

Shear strength of reinforced concrete one-way ribbed slabs

Resistência ao cisalhamento de lajes nervuradas unidirecionais de concreto armado



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Abstract

The shear strength of one-way reinforced concrete ribbed slabs without stirrups in the ribs is still controversial when its estimate does not consider the contribution of the monolithic flange. To contribute to a better understanding of the behavior of these slabs were fabricated in the laboratory 8 one-way reinforced concrete ribbed slab panels where the main variables were the distance between the ribs and the thickness of the flange. The normative recommendations of NBR 6118, ACI 318 and EUROCODE 2 for the ultimate resistance of these slabs were evaluated. The experimental results showed an increase of shear strength with increasing thickness of the flange, also resulting in greater reinforcement strains and higher deflections.

Keywords: ribbed slab, shear, design codes.

Resumo

A resistência ao cisalhamento de lajes nervuradas unidirecionais de concreto armado sem estribos nas nervuras ainda gera controvérsias quando sua estimativa não considera a contribuição da capa monolítica. Visando contribuir para o melhor entendimento do comportamento destas lajes, foram confeccionados em laboratório 8 painéis de lajes nervuradas unidirecionais de concreto armado onde as principais variáveis foram a distância entre as nervuras e a espessura da capa. Foram avaliadas as recomendações normativas da NBR 6118, ACI 318 e EUROCODE 2 para as resistências últimas destas lajes. Os resultados experimentais mostraram que houve acréscimo de resistência ao cisalhamento com o aumento da espessura da capa, resultando também em maiores deformações nas armaduras e em flechas mais elevadas.

Palavras-chave: laje nervurada, cisalhamento, normas.

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1. Introduction

The search for projects and most economical structural solutions, sophisticated and efficient has become a concern for designers. This situation favored the emergence of structural systems with ribbed slabs cast "in situ", precast and/or prestressed, among others, who brought several advantages over solid slabs as architectural freedom (as they allow the use of large spans) and simplifying the implementation of the work in terms of formwork and supporting, when applied to systems without beams (flat ribbed slabs). According to BOCHI JR [1], the use of ribbed slabs, due to the minimization of cost and time, supplied the growing need for rationalization in construction.

With respect to the bearing capacity of ribbed slabs, the Brazilian standard does not provide recommendations for consideration of the concrete table in the shear strength of these slabs. Thus, this paper presents experimental results for assessing the involvement of the concrete table in shear aiming to contribute to a better understanding of the structural behavior of the one-way ribbed slabs in reinforced concrete system, and the main variables were the thickness and spacing of the layers axes of the ribs, with no shear reinforcement in them. The study focused on the analysis of the results for deformations, deflections and failure loads, these resistances were compared with national and international normative estimates.

2. Available information

The Brazilian standard for reinforced concrete structures, NBR 6118 [2] defines ribbed slabs as reinforced concrete plates with a table supported by ribs, linking them and matching their displacements. The ribbed slabs are designed as a set of T beams slightly

spaced from each other, as shown in Figure 1, and it is this spacing between the ribs which governs the shear design of the slab.

2.1 Structural behavior

The structural engineering seeks to improve the quality of procedures for concrete projects with respect to shear force. Unlike rupture by bending, shear ruptures in reinforced concrete structures are sudden, with little or no notice. They also tend to be less predictable than the flexure failure due to the considerably more complex mechanisms of rupture. The one-way ribbed slabs are characteristically a configuration of T beam with attached tables, the understanding of shear stress and shear forces that arise due to external forces is necessary. According to RÜSCH [3], the shear cracks may be originated from bending cracks. In such cases, these bending cracks so that arise, leave to a substantial redistribution of internal stresses with consequences difficult to calculate and influencing the of shear cracks inclination.

2.2 Reinforced concrete without stirrups

According to LATTE & ROMBACH [4], the determination of the shear strength capacity in reinforced concrete structures without transverse reinforcement is a classic structural problem studied for over 100 years, and this difficulty is the quantification of the strength that the compressed concrete can offer against external forces. Despite this work, most research focuses on simply supported beams or slabs with loads applied over strips with width of less than $4b$, and d is the effective depth of the slab. Experimental studies show that the shear bearing capacity of a reinforced concrete beam can be divided into two parts: one resisted by concrete and its auxiliary mechanisms, which will be addressed in this item, and the other resisted by shear reinforcement. That is, a beam, even without

Figure 1 – One-way ribbed slab



stirrups, has the ability to resist to a certain shear force. MACGREGOR [5] shows the contribution of various components of the mechanisms of shear strength of beams without shear reinforcement, except the part on arch effect, as shown in Figure 2. When the structure is loaded, the shear is transferred in part through ABC line by the mechanisms of resistance of concrete: not cracked concrete (V_{cy}), friction of aggregates (V_a) and the longitudinal reinforcement dowelling (V_d). Immediately after the bending cracks inclination, between 40% and 60% of the total shear strength is resisted by the combined strengths of V_d and V_{cy} . Considering the DEF part of the cracked structure and assuming the moments with respect to point E, it follows that V_d and V_a cause moment at point E, which should be balanced by the compressive force $C'1$. The balance of the resulting horizontal section ABDE shows that $T1=C1+C'1$ and, finally, $T1=C1+C'1$ must balance external moment in this section.

