

EXPERIMENTAL ANALYSIS OF STRAINS ON WELDED WIRES IN THIN REINFORCED CONCRETE PAVEMENT

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Abstract

This paper presents the experimental results of strain and stresses in steel bars gotten from a full instrumented reinforced concrete pavement experimentally built on the Federal Highway BR 232/PE, in the Northeast of Brazil. Contact strain gages were placed at several edge longitudinal and tangential wire reinforcements and load tests were accomplished one month later in order to check design stresses. Results are very clarifying and show how steel bars are tensioned and share flexural responses of slabs between concrete and steel reinforcement. This kind of solution, rarely speculated in past technical literature or through field experiments, seems to be an easy and effective alternative for constructing thin concrete slabs when required due to limitations on grade levels, especially for whitetopping constructions. The bars, designed using conventional reinforced concrete design theory, did not present any case of over deformation inducing its fracture when comparing strains limits and field induced strains by loads.

Research Significance

Day after day, reinforced concrete pavements are providing a new alternative for concrete paving with main concerns towards industrial heavy duty floors. However, there was not a specific design criterion for this kind pavement and cares concerning steel bar design should be addressed. The field experiments were able to convey safety in using the ultimate strength limit theory for the design and dimensioning of the structural passive steel reinforcement for slabs-on-grade.

Introduction

Engineering concerns for the rational employment of reinforcing steel meshes on concrete slabs on grade first appear in the work presented by Bradbury (1938) with special regards to issues such as curling stresses by direct application of Westergaard's temperature theory (1927). Nevertheless, for decades the practical use of reinforced slabs for pavements was uncommon, taking as design criterion the ultimate load theory for defining the steel reinforced section of concrete slabs, when the combat to load and environmental issues imposed stresses on slabs is prior to the steel mesh and allowed partial cracking on concrete (ACI, 1999).

Later in the seventies, Losberg (1978), faculty of the Chalmers Technology University, Göttemburg, Sweden, published a practical paper proposing the use of the *Yielded Line Theory* as the foundation for defining ultimate cracking stresses on concrete slabs sections, allowing the definition of flexural moments and afterwards the steel resistant area for the imposed loads on slabs on grade. Reinforced slabs-on-grade theories were also formerly studied by Johansen (1962), Meyerhof (1962) and Baumann and Weisgerber (1983). All these developments have been used by designers when considering floors on grade projects in civil construction.

Still rare nowadays are pavement projects taking into account the use of welded wire meshes placed at the bottom of concrete slabs to reach a resistant section to bear the external induced stresses in the pavement structures. On the other hand, the use of steel meshes is widely used for continuously reinforced concrete pavements, when the steel rule does not resist the load effects but keeps strictly close to the crack-induced slab sections by concrete shrinkage, when the steel is placed over the mid depth of the slab thickness.

The paramount difference between both reinforcement concepts for pavements is that, when using a negative moment resistant steel mesh (at the bottom of slabs), some huge modifications are possible in the construction technique; among the relevant ones, the increasing span between transversal and longitudinal joints (the steel compensates for concrete shrinkage), reduction of concrete slab thickness (the flexural stress is to be resisted primarily by wires and not by the concrete itself) and, likely, the employment of more typical structural concretes (around 25 MPa compressive strength) with evident reduction in cement content for concrete mixtures. Aspects as those are strongly attractive for whitetopping construction in urban roads and streets as well as heavy industrial floors restoration.

On the standpoint of construction costs, effectiveness must be analyzed case-by-case once welded wire meshes and their transportation to the construction site represent a significant cost compared to jointed plain concrete slabs. However, recent experiences from the last decade using reinforced concrete pavements using steel meshes, were able to disclose an easy-to-build construction system as well as a durable solution, with some advantages in respect to construction costs to other solutions in the case of heavy duty industrial floors (Marin & Balbo, 2004). It is convenient to mention some projects, as presented in Table 1, where one can realize the reduced thickness of such concrete pavements; thickness between 130 and 160 mm for highway and urban vehicles are common while airport and heavy equipment terminals require 200-250 mm.

Besides the growing industrial use of reinforced concrete pavements, this solution has been successfully employed in eight highway tools as well as in some bus corridors and terminals in several cities in the country, increasing engineering concerns about designing this kind of pavement during the last decade. More than twenty industrial floors have been built since 1996 only in the State of São Paulo, Brazil. Unfortunately, the projects are too recent to issue a definitive judgment about their performance.

As a new solution for paving, as usual, most practicing engineers have been concerned about aspects such as coherent design theories and construction suitability of such reinforced pavements using modern concrete pavers, and both aspects were explored in the recent construction of two new lanes in a federal highway (BR 232/PE), when a 500 m-long experimental section of reinforced concrete slabs was designed and built. One out of 33 slabs was fully instrumented in order to achieve and verify the compatibility of the strain level in steel meshes.

Figure 1 presents some pictures taken during the concrete laying by using a slip-form paver, showing the feasibility of reinforced concrete slabs using conventional slip-form pavers; the concrete thickness was 160 mm and the slump was adjusted to 80 mm, which may be a little high for the common application with slip-form equipment. However, no finishing problems were found out during the construction as is observed in Figure 1. For urban streets and industrial floors, screed-laser equipment was used successfully, and no issues seem to be present nowadays under the construction suitability of such reinforced pavements.

Table 1. Examples of reinforced concrete pavements built in Brazil recently.

