for a set of average measurements at Site I. From the achieved results, it can be inferred that, in general, the numerical predicted strains are larger than the field measured ones and the ratios between measured and calculated strains varied from 0.34 to 1.15. Hence, numerical modeling predicts generally larger strains than the actual ones. In field condition, it is reasonable to believe, concrete strength and its elasticity modulus vary more than in laboratory tests, and for field conditions the statistical ratio between the standard deviation and the average of the measured parameter is up to 15% due to the loss of homogeneity during the construction process.

Table 1. Results (Site I) for field measured and ILSL2 computed strains.

Strain gage at bottom	Average Measured Strain (με)	Predicted Strain (με)	ε_{field} / $\varepsilon_{numerical}$
slab E1, at center, x	13.02	27.477	0.47
slab E1, at corner, x	6.26	9.962	0.63
slab E1, at center, y	-3.32	-9.743	0.34
slab E1, at corner, y	4.65	9.076	0.51
slab E2, at center, x	8.41	17.293	0.48
slab E3, at corner, x	18.30	15.969	1.15
slab E1, at center, x	4.18	8.155	0.51

On the basis of such acceptable statistics for variation of concrete characteristics, the standard deviation for the elasticity modulus of the concrete would be 3.86 GPa and maximum statistical value for this parameter could be 29.62 GPa. If one supposes that at the measurement point in the concrete there is an increment of such parameter, the field / numerical ratios in Table 1 would rise to the range from 0.39 to 1.32; note that, during the finite element simulation, the average expected elasticity modulus should be invariable for the analysis (no design takes concrete anisotropy into account).

Nevertheless, other factors may contribute to the verified discrepancies, as possible heterogeneity in the coefficient of Poisson of the concrete as well as the inherent precision of the measuring devices and their relative position in the slabs (this last point was very well controlled during the experiment). The linear correlation (with R^2=0.69) between measured and calculated strains for the average points in Table 1 is:

$$\varepsilon_{\text{field}} = 0.50 \text{ x } \varepsilon_{\text{numerical}} + 1.75 \quad (6)$$

The last equation, for strains at the order of 20 με, set field strains around 60% of the numerical ones. The ratios in the last column in Table 1 result, however, in high standard deviation, which recommends caution in its employment, and it should be more reasonable to assume average plus standard deviation (0.85) as representative value for safety.

Eq.6 above shows that the field strains, statically, resulted in about 75% of values calculated through finite element analysis; for plain concrete pavement using conventional concrete, with flexural strength of 5 MPa, supposing the tolerable range of design stress between 2.5 and 3.5 MPa, the actual stress on slabs approaches the range from 2.13 to 2.98 MPa. The design shall be conservative and the use of "safety load factors" lead to overestimate the design stresses. It is remarkable to note that at the experimental site load transfer on the longitudinal edges occurs solely due to aggregate interlock (no tied shoulder).

For Site II analysis, in Table 2 the numerical values of stresses for a set of ten repetitive tests (in December, 2002) are shown, when wheels passed close to the longitudinal slab edge ten times. Note that, for this pavement, bond between the RCC base and the concrete slab was ensured during construction. The average for the ten field stresses tests (flexural compression), taking into account the generalized Hooke´s law stresses, was -0.94 MPa (at top surface middle longitudinal edge position), with standard deviation of 0.21 MPa.

Relating this average value to the predicted numerical value (Table 2) leads to $\sigma_{field} / \sigma_{numerical}$ approaching 80%, corroborating the results found out at Site I. For this case, results identify evidences of bond between PCC and RCC layers, the same as verified by Barenberg and Zolliger (1990), although these authors disclosed loss of bond in their field experiment some years later. If one takes Hooke´s law relating stresses and strains, for the ratio $\varepsilon_{field} / \varepsilon_{numerical}$ equals to 0.85 herein defined, it results in:

$$\sigma_x^{field} = \frac{E}{1-\upsilon^2} \times \left(\varepsilon_x^{field} + \upsilon \times \varepsilon_y^{field} \right) \quad (7)$$

$$\sigma^{field} = 0.85 \times \sigma^{numerical} \quad (8)$$

Therefore, the new standard suggests the reduction factor of 0.85 for numerical computed stresses to be applied during thickness design through fatigue analysis.