With the progress of the diagonal crack, V_a parcel decreases, increasing the fraction resisted by V_{cy} and V_d . The dowelling V_d leads to separation of the concrete along the reinforcement. When the cracking occurs, V_d falls, approaching zero. When V_a and V_d disappear, so does with V_{cy} and $C1$ the entire shear and compression is transmitted to the depth AB above the crack. At this time the life of the structure, the AB section being much shallower, will not support the compressive forces needed for balance. As a result this section crushes up, not bringing any resistance to the loads. COLLINS et. al. [6] present an analysis review of 60 years of research on the behavior of short reinforced concrete beams without stirrups, which concluded that the largest parcel of shear strength is not in the characteristic strength of the concrete, but is affected by the dimensions the structural element, by engagement of the aggregates of the longitudinal reinforcement dowelling. According to ACI-ASCE 426 Committee [7] for reinforced concrete struc-

tures subjected to shear, the resistance due to concrete is the sum of several mechanisms capable of transmitting forces between sections as:

- **Aggregate interlock:** This mechanism occurs between the two surfaces caused by a crack. The contribution of the aggregates engagement for the shear strength depends on the crack opening and the roughness of the surfaces;
- **'Arching action':** this mechanism occurs more significantly in beams with reduced spans due to the accommodation of the compressive stresses in the arch and intensity of these stresses depend mainly on the slope of the arc, being directly linked to the ratio a/d , that is the lower the value of the shear span (short beam) is, the greater the effect of arc (LEONHARDT & MÖNNIG [8]);
- **Cantilever action:** this mechanism occurs in non-cracked sections of the beam (between two consecutive cracks) or in no-cracked parts of cracked elements (compression zone of a cracked section);
- **Dowel action-effect:** a longitudinal reinforcement resists a parcel of the displacement caused by shear force due to the dowel effect on the rebar. The dowel power in the longitudinal reinforcement bar depends on the stiffness of the bar at the intersection with the crack.

2.3 Important factors for shear strength

Analytical and experimental studies revealed that the shear strength capacity in concrete structures is controlled by the following parameters: (1) characteristic compressive strength of concrete (f_{ck}), (2) size effect, (3) longitudinal reinforcement ratio and (4) axial force.

2.3.1 Characteristic compressive strength of concrete

Typically, concrete structures with high compressive resistance have high load capacity, i.e. higher shear strength. As the strength of concrete is represented by the combination of tensile and compressive strengths, then the use of each parcel of resistance will result in the rupture mechanism of the structure. As it is assumed that the concrete cracking is caused by the principal tensile stresses, so the tensile strength of the concrete will have decisive influence on the shear load capacity of concrete structures.

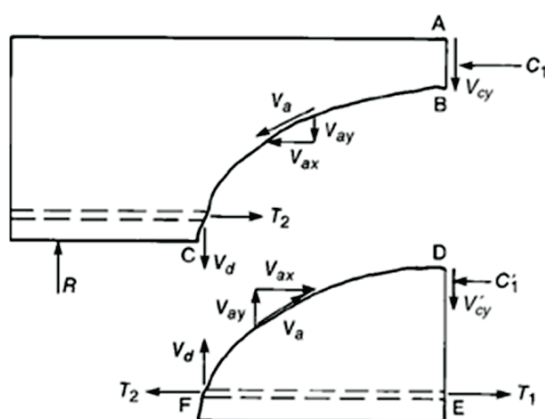
2.3.2 Size effect

KANI [9], in his studies with beams without stirrups, noted that the shear strength of these structures decreases as the height of the beam increases. For such a phenomenon, it is understood that the width of the cracks is proportional to the depth of the beam, i.e. the wider the crack is, lower will be the transferability of friction between the aggregate, since the width of the cracks decreases the ability to transfer shear between aggregated. COLLINS & KUCHAMA [10] present studies on reinforced concrete structures missing transverse steel, concluding that higher or deeper structures failure with lower loads. This conclusion is clear in structures made of high-strength concrete, because they are more sensitive to this effect.

2.3.3 Flexural reinforcement geometrical rate

KANI [11] studied 133 rectangular beams without stirrups, concluded that the influence of the longitudinal reinforcement ratio on shear strength is considerable. It has been demonstrated that the percentage of longitudinal reinforcement ratio regulates the height

Figure 2 – Forces along an inclined shear crack (MACGREGOR, 1997)



- V_{cy} - Non cracked concrete
- V_a - Aggregates interlock
- V_d - Longitudinal reinforcement dowelling
- C - Compressive balance force
- T - Tensile balance force

of the resulting compressive and tensile stress of the concrete. The increased rate of longitudinal reinforcement increases the height of the compression zone and decreases the width of the cracks and as a result, there is an increase in the shear strength of the structure. This has been widely accepted by the scientific community and that is why the rate of longitudinal reinforcement appears in the formulas of shear strength of most normative codes.

2.3.4 Axial force

It is widely accepted that the axial tensile force reduces the shear strength of concrete structures and the axial compression force, due to the application of normal loads or prestress, increases the shear strength of concrete elements. Since the axial tensile force reduces the height of the concrete compression zone, as well as increases the width of the cracks, so there is a reduction of shear in the compression zone and transfer of cracking at interfaces.