Year	Project	Concrete thickness (mm)	Steel section (kg/m ²)	Pavement base type	Average daily traffic/movements
1997	Road access to a cement plant – Paraná	140	6.0	cement treated aggregate	200
1998	Bus corridor – Curitiba	140	6.0	crushed stone	450
1999	Bus corridor – São Paulo	150	5.3	old asphalt pavement	950
1999	Maintenance area – São Paulo airport	160	11.0	cement treated aggregate	220
1999	Highway toll pavements – São Paulo state	125	6.7	rolled compacted concrete	7,500
1999	Bus terminal – Paraná	140	6.0	crushed stone	150
1999	Bus terminal – Rio Grande do Sul	170	6.5	crushed stone	150
2000	Freight terminal – Fortaleza airport	150	7.0	crushed stone	-
2000	Urban avenue – Rio de Janeiro	140	6.0	crushed stone	1,500
2001	Bus stops and terminals – Manaus	140	6.0	soil-aggregate mix	340
2001	Aircraft Maintenance Center TAM Airlines – Sao Paulo	200	9.4	cement treated aggregate	-
2001	Bus Terminal – Carapicuíba	140	6.0	crushed stone	650
2002	Bus corridor – São Paulo	140	7.6	old asphalt pavement	500
2003	Bus corridor - São Paulo	125	7.0	old concrete pavement	2,700
2007	Heavy Cellulose industrial floor – Ipatinga, MG	250	9 to 12	rolled compacted concrete	1,800



Figure 1. Using of slip-form paver for laying concrete over steel welded mesh (construction site managed by the authors; pictures by Ganassali).

This paper covers the analysis of strain and stresses in steel welded meshes and seeks for comparison between the field physical measured parameters and design stresses as well as stresses predicted by finite element model using ISLAB 2000. A fundamental discussion is presented about the usage of double meshes at slabs longitudinal borders on the basis of received strains during field tests. The main purpose of the experiment was to check the criteria in use for the design of reinforced concrete pavements as well as to address construction issues.

Theoretical Background

Strains and Forces in Reinforced Concrete Rectangular Sections

Figure 2 shows the possible limits on concrete shortening and steel stretching in a reinforced concrete rectangular section. Possible combinations could be divided in five domains. In domain 1, the section will be full in tension, both in concrete and in steel; an opposite situation of full compression in the section is called domain 5. Between these extreme possibilities, there will be conditions in which the section is partially in tension (lower section) and partially in compression (upper section), defining a neutral line or axis (LN). Those intermediate domains are taken as domains 2, 3 and 4.

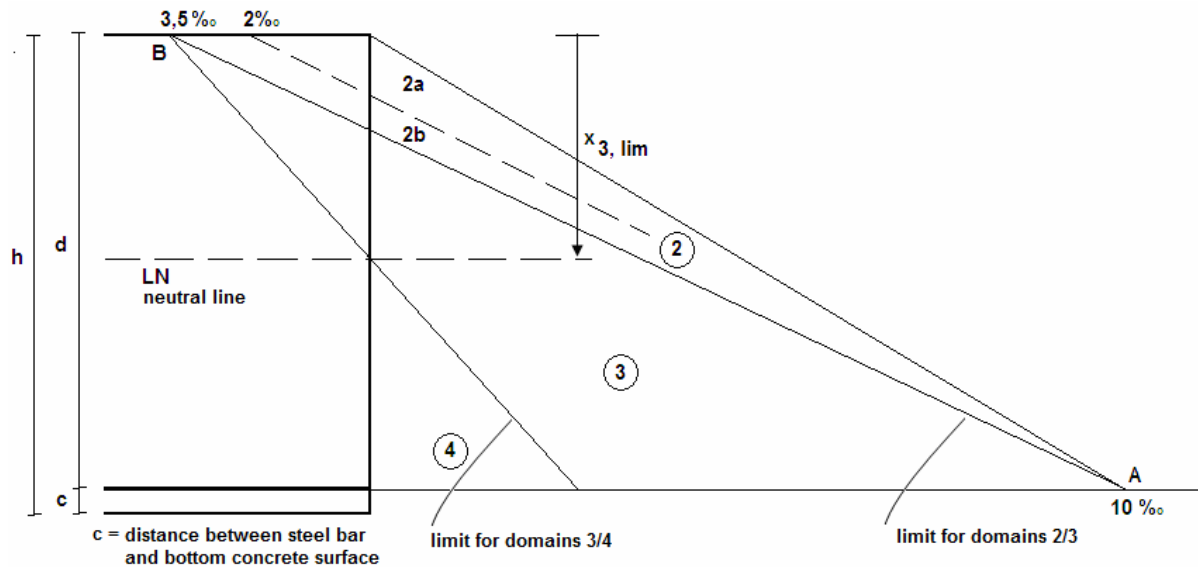


Figure 2. Domains of normal efforts in a rectangular transversal section of reinforced concrete.

Between limits 2 and 3, a condition occurs when the beginning of steel (bar) stretching is found and the upper part is under a shortening of 3.5‰, leading to an ultimate deformation design criteria for both concrete and steel, allowing the full economic use of materials. The strains are ϵ_{cd} (compression in concrete) and ϵ_{yd} (traction in steel bar). Note that h is the height of the rectangular section and d is the distance between the top compressed concrete fiber and the steel bar position, while c is the assumed covering distance from the steel bar down to the concrete fiber. The neutral line position (x) is measured from the top and can be defined by the dimensionless parameter:

$$\epsilon_{cd} = \frac{x}{d} \quad (1)$$

From Figure 2, the relations for strain in steel and the limit for neutral line position are given by the relation:

$$\frac{0.0035}{x_{3,\text{lim}}} = \frac{\varepsilon_{yd}}{d - x_{3,\text{lim}}} \quad (2)$$

By replacing eq. (2) by eq.(1) the following can be found:

$$\xi_{3,\text{lim}} = \frac{0.0035 \times d}{0.0035 + \varepsilon_{yd}} \quad (3)$$

As verified, in domain 3, the neutral line position within the concrete section is dependent on the steel stretching limit (ε_{yd}); however, one shall note that this value is dependent on the grade of steel to be employed. Table 2 presents some values of $\xi_{3,\text{lim}}$ for different steel grades, as classified in Brazil according to the NBR 6118 standard (ABNT, 2003).