Table 2. Numerical computed stresses (ILSL2) for load tests at Site II (Dec., 2002)

Position of load (axle)	Top stress (MPa)	Bottom stress (MPa)
Longitudinal edge	**- 1.22**	0.62
One meter from longitudinal edge	- 0.50	0.24

MEF Generated Equations for Computing Stresses

The new method strongly recommends that designers search slab stress values through employment of appropriate numerical tools; nonetheless, the standard offers new equations for critical stress computation. Four types of numerical-statistical models have been sought, all of them considering the use of cemented RCC bases for the concrete pavements, in addition to considering a broad range for the characteristics of cemented bases when this layer is not to be bonded to the concrete slab.

Bands for thermal gradients at daytime were simulated taking into account the values obtained in the parallel temperature study at the experimental Site I (Balbo and Severi, 2002). The standard axle of 80 KN was considered during simulations and the slab dimensions were 5.5 x 3.5 m, with transversal joint transfer through dowels. Two possible situations for stress analysis were considered in this study: cemented base not bonded to the slab and the bonded base case. The factorial design of parameters for simulations of bonded and unbonded base cases resulted an amount of 14,256 simulations for the unbonded case and 17,820 for the bonded case, with the following ranges for the parameters: slab thickness, e1, from 0.15 to 0.35 m; base thickness, e2, from 0.10 to 0.30 m; total load of single axle, Q, from 60 to 100 kN; PCC elasticity modulus, E1, from 28 to 32 GPa; cemented base elasticity modulus, E2, from 10 to 28 GPa; modulus of subgrade reaction, k, from 25 to 130 MPa/m; linear thermal differential on slab, DT, from 0 to 25º C.

Some explanation is due for the different factorial designs. When the cemented base is bonded to the slab, it is necessary to consider that the base material is not a common cemented base as, for instance, soil cement, thus requiring stiffer bases; besides that, for the bonded case, the slab system will be thicker and thermal differentials are therefore larger. For reduction of the PCC slab thickness when bonded to the base, on the other hand, thicker cemented bases should be required, and this fact was taken into account in factorial design. Concerning the modulus of subgrade reaction, it should be noted that when the base is not bonded, as suggests the PCA (1984) method, the cemented bases expressly increase the k-value, whereas for the bonded case, there will be only one slab (monolithic and composite concrete plus cemented base), and the support shall be due exclusively to the foundation.

The statistical models that presented the best fit (R^2 close to 1) for the maximum stresses simulated through ILSL2 are given in Table 3, where x_i (i = 1. . .8) represents the multiple linear correlation coefficients (constants). Table 4 presents the regression coefficients for prediction of PCC stresses (for k=30 MPa/m) for the case of unbonded interface between the slab and the base. The coefficients for the bonded cases as well as coefficients for prediction stresses in the base layer can be found elsewhere (Sao Paulo Department of Infrastructure, 2004).

Table 3. General form of equations for computing stresses in concrete pavements.

Case	Layer	Generated stress equation (MPa)
Not bonded base	Slab	$\sigma = I + x_1 Q + x_2 e_1^2 + x_3 e_1 + x_4 e_2^2 + x_5 e_2 + x_6 E_2^2 + x_7 E_2 + x_8 DT$
	Base, DT≠0	$\sigma = I + x_1 Q^2 + x_2 Q + x_3 e_1^2 + x_4 e_1 + x_5 e_2^2 + x_6 e_2 + x_7 k + x_8 DT$
	Base, DT=0	$\sigma = I + x_1 Q^2 + x_2 Q + x_3 e_1^2 + x_4 e_1 + x_5 e_2^2 + x_6 e_2 + x_7 k$
Bonded base	Slab	$\sigma = I + C_1 e_1^3 + C_2 e_1^2 + C_3 e_1 + C_4 E_1 + C_5 E_2 + C_6 k + C_7 DT$
	Base	$\sigma = I + C_1 e_1^3 + C_2 e_1^2 + C_3 e_1 + C_4 E_1 + C_5 E_2 + C_6 k + C_7 DT$

The former method released by PCA (1984) is very well documented concerning its concepts and formulation. The equivalent stress concept was employed taking into account the fatigue consumption due to single axles placed on the longitudinal edge and comparing the summations for fatigue prediction for the same axle located at the actual position on the slabs, resulting in a conversion factor of 0.89 (to convert critical stresses on slabs computed through J-Slab finite element program) to define the so-called equivalent stress.

For simulation of stresses on slabs a standard axle of 80 kN was taken. The original publication points out that thermal gradients were not taken into account at that time due to imprecision in the determination of thermal stress (and also due to lack of precise information about that matter as well). Nowadays, after several field research efforts in the last two decades, as well as the enormous improvement of numerical tools, such limitations can be easily dealt with. The models developed for this study are a kind of proof of that, and they could be developed since their prediction potential has proved to be very suitable by confrontation to field measured stress.