3. Codes' prescriptions

3.1 NBR 6118 [2]

The Brazilian standard brings different recommendations for unidirectional and bidirectional ribbed slabs. One-way ribbed slabs shall be calculated according to the direction of the ribs, being neglected transverse stiffness and torsional rigidity. NBR 6118 [2] provides that the ribbed slabs with spacing (l₀) between ribs axes less than or equal to 650 mm can do without shear reinforcement to resist tensile stresses caused by shear force when the design load value does not exceed the value obtained with Equation 1. However, if the spacing between the ribs is greater than 900 mm, the design shear strength of the ribs without transverse reinforcement can be obtained from Equation 2.

$$V_{Rd} = [\tau_{Rd} \cdot k \cdot (1,2 + 40 \cdot \rho_1)] \cdot b_w \cdot d \quad (1)$$

Where,

$$\tau_{Rd} = 0,25 \cdot f_{ctd} = 0,25 \cdot \frac{f_{ctk,inf}}{\gamma_c}; \rho_1 = \frac{A_{sl}}{b_w \cdot d} \leq 0,02; k = (1,6 - d) \geq 1, \text{ com } d \text{ em m.}$$

$$V_{Rd} = V_{c0} = 0,09 \cdot \sqrt{f_{ck}}^2 \cdot b_w \cdot d \quad (2)$$

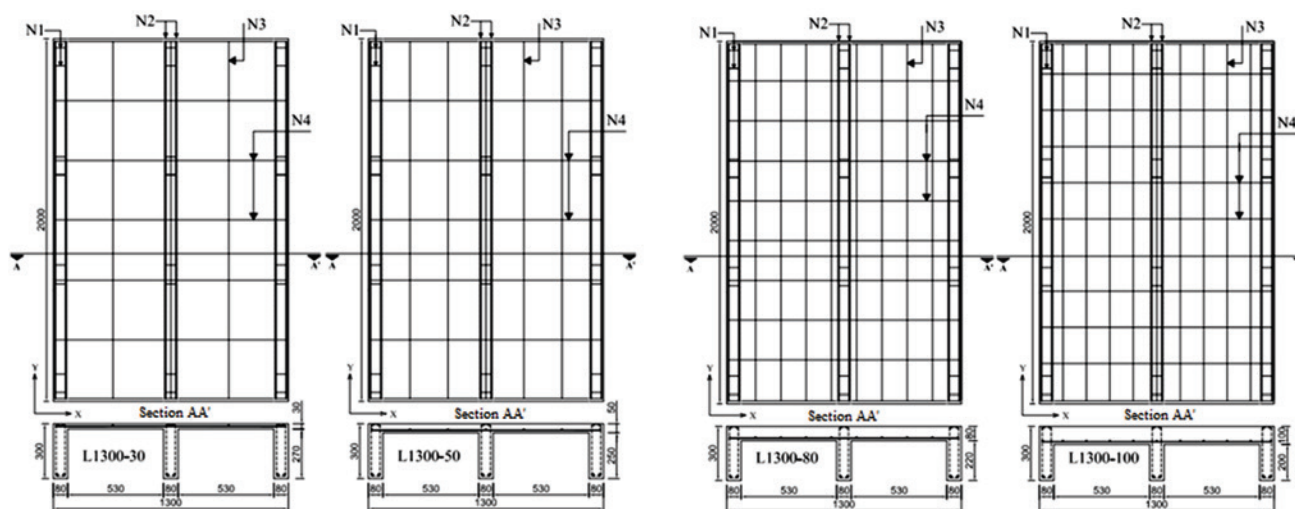
With,

τ_{Rd} = design shear stress;
 f_{ctd} = concrete design tensile strength;
 $f_{ctk,inf}$ = lower characteristic tensile strength of concrete;
 A_{sl} = longitudinal tensile reinforcement area;
 b_w = cross section minimum width along useful depth d.

3.2 ACI 318 [12]

According to ACI 318 [12], the shear strength of reinforced concrete elements is based on the average shear stress of the effective cross-sectional surface $b_w \cdot d$ determined by the resistant parcel of concrete V_c and shear reinforcement V_s , as shown in Equation 3. But for the calculation of elements without shear reinforcement as $V_n = V_c$, the shear strength can be expressed by Equation 4.

Figure 3 – Reinforcements of the L1300 group of slabs



2 N1=5.0c100; 2 N2=12.5-1980; 5, 7, 10 and 11 N3=5.0-1980; 7, 7, 10 e 11 N4=5.0-Var.

$$V_u = V_n = V_c + V_s \quad (3)$$

Where,
 $f_c < 70$ MPa is the compressive strength of concrete;
 M_u is the ultimate bending moment in N.mm.