Table 2. Relative position of the neutral line according to steel category.

Steel grade	f_y (MPa)	f_{yd} (MPa)	ε_{yd}	$\xi_{3,\text{lim}}$
CA-25	250	217	1.03	0.7726
CA-50	500	435	2.07	0.6284
CA-60	600	522	2.48	0.5853

For domain 2 (subdivided into sub-domains 2a and 2b in Figure 2), the ultimate limit is given by the steel strain $\varepsilon_{sd} = 10\text{‰}$. For such a condition, supported by Figure 2, the following can be written:

$$\frac{0.0035}{x_{2,\text{lim}}} = \frac{0.01}{d - x_{2,\text{lim}}} \quad (4)$$

In this case, the relative position of the neutral line within the section is independent of the steel category, and given by $\xi_{2,\text{lim}} = 0,2593$. Exactly on the limit between sub-domains 2a and 2b, the following can be written:

$$\frac{0.002}{x_{2a,\text{lim}}} = \frac{0.01}{d - x_{2a,\text{lim}}} \quad (5)$$

And it is inferred that $\xi_{2a,\text{lim}} = 0.1667$. It is worth observing that within sub-domain 2b a pseudo plastic rupture occurs within the concrete.

Forces Equilibrium in Transversal Section

If the slab (or section) is admitted to respond by hinging (bending) and there is no need of steel compression at the top level, the forces equilibrium can be determined around the steel section in

Figure 3 by using a simplified rectangular diagram of tension distribution at the compressed area of the section. This equality of acting forces is given through:

$$R_c = R_s \quad (6)$$

Since the bending moment acting within the section is previously known (by use of analytical or numerical method, elastic or plastic regimen), by equilibrium of moments the following can be written:

$$M_d = R_c \cdot (d - a) = 0.85 \times f_{cd} \times 0.8 \times x \times b \times (d - 0.4 \times x) \quad (7)$$

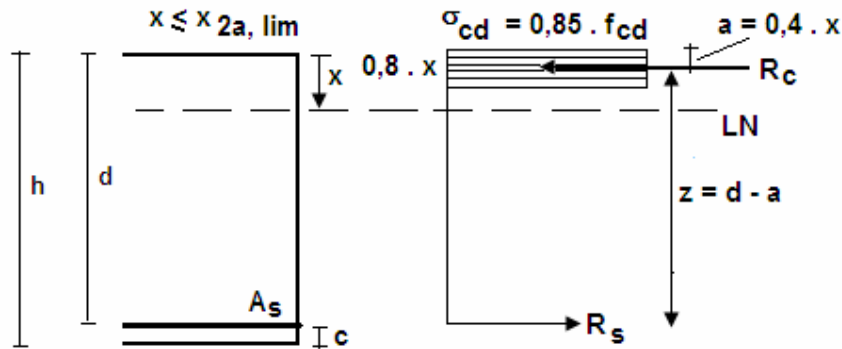


Figure 3. Rectangular diagram of compression forces (R_c) and steel response (R_s).

By replacing eq. (1) by eq. (7), the result is:

$$M_d = 0.68 \times f_{cd} \times b \times d^2 \times \xi \times (1 - 0.4 \times \xi) \quad (8)$$

Once the domain is chosen, eq. (8) is used to verify the compliance of ξ to the value of ξ_{lim} defined by eq.(3), (4) or (5). Since the verification prevails, the definition of the steel area within the section is achieved by means of eq.(6), rewritten as:

$$R_c = \frac{M_d}{(d - a)} = A_s \cdot f_{yd} = R_s \quad (9)$$

Therefore, if the values for d and ξ are known, and taking into account that $a = 0.4x$ and $x = \xi d$, the steel section area remains defined per unit of length (as the moment is also per unit of length) as:

$$A_s = \frac{M_d}{(d - 0.4 \times \xi \times d) \times f_{yd}} \quad (10)$$

Note that the deductions presented above are adequate for conditions of inexistence of normal force, eccentric or not, in the transversal section. Forces due to curing shrinkage or expansion as a result of temperature changes should be considered separately. It is also important to

acknowledge that the “moment of calculus”, in reinforced concrete design, is defined as the critical moment within the section; also, its calculated value is generally majored by a safety factor, namely, 1.4. Is worth noting, too, that f_{yd} is the reduced ultimate strength for the steel, generally given by the division of the steel characteristic strength by the safety factor of 1.15 (Wire Reinforcing Institute, 2006).

The acting moment in the reinforced concrete section can be defined by several means, both by Westergaard or Losberg theories and by numerical tools using plate finite elements. Since critical stresses are the main parameters for experimental analysis, the relation between stresses and moments is addressed by the expression:

$$\sigma_f = \frac{6M_d}{h^2} \quad (11)$$

Years ago, before the accessibility to numerical tools, moments due to loads were seldom defined through finite elements, and the usage of Pickett and Ray (1951) charts was almost mandatory for a variety of wheel loads and axles. Currently, closed form finite elements programs are available, allowing designers to specify issues such as temperature differentials for days and seasons. Since top steel is not required in design, at least shrinkage compensation meshes are designed to take position at the slab top, as suggested by Losberg (1978). For defining the steel area for shrinkage control, the following drag equation is used:

$$A_{sT} = \frac{W \times f \times L}{2 \times f_y} \quad (12)$$

where A_{sT} is the steel area per unit of length (meter), W is the slab weight per square meter, f is the friction resistance coefficient for the steel, L is the slab length and f_y is the steel nominal strength.