It is necessary, however, to compare the new models to the former one since the latter has been used with good results for the last fifteen years in the country. So when a comparison is made with regard to the traffic load, as shown in Figure 3, with zero thermal differential, the results are very close between the present models and the PCA (1984) method (stress analysis parameters: k= 30 MPa/m; e_2=0.1 m, E1= 2.8 GPa and E2=2.4 GPa; no concrete shoulder). The advantage of the present equations is the simultaneous computation of load and thermal stresses since values for the thermal differential are available, like the above mentioned prediction stresses for tropical conditions.

Table 4. Regression coefficients for unbonded concrete slab stress

x_i	Thickness of Concrete Slab (m)		
	0.15 ≤ e1 ≤ 0.19	**0.19 ≤ e1 ≤ 0.25**	**0.25 ≤ e1 ≤ 0.35**
I	4.827795544	4.474855105	4.529942884
x1	0.024437136	0.019479424	0.01392786
x2	8.557526083	20.1091493	18.40750991
x3	-16.32412849	-20.62885666	-19.68103271
x4	-11.73952917	-14.48337187	-5.876595139
x5	-6.558617848	-0.146208073	0.494847813
x6	5.7254 E-10	1.81825 E-10	3.14652 E-11
x7	-4.94444 E-05	-2.00845 E-05	-5.10837 E-06
x8	0.156020102	0.139112935	0.104822411
R^2	0.977687545	0.98538211	0.980722442
SD	0.163185743	0.110052464	0.097753748

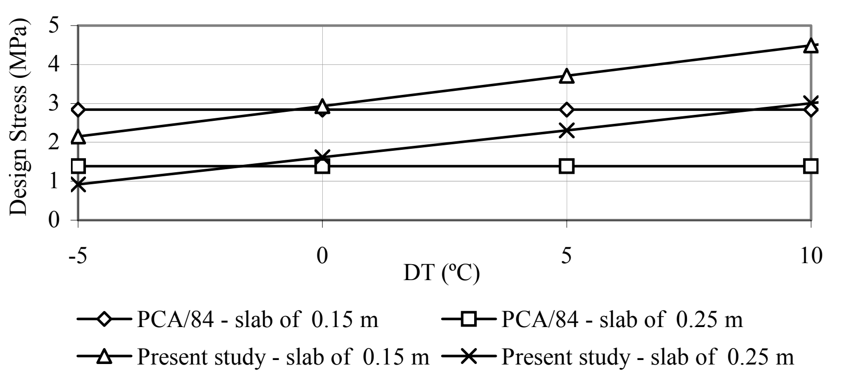

Figure 3. Stresses on concrete slabs for 80 kN standard axle.

Fatigue Model Development

In the PCA method (PCA, 1984), the concrete fatigue model, as illustrated by Packard and Tayabji (1985), is similar to the 1966 version with a modification from 500,000 to 10,000,000 load repetitions to avoid discontinuity and the possibility of unrealistic results given by the original equation. This model proposed an unlimited number of load repetitions for SSR lower than 0.45.

In order to verify the reliability of the PCA´84 fatigue equation to predict fatigue for typical concrete mixture standard (Table 5) in use nowadays for highway construction in São Paulo, a laboratory study was carried out by Cervo (2004) seeking the experimental performance of this concrete in fatigue. 105 prismatic samples of the concrete (flexural strength varying from 4.2 to 5.1 MPa) were tested at 10 Hz frequency by using a servo-controlled MTS Universal machine with 10,000 kN ultimate load capacity. Tests were performed for stress-strength ratio (SSR) allowing from 100,000 to 1,500,000 load cycles to fatigue. As shown in Figure 4, rupture in the samples always occurred in the middle of the span.

Table 5. PCC characteristics and properties.

Materials or Properties	Design values
Cement type 40 MPa with slag addition (kg/cm)	328
Fine quartz Sand (kg/cm)	691
Granite crushed stone diameter up to 12 mm (kg/cm)	483
Lime crushed stone diameter up to 25 mm (kg/cm)	724
a/c ratio	0,553
Plasticizer (L/m^3)	1,148
Air entrained (%)	2,6
Slump (mm)	65
28-day flexural strength (MPa)	4,8