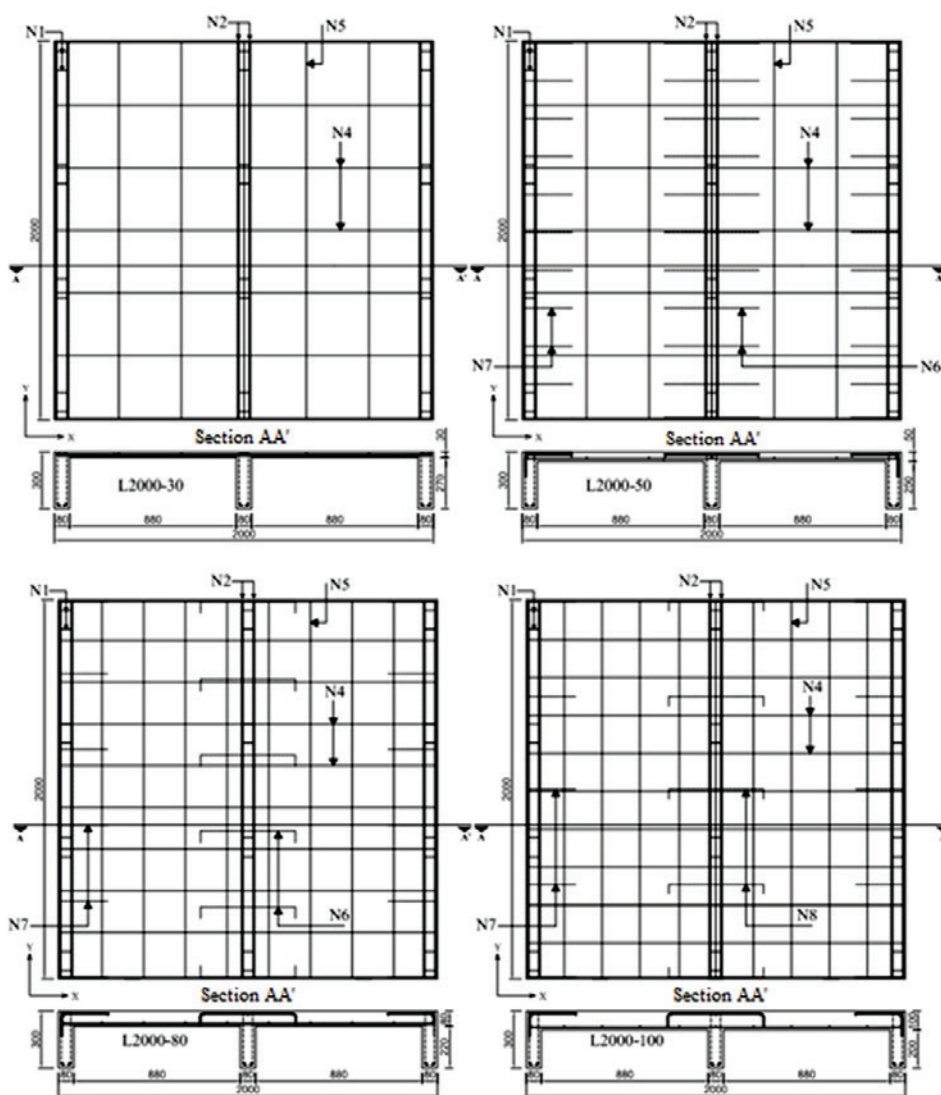
3.3 EUROCODE 2 [13]

The European standard recommends that waffle slabs need not be

dimensioned from the discretization of its elements, table and rib, since the system has torsional stiffness. For the sections that do not require transverse reinforcement the shear resistance element is given by Equation 5.

$$V_{Rd,c} = 0,12 \cdot \left(1 + \sqrt{\frac{200}{d}} \right) \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} \cdot b_w \cdot d \quad (5)$$

Figure 4 – Reinforcements of the L2000 group of slabs



7, 7, 10 e 11 N5=5.0-1980; 11 e 6 N6=5.0-500; 11, 6 e 5 N7=5.0-350; 5 N8=5.0-650

Table 1 – Slabs' main characteristics

Slab	l_0 (mm)	h_i (mm)	d (mm)	f_{ck} (MPa)	ρ (%)
L1300-30	610	30	277	35,2	1,1
L1300-50	610	50	277	35,2	1,1
L1300-80	610	80	277	35,2	1,1
L1300-100	610	100	277	35,2	1,1
L2000-30	960	30	277	35,2	1,1
L2000-50	960	50	277	35,2	1,1
L2000-80	960	80	277	35,2	1,1
L2000-100	960	100	277	35,2	1,1