Experimental Project

Design Parameters

The experimental reinforced concrete pavement section was built in a 500 m length section at BR-232 highway, resulting in 32 slabs of 15 m x 7.2 m and one last slab of 20 m x 7.2 m for the test. This last slab was instrumented prior to laying concrete and the construction process was manual. For the rest of the section, a slip-form paver was employed as shown in Figure 1. Transversal dowelled joints were cut by diamond saw-cut machine. Pavement structure, as shown in Figure 4, consists of reinforced concrete slab over rolled compacted concrete base laid on the subgrade. The concrete slab critical design moment for longitudinal edge was 11.56 kN.m/m for a rear single axle truck of 100 kN, representing flexural stress of 2.71 MPa according equation (11) aforementioned. Such value for design moment was originally defined through Pickett and Ray (1951) chart for free edge. Specified welded steel wire (meshes) presents rupture stress at 600 MPa. The concrete compressive strength after 28 days was 30 MPa.

Instrumentation Design

As aforementioned, the last 20 meter long slab, handily built, was instrumented in order to allow the measurements of strains on steel bars during load tests with loaded truck. Contact strain gages were bonded to the bars using epoxy resin and then wrapped with liquid silicone (Figure 5) in several positions close to the slab longitudinal edge. The design for instruments positions were carried out

taking into account the effects, close to the slab edge, of truck axles, using the ISLAB 2000 finite element program (with improvements as those provided by Khazanovich & Ioannides, 1993).

Three zones were considered more critical for longitudinal edge stresses after several numerical simulations: close to the transversal joint (position 1), at one quarter of the slab length (position 2) and in the middle of the longitudinal joint (position 3). During numerical simulations, two truck axles effects were evaluated: single rear axle (13 kN) and a tridem axle (30 kN), actually considering the overloaded trucks in the highway with no weight control. Table 3 presents the values for flexural stresses obtained through simulations of ILSAB 2000, where stress in x and y directions are related, respectively, to transversal and longitudinal directions. Figure 6 shows a graphical representation of stresses for the simulation when the single axle is placed at the mid longitudinal edge. Figure 7 shows the instruments location coordinates within the slab.

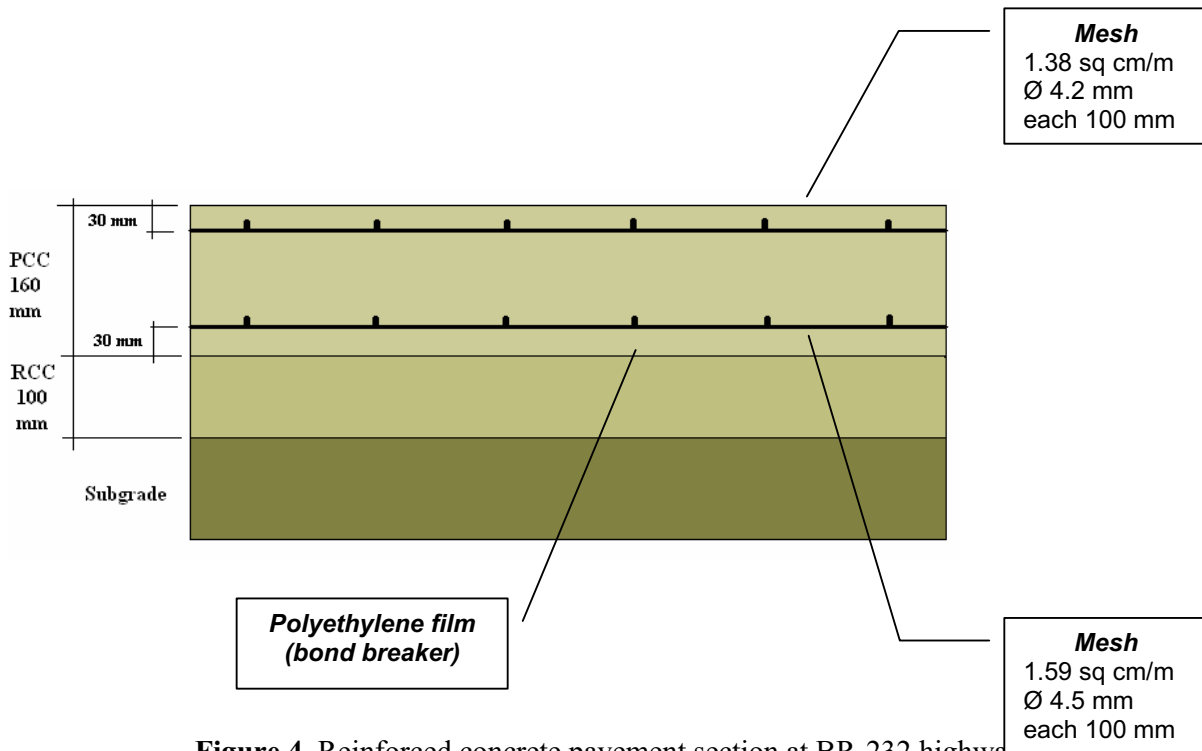


Figure 4. Reinforced concrete pavement section at BR-232 highway.

Table 3. Predicted stresses for concrete slab (20 m x 7.2 m).