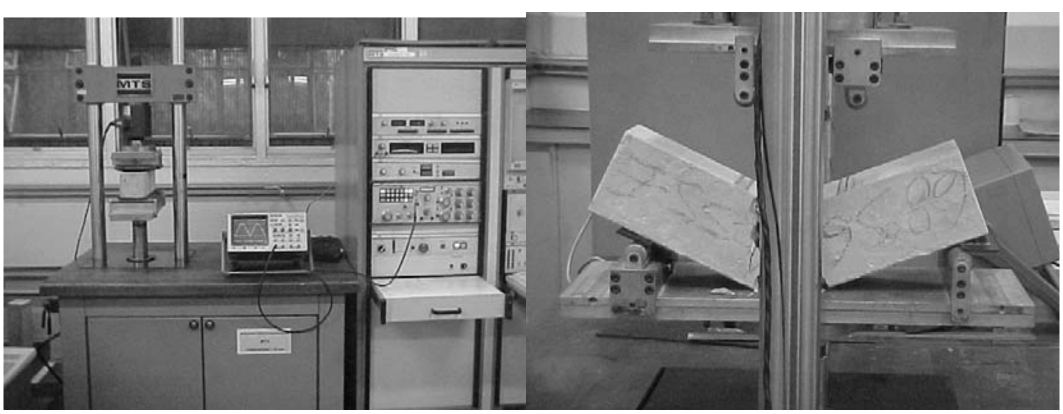

Figure 4. Fatigue tests in laboratory using an MTS machine.

The resulting log-linear model by regression of points was:

$$\log \text{N} = 25{,}858 - 25{,}142(SSR) \tag{7}$$

where N is the number of cycles to fatigue, and the regression equation resulted in standard error of 0.18 and a determination coefficient (R^2) of 0.91. In Figure 5 the resulting points from tests are presented as well as a comparison of Eq.8 with the PCA´84 fatigue model, from which the understanding of a less conservative result for the new fatigue model is more clear. Several details concerning the recent fatigue tests are under submission for publication in another paper. Due to results, Eq.8 was adopted for the Brazilian design standard, leading to thinner slabs when thermal gradients are not computed. However, based on former analysis carried out under field conditions (Balbo, 2003), Cervo (2004) defined during her work the following shift-factor for converting laboratory-field fatigue performance:

$$N_{FIELD} = \left(\frac{1}{SSR}\right)^{-4,20231} .N_{LAB} \qquad (9)$$

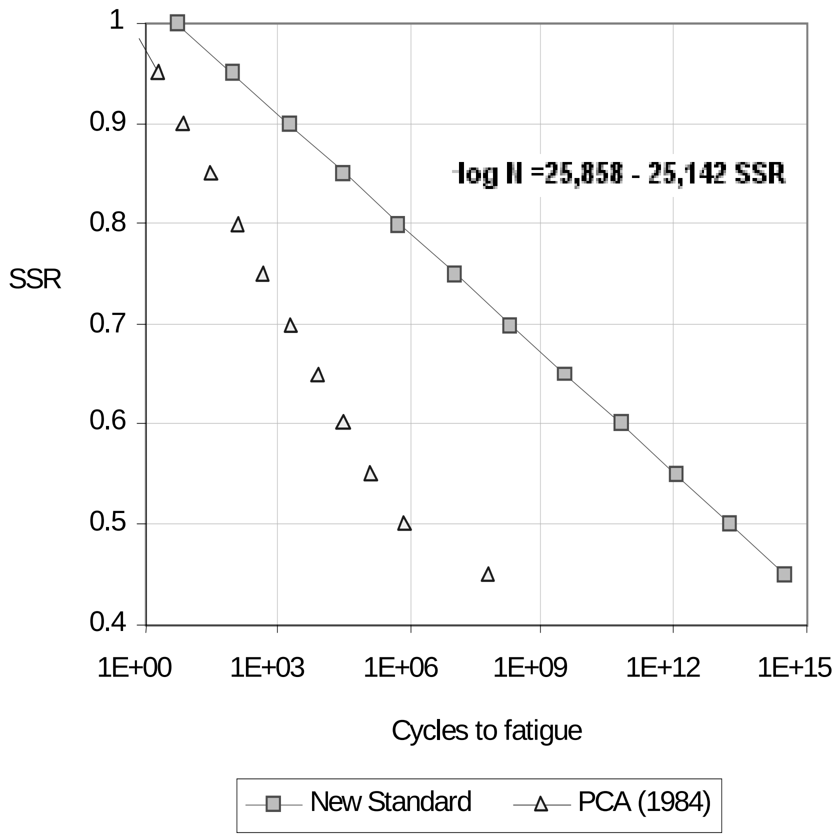

Figure 5. Fatigue model graphical representation.