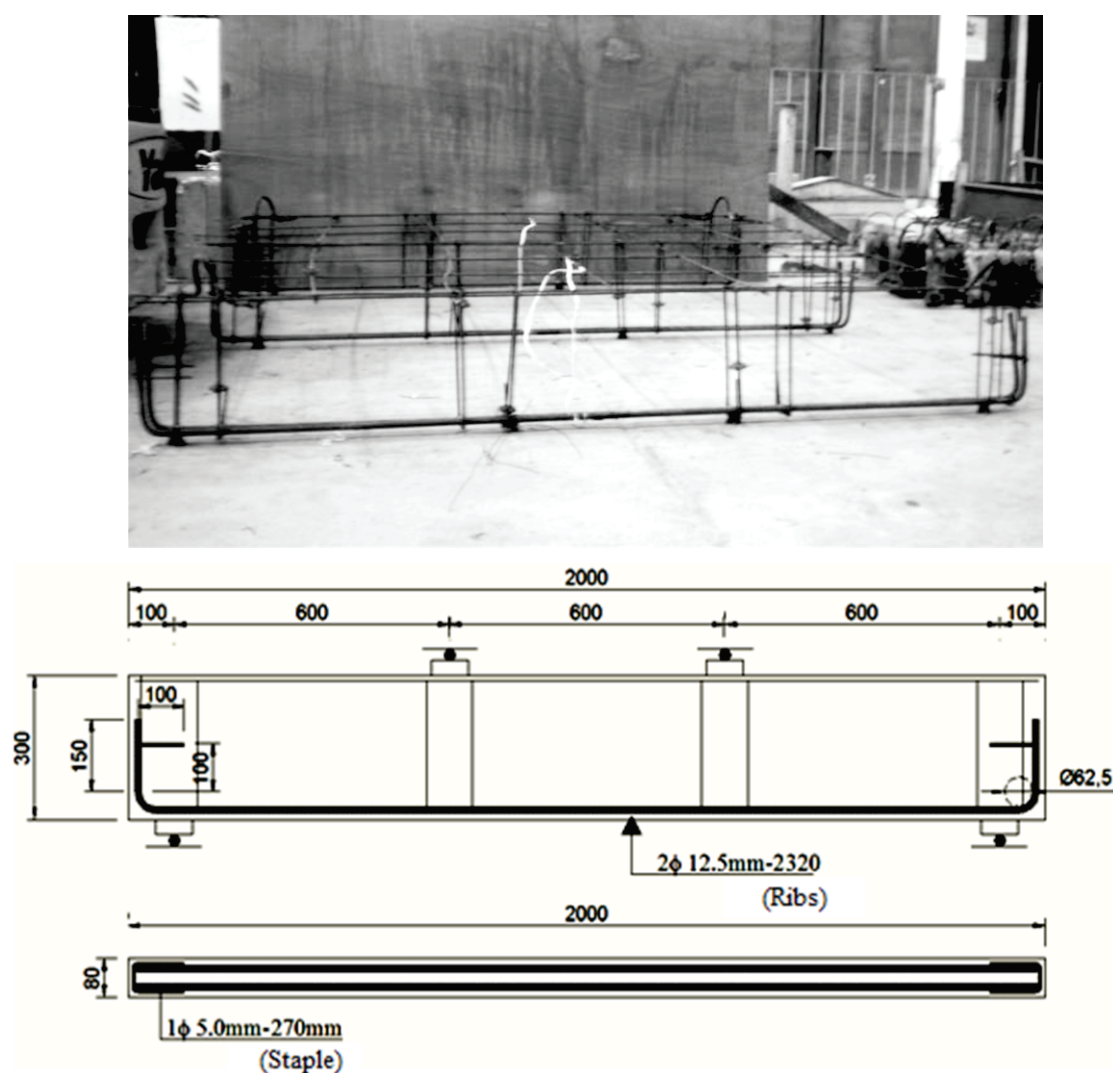
4. Experimental program

The experimental program describes a series of tests carried out at the Laboratory of Civil Engineering, Federal University of Para. The models tested in the laboratory does not have the same boundary conditions of a real structure, as this presents a continuo cooperating table which would lead to very different loads from the laboratory.

4.1 Slabs' characteristics

Eight unidirectional ribbed reinforced concrete slabs panels were tested to failure. The panels were rectangular and square where the L1300 group, consisting of four panels, has dimensions (1300 x 2000) mm² varying the height of the concrete table (h_f) of 30, 50, 80 and 100 mm, while the L2000 group, composed of four

Figure 5 – Ribs' reinforcements



slabs, has dimensions (2000 x 2000) mm² and the same variations to the height of concrete table (Table 1). The total height of the slabs was 300 mm. The slabs ribs were 80 mm wide and spaced of 610 mm in group L1300 and 960 mm in group L2000 between their axes - below and above the Brazilian normative limit of 650 mm. The concrete cover was set at 15 mm. All panels have the same flexural reinforcement distribution, consisting of 2Ø12.5 mm with experimental yield strain of $\epsilon_{ys}=2.3\%$ and

positioned along the ribs, to a useful height $d=277$ mm resulting in a geometric rate of flexural reinforcement of $r=1.1\%$ for using the collaboration of longitudinal reinforcement in its entirety, and remark the participation of the concrete table in shear strength, and with a relation $a/d=2.15$.

No shear reinforcements were used in the ribs, with the aim of highlighting and quantifying the contribution of the concrete table in resistance. Because of the lack of shear reinforcement,

Figure 6 – Slabs' monitoring (strain gauges)