Axle	Position	Maximal stresses at slab bottom (MPa)	
		Direction x	Direction y
Single	1	1.70	1.04
	2	1.26	3.27
	3	1.05	2.40
Tridem	1	1.44	1.83
	2	1.26	1.88
	3	1.02	1.88

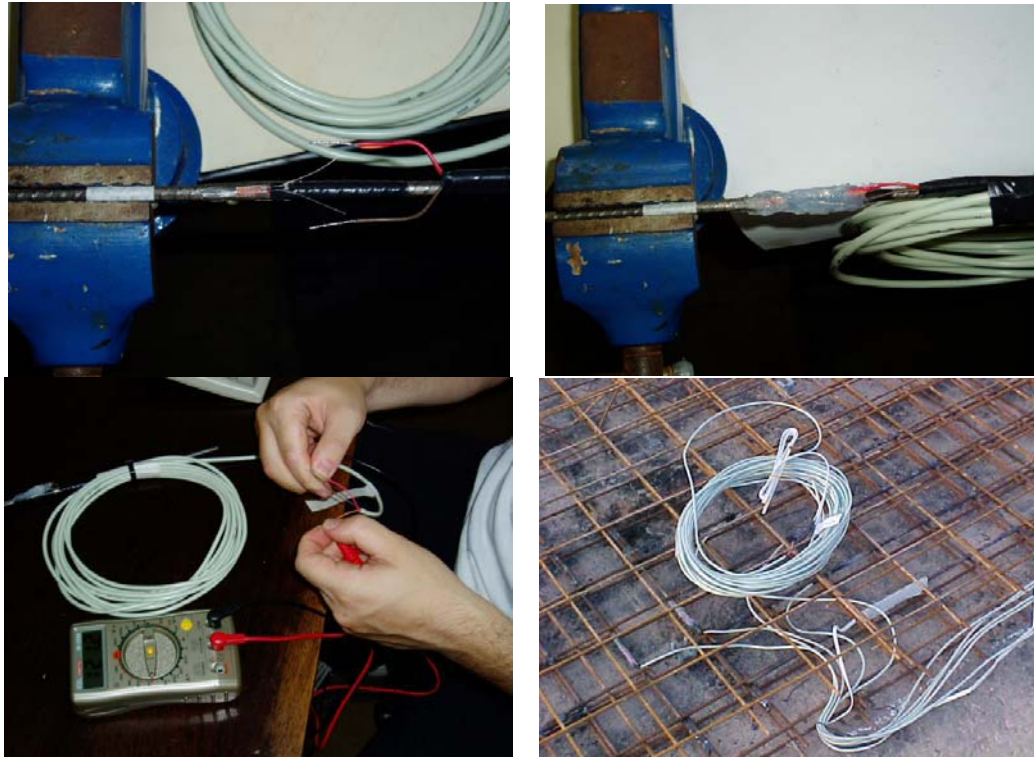


Figure 5. (a) preparing for welding (b) wrapped with silicone (c) checking for strain gage impedance (d) view of instrumented steel bar in field.



Figure 6. Graphical simulation of normal stresses (longitudinal direction on slab, position 3).

One can realize from the results presented in Table 3 that the single axles induce more flexure on slabs, resulting in increased stresses when the axle is leaving the transversal joint (position1), i.e., when it is placed closer to the mid longitudinal joint. The effects of tridem axles are similar for any position due to the fact that the first axle of the set is close to the transversal joint but the second and third act in a remote position from the transversal edge, and by doing so, the critical stress is located far from the edge and close to the second axle. Looking at the entire set of results, single axles seemed to be more critical than tridem ones when placed far from the transversal joint.

Central slab positions were eventually tested for stresses prediction but showed to be much less critical than the longitudinal edge position.

Actually, the longitudinal stresses due to single axles appear to be much too critical for the employment of plain concrete once this material derived 4 MPa flexural strength, supposedly resulting in a stress-strength ratio over 80%, probably inducing premature fatigue consumption for the concrete (the slab is only 160 mm thick!).

Comparing to the design critical stress found using a 100 kN single axle and the Pickett & Ray chart number 6 (slab over dense liquid) during design, using proportions and linear stress-load relation, one can conclude that the numerical stress should be 2.52 MPa, slightly lower than 2.71 (design stress). Based on numerical simulations, the decision was to instrument both longitudinal and transversal bars in the meshes, especially to control bottom strains and stresses in steel bars. In Figure 7, the full steel instrumentation work done prior to slab construction is represented.

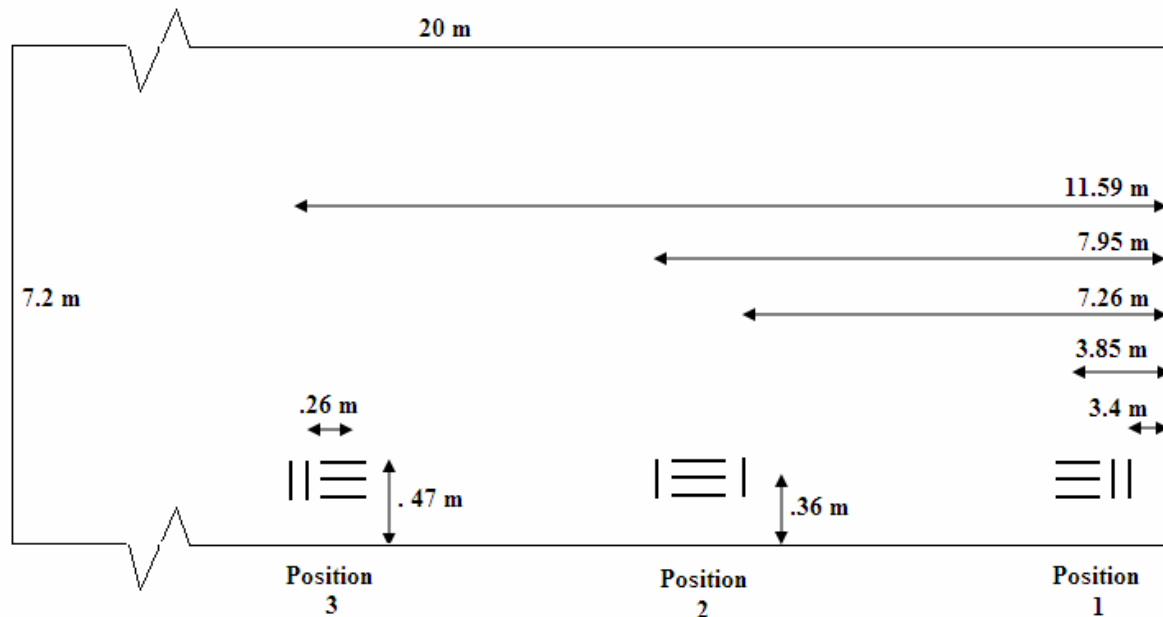


Figure 7. Location of contact *strain gages* attached to steel bars (no scale).