New Design Method applied to the Metropolitan Ring (Design Review)

During the first construction phase of the São Paulo Metropolitan Ring, due to the lack of a National or Regional standard, designers were induced to consider the PCA´84 method for the design, as it was the only design tool available at that time. The anticipated traffic for a 30-year design period was determined, as shown in Table 6.

The design was carried out for JPCP employing a load safety factor of 1.25 (as suggested by PCA), k-value of 80 MPa/m, and concrete 28-day flexural strength of 4.5 MPa over a non-bonded RCC base 100 mm thick. The design model prescribed 240 mm slab thickness for a fatigue consumption of 52% during the design period; for a 230 mm slab, fatigue consumption should be 130%. Erosion analysis did not result in the main deterioration criterion for the design (due to the RCC base not being subjected to contamination).

Employing the equations herein presented as the new standard design method for the same conditions, through a consideration of the proposed shift-factor and the zero thermal gradient analysis (as in the PCA´84 method), the resulting thickness design for the Ring project should be 190 mm for a fatigue consumption of 8% (180 mm leads to 144%). Taking into account the thermal gradients in São Paulo along the seasons as proposed by Balbo & Severi (2002), the new design for the Metropolitan Ring should be 260 mm, i.e. a thicker slab than the ones built in the first phase of construction. If one takes into account the PCA´84 fatigue model and the stress models presented here thickness should be 290 mm. Moreover, it should be remarked that there are several tunnels on the highway, and the pavements, not subjected to direct solar radiation in these cases, should be thinner than the 240 mm actual design, perhaps 190 mm as calculated above.

Table 6. Traffic for design period (30 years) on Metropolitan Ring.

Single axle		**Tandem axle**		**Tridem axle**	
Axle load (kN)	**Repetitions**	**Axle load (kN)**	**Repetitions**	**Axle load (kN)**	**Repetitions**
160	53,930	250	0	300	0
150	0	240	0	290	196,159
140	0	230	0	280	1,410,943
130	35,593	220	72,262	270	7,900,085
120	1,092,574	210	0	260	7,001,308
110	9,624,589	200	108,252	250	3,967,070
100	12,144,396	190	571,575	240	1,896,374
90	4,869,033	180	6,789,975	230	1,321,812
80	3,233,295	170	7,269,832	220	1,492,945
70	3,584,501	160	5,590,083	210	1,450,947
60	30,386,168	150	3,838,582	200	1,163,743
50	34,445,515	140	2,386,231	190	520,292
40	23,522,315	130	2,378,167	180	749,405
30	16,834,162	120	2,260,820	170	718,988
20	5,236,155	110	2,525,827	160	639,623
10	0	100	12,308,868	150	2,418,903
-	-	50	1,220,807	100	2,366,073
-	-	-	-	50	491,815

Concluding Remarks

Improvements in the design criteria for computing stresses in concrete pavements were proposed as desired by several governmental agencies in the country. These design improvements took into account the level of reliability presented by numerical tools for the prediction of bending stresses in JPCP due to traffic loads and thermal curling (during daytime). The new design standard was entitled

“Design Instruction 07 – Dimensioning of Concrete Pavements” by the São Paulo Infrastructure Department.

Before computational simulations, a concrete pavement experiment was conceived to allow the analysis of discrepancies between field actual stresses and numerically predicted ones, which allowed the conclusion that field stresses are slightly smaller than the numerically predicted ones; comparison of stress values showed it to be possible to assume the calculated stresses as the design stresses since there will be variability among the concrete properties, such as the elasticity modulus or Poisson coefficient, as discussed.

Since the mechanistic approach has a field validation for its employment without risks during concrete pavement design, it was reliable enough to proceed with the development of new equations for computing stresses on concrete pavement structures with regard to the special cases of bonded and unbonded cemented bases. The approach also allowed us to appropriately consider the effects of daytime curling effects on pavements. Results have shown that the new stress models herein proposed agree very well with the similar approach developed by PCA (1984) two decades ago when thermal effects are neglected; nevertheless, the new models have the advantage of defining the design stresses also for the cemented bases and for the combined load and thermal stresses for each layer, either bonded or not.

In addition to the above, the research sought out a more sound fatigue model for the typical concrete mix employed in the region. The fatigue test results and calibration pointed out a less conservative equation for the design of JPCP. However, the new standard accomplished recently shall be put under general field investigation, by monitoring the newly built concrete pavement, in order to make it possible in the future to add new degradation mechanistic models to the method, as in the cases of pumping and faulting; these are the next steps for a long term research.

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