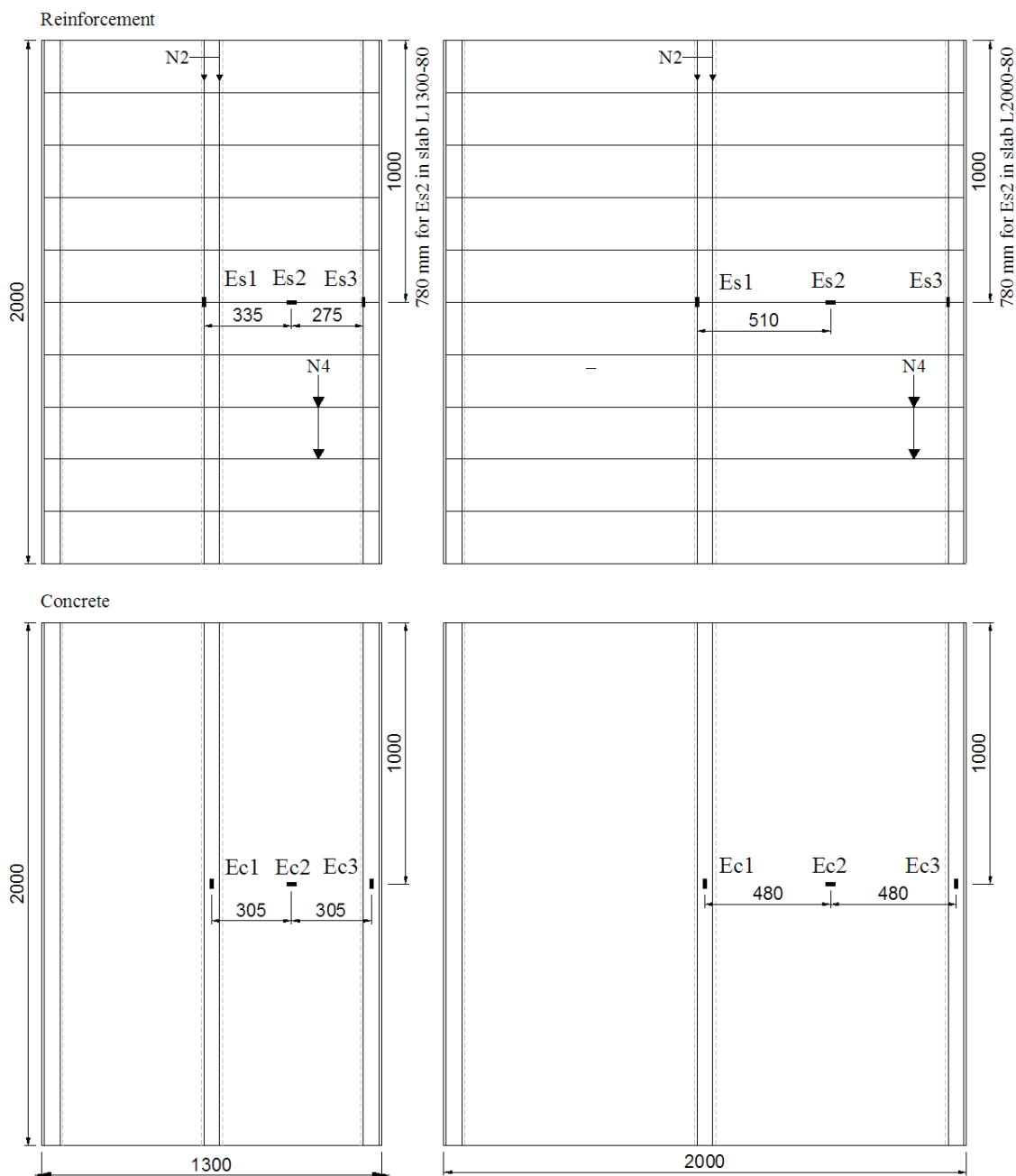
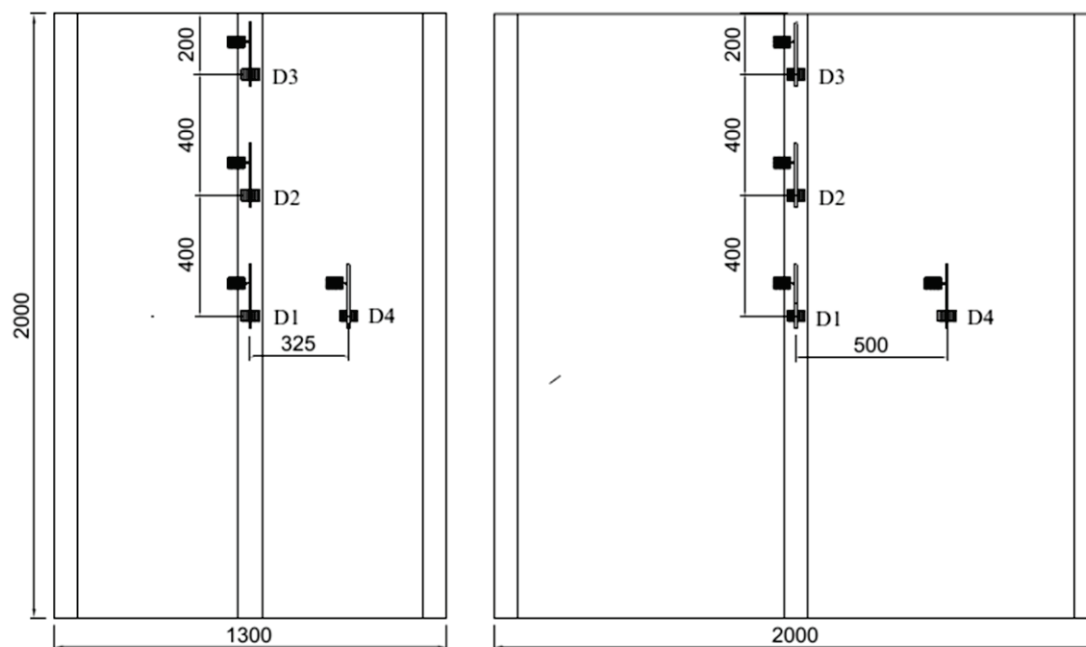


Figure 7 – Dial gauges' distribution on the slabs



there was the possibility of crushing the concrete located due to stress concentrations in regions of support and application of loads and it was decided to strength these regions using two stirrups with diameter 5.0 mm and 100 mm distant from each other,

and facilitates the assembly of the longitudinal bars shown in Figures 3 and 4. Figure 5 shows details of the reinforcement of the ribs, where it is possible to observe clearly the position of the stirrups composition and the distances between supports