Strain Measurements Results

Load tests were carried out six weeks after the complete curing of concrete. In order to induce strains in steel bars, a truck provided with a tandem dual axle loaded by 173.30 kN was employed. The front axle was loaded with 32.4 kN. During the tests, several trials were performed passing the external rear wheel as close to the longitudinal edge as possible, as represented in Figure 8, for the strain gages excitation. This lateral distance from the edge was checked by means of a VHS camera for each passing. The electronic acquisition system recorded any strain gage signal provided during the tests; the data acquisition system was installed within a cabin close to the pavement shoulder. Temperatures along slab thickness were also achieved during the tests. Figure 9 shows a typical response of strain gages during the tests. Tests were conducted in the afternoon to ensure thermal differentials in slabs close to zero as temperature measurements were in course during tests.

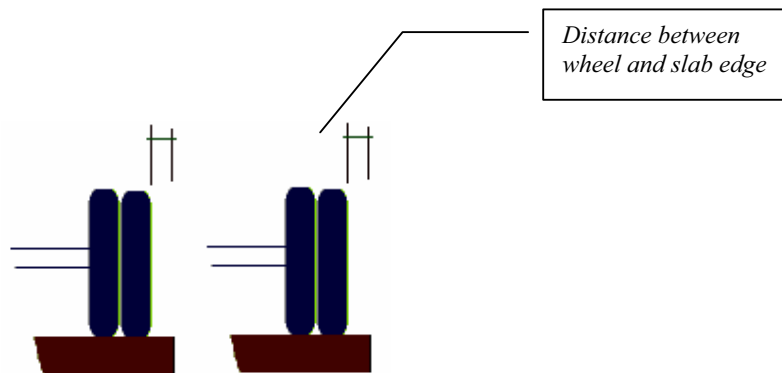


Figure 8. Schematic distance from wheel external face to longitudinal edge.

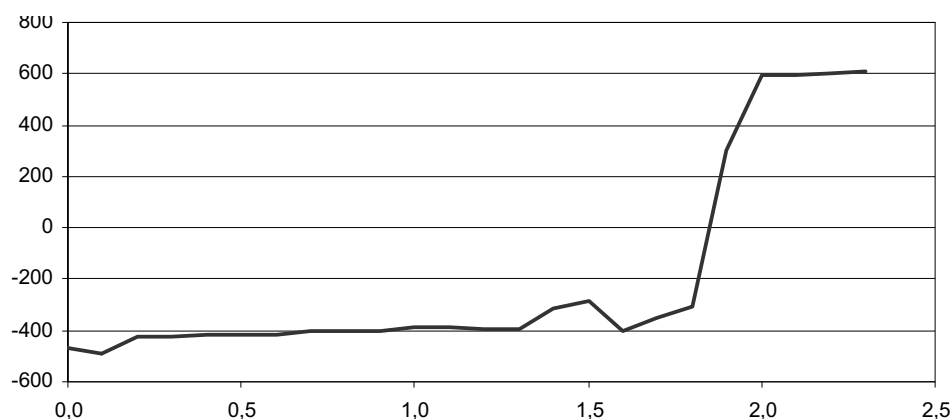


Figure 9. Load test #08, bottom strain gage, direction y , position 2 (vertical axis unit is $\mu\epsilon$).

Strain gages are sensitive to steel deformations due to load passing and the system is able to record these specific deformations (strains) along time. The difference between maximal and minimal strains achieved from the records multiplied by the steel modulus of elasticity (210,000 MPa) results in the actual stress in the steel bar. In Table 4, a general view of field measurements of strains in steel bars during the tests is presented. By comparing between strains on transverse and longitudinal directions, it seems clear that the longitudinal strains are far more important than the transversal ones; this is clear for Position 1, instrument #A2, when an average factor of 5.5 relates those strains. During the tests, some instruments failed and others did not work at all. By comparison, between records C2, C3 and A1, good coherence of measurements is shown.

Table 5 presents the average results for the measured strains during the ten truck passages excluding load test #1 due to its position during the test, quite different from the other nine tests. Within this statistics values, 500 $\mu\epsilon$ can be found as the longitudinal bottom strain resulting in average stress of 105 MPa for the steel; stresses in transversal direction resulted in 20 MPa on average. It is worth verifying that for positions 1 and 2 strain gages C2(y) and A1(y) disclosed statistically close results.

Table 6 presents the actual stress values in field, based on the strain records, for Positions 1 and 2, from which values in the range of 100 to 200 MPa are typical for the steel bars, with peak

tension of 210 MPa (corresponding to 1,000□□). The most important result from the test is related to these stress levels when focused on the actual steel strength. For the CA-60 steel pattern, the tolerable stress is 521 MPa (600 MPa divided by 1.15).

Table 4. Field strains (□□) computed from strain gage measurements during the tests.

Load Test #	Distance from edge (m)	Strains in direction x (□□)		Strains in direction y (□□)		
POSITION 2		C1	C2	C1	C2	C3
1	0.65	failed	6	n.r.	40	failed
2	0.28	failed	4	n.r.	400	340
3	0.24	failed	6	n.r.	480	380
4	0.50	4	5	n.r.	900	380
5	0.18	4	15	n.r.	430	380
6	0.16	5	8	n.r.	480	400
7	0.19	3	5	n.r.	480	410
8	0.18	8	failed	n.r.	870	780
9	0.13	2	2	n.r.	420	400
10	0.17	4	failed	n.r.	failed	failed
POSITION 1		A1	A2	A1	A2	A3
1	0.65	6	failed	75	55	20
2	0.28	7	35	420	150	100
3	0.24	4	100	550	120	125
4	0.50	6	60	560	95	85
5	0.18	5	75	220	150	115
6	0.16	8	110	600	140	135
7	0.19	10	200	500	140	120
8	0.18	6	70	1000	270	250
9	0.13	failed	failed	420	115	110
10	0.17	5	60	100	failed	failed
POSITION 3		D1	D2	D1	D2	D3
1	0.65	6	7	20	20	n.r.
2	0.28	13	12	120	80	n.r.
3	0.24	12	12	115	60	n.r.
4	0.50	20	6	90	250	n.r.
5	0.18	25	4	110	120	n.r.
6	0.16	9	4	110	90	n.r.
7	0.19	7	5	140	120	n.r.
8	0.18	12	8	230	200	n.r.
9	0.13	7	7	130	90	n.r.
10	0.17	failed	failed	failed	60	n.r.

* n.r. = not responding

Table 5. Statistical results for strains (□□).

Instrument Position □	2		1				3	
Basic statistics □	C2(y)	C3(y)	A2(x)	A1(y)	A2(y)	A3(y)	D1(y)	D2(y)
Average strain (□□)	500	434	93	483	137	118	118	109
Standard deviation (□□)	258	142	53	259	58	60	54	69
Average + sv	758	575	146	741	196	178	173	178

Table 6. Critical strains (□□) and stresses (MPa) during the load (10) tests.

Measured strains (□□)			Corresponding stresses (MPa)		
Position 2		Position 1	Position 2		Position 1
C2	C3	A1	C3	C2	C2
40	-	75	8	-	16
400	340	420	84	71	88
480	380	550	101	80	116
900	380	560	189	80	118
430	380	220	90	80	46
480	400	600	101	84	126
480	410	500	101	86	105
870	780	1000	183	164	210
420	400	420	88	84	88
45	18	100	9	4	21

Since designers usually require, as a safety procedure, the employment of a double welded steel mesh close to longitudinal edges of slabs, such a requirement could be clarified through this experiment. If one considers the linearity between load and tension in steel, the field results could be extrapolated for different types of loads. For instance, taking the small thickness of the slab, effects of a dual tandem axle loaded with 170 kN (as used during the tests) should be similar to the effects caused by a single 80 kN axle. As theoretically verified, tridem axles effects should be less important than the others. Following this rule, if single or tandem axles are loaded in excess, let us say, overloaded by 50%, the above mentioned peak stress of 210 MPa on steel should be 315 MPa, *i.e.*, circa 60% of steel strength limit. Overestimating the load to 100% of excess, the stress should be circa 80% of steel strength limit.

Taking into account the generalized employment on past projects (see Table 1) of double steel meshes for slabs edges, as mentioned, one can consider that this procedure was advocated by designers on the basis of a lack of a perfect knowledge about stresses on actual slab. Sometimes this decision, taken due to design stress, has been stated by calculating stresses due to loads in a remote position from the edge (Westergaard equations) and then multiplying such effects by a factor such as 2 for estimating edge stresses, which could be distant from the actual slab response. The results from the experiment brought new insights to those issues.

As the instrumented slab employed double steel meshes for the longitudinal edges, the

instruments measured strains for one mesh layer. Supposing that the measured strain is equal, therefore, for both meshes, the 210 MPa of stress is occurring in each one of them. So, 420 MPa tensile stress is the total amount to be combated by steel bars. If no overloaded axles are allowed to pass, no reasons for double meshes (two times in steel cross section) to bear stresses once a single mesh can bear the stress alone.

On the other hand, the load excesses can be anticipated according to former truck load statistics and then, by employing a numerical tool, design stresses can be accurately defined. On the case of aircraft, industrial and harbor equipment, the design can be conducted just on the basis of ultimate loads. The issue of using edge double steel mesh is also heavily questioned in terms of construction facility since the big density of steel close to the slab edge can be a conflicting factor for laying and compacting fresh concrete, and one more robust mesh should be more suitable for construction issues. This does not represent a problem since welded steel meshes are manufactured in several panel shapes, and can vary along the road width.

Evaluation of Stresses

In order to verify compatibility of measured and computed stresses by the Finite Element Method, the ILSAB 2000 computer program was employed for simulating field conditions for the pavement section. All the parameters employed for simulation of stresses were got from laboratorial tests and backcalculation conducted in slabs near the experimental section. Data recovered from tests in position 2 (direction y) was chosen for such verification. The modulus of elasticity for concrete plate is 28,000 MPa and for RCC base is 15,000 MPa; modulus of subgrade reaction (k-value) is 45 MPa/m. Load transfer efficiency across joints was 90% in average.

Figure 10 presents the contour for stresses in y direction for the tandem axle placed exactly in position 2 when the axle distance to the edge is 130 mm; one can note that, for such a position, comparing to Figure 7, the tire will be placed exactly over strain gage C2, reaching a stress of 1.88 MPa at the point; this value of stress tends to decrease as the tire gets far from the edge. By use of eq. (11), the critical moment within the section should be 8.02 kN.m/m. Successively, given the data for the problem, eq.(10) allows defining the steel section per unit of length within the reinforced concrete section, which is 1.32 cm²/m, required for the design under the considered axle load for the tests. The result shows the existing steel section was enough to bear load such as 173.30 kN tandem dual axle used for the test. Hence, design criteria based on ultimate limits for concrete and steel strength showed to be reasonable for the dimensioning of such a reinforced pavement. However, it must be stressed that, for the edge areas, the design has specified double welded steel mesh, resulting in a steel section twice 1.59 cm²/m.