Figure 8 – Ribbed slabs ready for casting

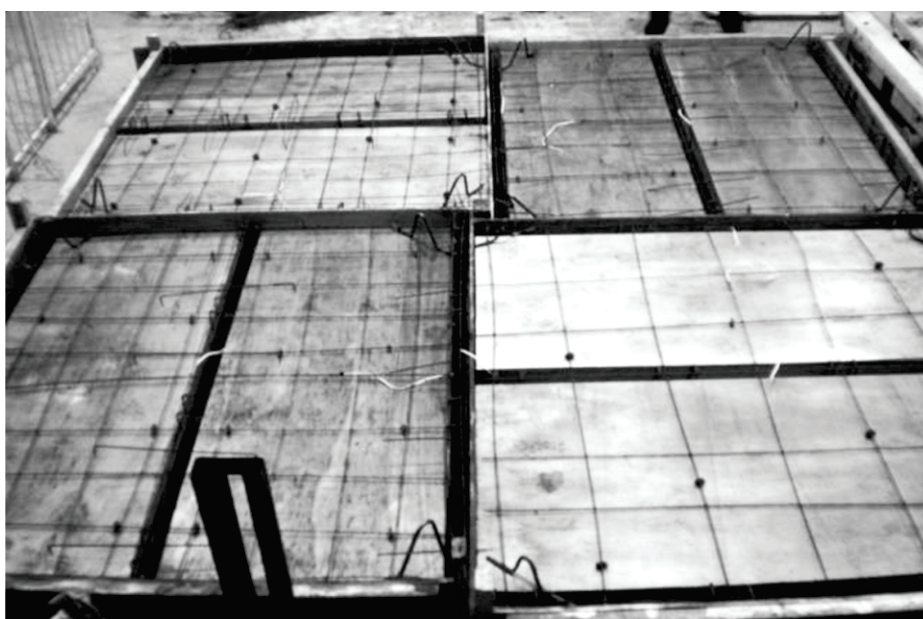


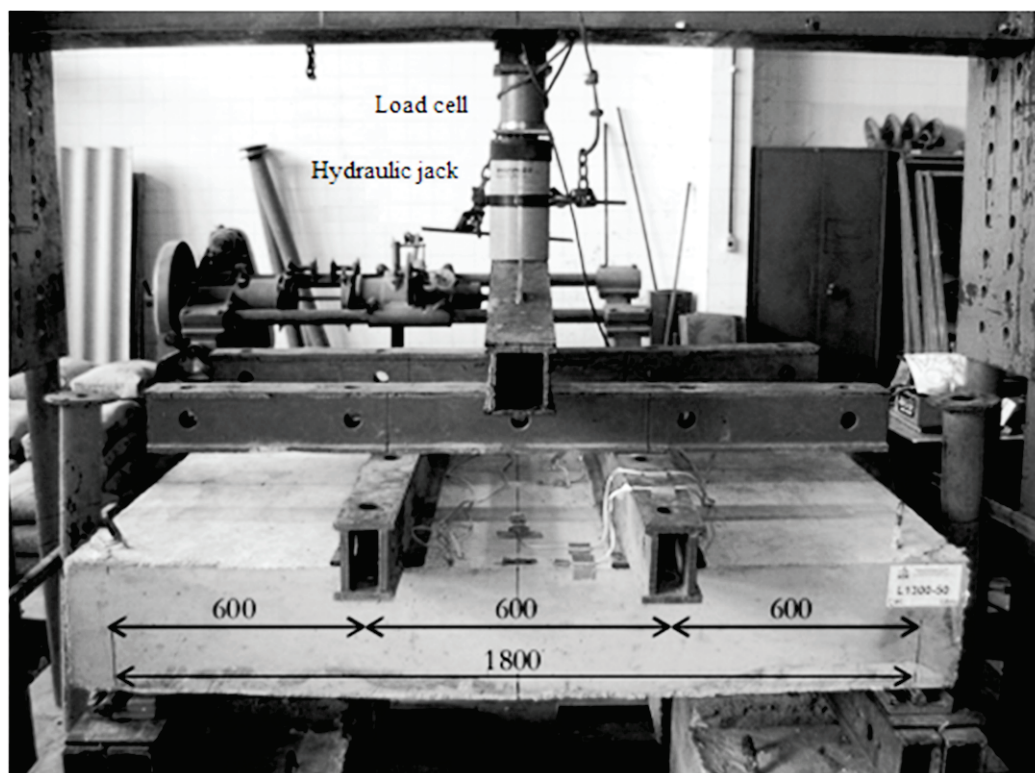
Figure 9 – Concrete placement and slabs' surface regularization



Figura 10 – Groups L1300 e L2000 of slabs with different tables' thickness



Figure 11 – Test system



and points of application of load (Stuttgart test system).

A secondary reinforcement was made and prepared in the concrete table of all slabs, representing monolithic link between ribs and table. The experimental yield strain of this reinforcement was $\epsilon_{ys}=4.6\%$. This reinforcement is designed to combat cross-bending of the concrete table due to the applied loadings, as well as represent the most realistic situation based on the superposition of the reinforcement of a slab with beam element, especially in slabs of L2000 group, who had concrete tables verified to flexure according to the Brazilian standard.

4.2 Slabs' monitoring

The reinforcements' strains were monitored using electrical strain gauges (EERs) manufactured by Excel Sensors with grid dimensions $(3.18 \times 3.18) \text{ mm}^2$ - model PA-06-125AA-120L, which were installed on the steel bars. The concrete's strains at the top surface of the slabs were also monitored with electrical strain gages (EERs) from the same manufacturer - model PA-06-201BA-120L. Strains in the ribs along the longitudinal direction were monitored at mid-span and on the table in the transverse direction. The strain gauges were positioned in regions that showed greater tensile stress for flexural and composition reinforcements, and compression of the upper face of the concrete slabs (Figure 6). Already the displacement measurements were obtained from gauges positioned on the bottom surface of the slab (Figure 7). These gauges were digital comparators watches of Digimess brand, with maximum course of