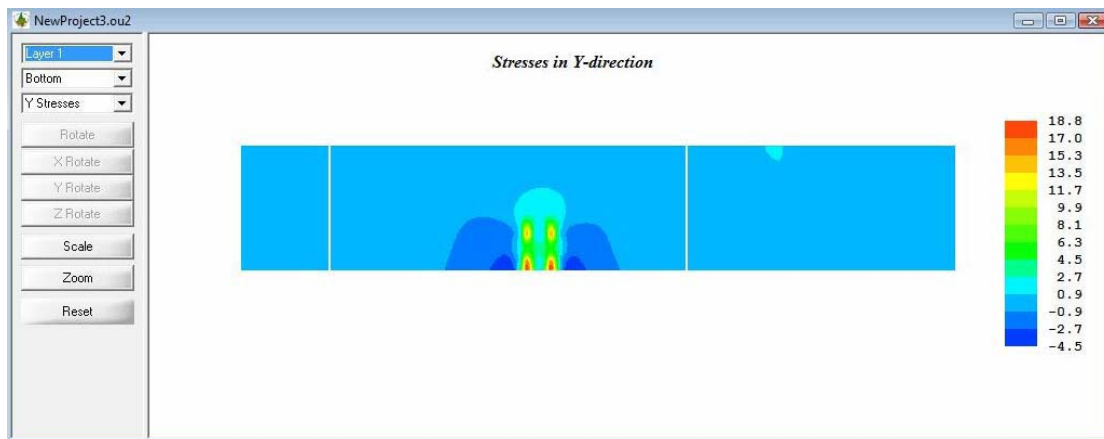


Figure 10. ISLAB simulation for load position 2 test # 9

Observing Table 7, it can be noted that the stress/strength ratio in steel bars is low and generally under 0.4, which does not lead the steel to outflow, ensuring long durability even in terms of fatigue. It must be remarked that field stresses in Table 6 are values for one steel bar while, at the edge region of the pavement, as aforementioned, a double steel welded mesh was used. Under the point of view of structural safety, it must be considered that, in the case of a single mesh, the stress level could reach 80% of the strength limit. Nevertheless, tests in field were carried out using a 170 KN tandem axle; therefore, within the legal load limit. The stresses values would overcome the strength for the steel using overloaded axles, which should be considered for the design. Hence, a design criterion for the most critical load forecasted for the road must be taken into account.

Table 7. Critical stresses (MPa) and stress/strength ratios during the tests.

Stresses (MPa)			Stress/strength ratio		
Position 2		Position 1	Position 2		Position 1
C2	C3	A1	C3	C2	C2
40	-	75	0.02	-	0.03
400	340	420	0.16	0.14	0.17
480	380	550	0.19	0.15	0.22
900	380	560	0.36	0.15	0.23
430	380	220	0.17	0.15	0.09
480	400	600	0.19	0.16	0.24
480	410	500	0.19	0.17	0.20
870	780	1000	0.35	0.31	0.40
420	400	420	0.17	0.16	0.17
45	18	100	0.02	0.01	0.04

Concluding Remarks

Reinforced concrete pavements are able to provide good characteristics for carrying the traffic: the concrete is responsible for compressive strength and superficial abrasion strength as well as for combating vertical shear stress; steel bars shall resist the flexural (in tension) moments in slabs. The combination seems interesting for taking advantage of each of the main characteristics of both materials. Nevertheless, the test itself is not able to provide a comparison between reinforced concrete and other solutions such as plain jointed concrete slabs and continuously reinforced concrete slabs. This goal could only be achieved by long-term studies. Hence, the aim of this study concerned the analysis of the design for the experiment.

The performed studies were able to provide some new insights on the design of this type of pavement with special regard to stress consideration due to load position and specifying double steel meshes as a safety procedure during design. Field results showed that the full stress value is close to the steel tensile resistance, which induces the requirement of specific design for the longitudinal joint load position. This can be accomplished by using numerical methods tools for analysis. Although the use of similar steel meshes is a practical procedure in field, different requirements of steel cross section areas could be provided by using different meshes at edge regions of reinforced slabs, avoiding the need for tying the meshes as well as any difficulty for laying concrete due to two layers of meshes.

Values for stresses in steel bars showed to be lower than the steel ultimate strength, which requires specific designs aiming at saving materials and less expensive solutions, for instance, using only one mesh supplying the required steel cross section area. The measured strains and deduced stresses are compatible to the steel ultimate strength used for the design. Other possible design improvements for similar experiments and site constructions must consider the following:

- ☐ Decreasing concrete thickness;
- ☐ Decreasing the steel area to acceptable limits;
- ☐ Design of steel area for both central and edge regions of the slabs, ensuring economy to construction practices;

Nowadays, field investigation on distresses in those reinforced concrete pavements in several locations will allow to more rigorously define the main cares to be taken during construction as well as the performance observed for such a pavement solution, allowing comparisons to traditional paving alternatives. Fatigue investigation and consideration for the design of concrete pavements reinforced with steel meshes design is another matter to be further acquired in the near future. Under the feasibility point of view, the concrete was easily laid over distributed steel meshes using a conventional slipform paver with little adjustment in concrete slump.

Acknowledgements

The authors are grateful to GERDAU Steel Company for providing the full support for field experiments. Opinions herein expressed do not mandatorily represent the opinion of welded steel manufacturers for pavement issues.

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