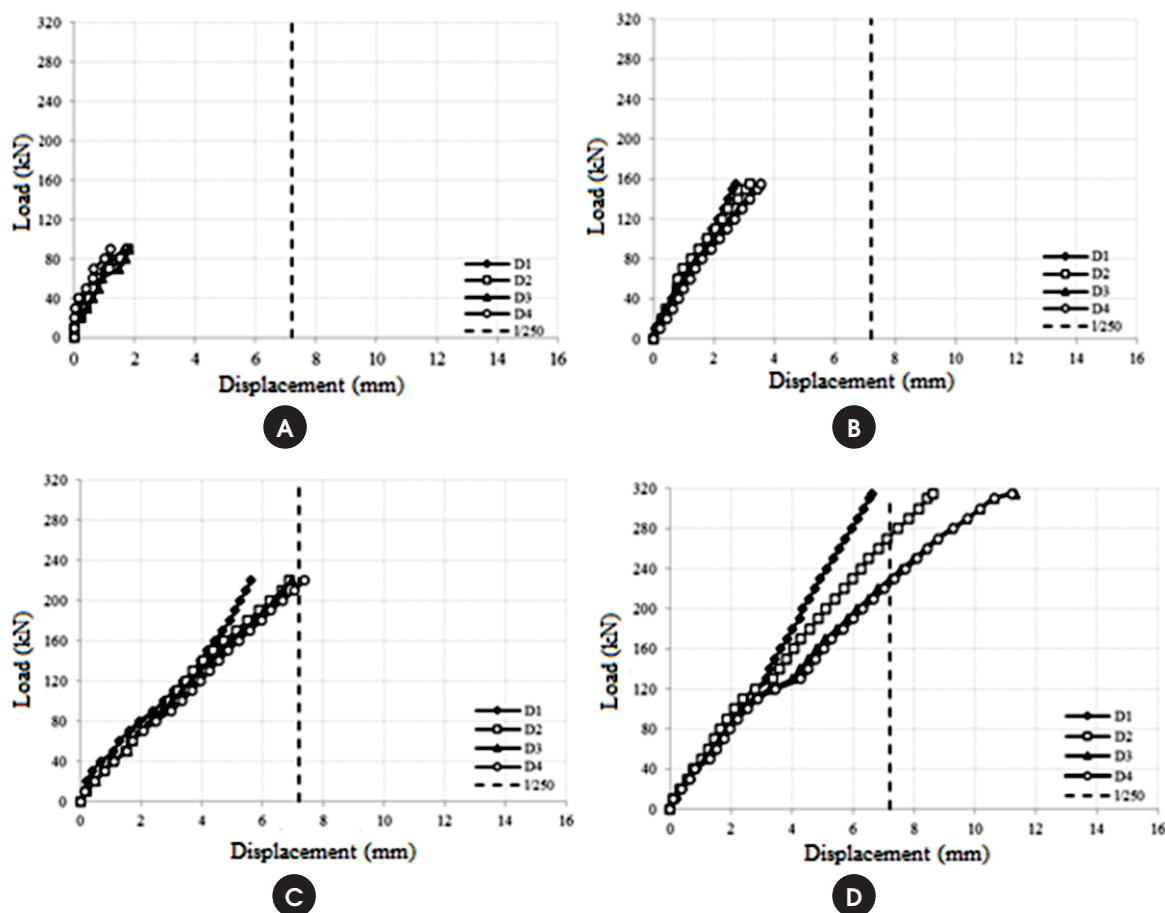
50 mm and reading accuracy of 0.01 mm, and were supported in auxiliary structures without contact with the test system.

4.3 Slabs' casting

The waffle slabs were made in two stages, the first of which were molded slabs of L1300 group and, in the second, slabs of L2000 group. The slabs molds were of compressed sawed wood pound of 10 mm thick. Then, the reinforcements were carefully positioned and mounted in the molds with the aid of plastic spacers, including negative and assembly reinforcements (stirrups of composition and staples) in order to keep the planned useful height for all slabs, as shown in Figure 8.

The monitoring of the ribs' longitudinal and distribution reinforcements was performed at points defined according to design specifications, receiving a preparation before gluing for removal of surface irregularities with rasp tool. After surface cleaning, the gages were bonded with adhesives based on epoxy and, after welding of connecting cables to terminals, were also protected with resin-based epoxy to later be surrounded by isolating tape. The casting of the slabs was carried out from commercially supplied mixed concrete whose compression strength of 30 MPa at 28 days was applied, with rolled pebble of 19 mm of maximum diameter. It took an approximate volume of 3 m^3 of concrete for the slabs and proofs, also considering possible losses. The concrete placement was completed with the regularization of the surface and removal of the excess material (Figure 9). While

Figure 12 – Slabs' vertical displacements: (a) L1300-30; (b) L1300-50; (c) L1300-80; (d) L1300-100



the slabs casting, 18 cylindrical proofs with dimensions of 100 mm diameter and 200 mm in length for the tests to obtain the mechanical properties of concrete were cast. The curing of concrete took place in a laboratory environment for 7 days, made with regular wetting. Figure 10 shows the identified and stored slabs before tests.

4.4 Test system

The test system was mounted on the reaction slab at the Civil Engineering Laboratory (LEC) of the Federal University of Para (UFPA) with the main objective to obtain the response of the structure to the distributed load applied transversely to the ribs. Thus, the slabs were tested using auxiliary devices with rollers over bearing blocks, supporting the load applied perpendicular to the longitudinal axis of the ribs, on the upper surface of the panel, through a simply supported steel beam for loading distribution in two points simulating a simple flexure situation, coupled to a steel frame on the reaction concrete slab, according to Figure 11.

The applied loadings were established by load steps every 5 kN. Prior to beginning the test, the slabs were subjected to

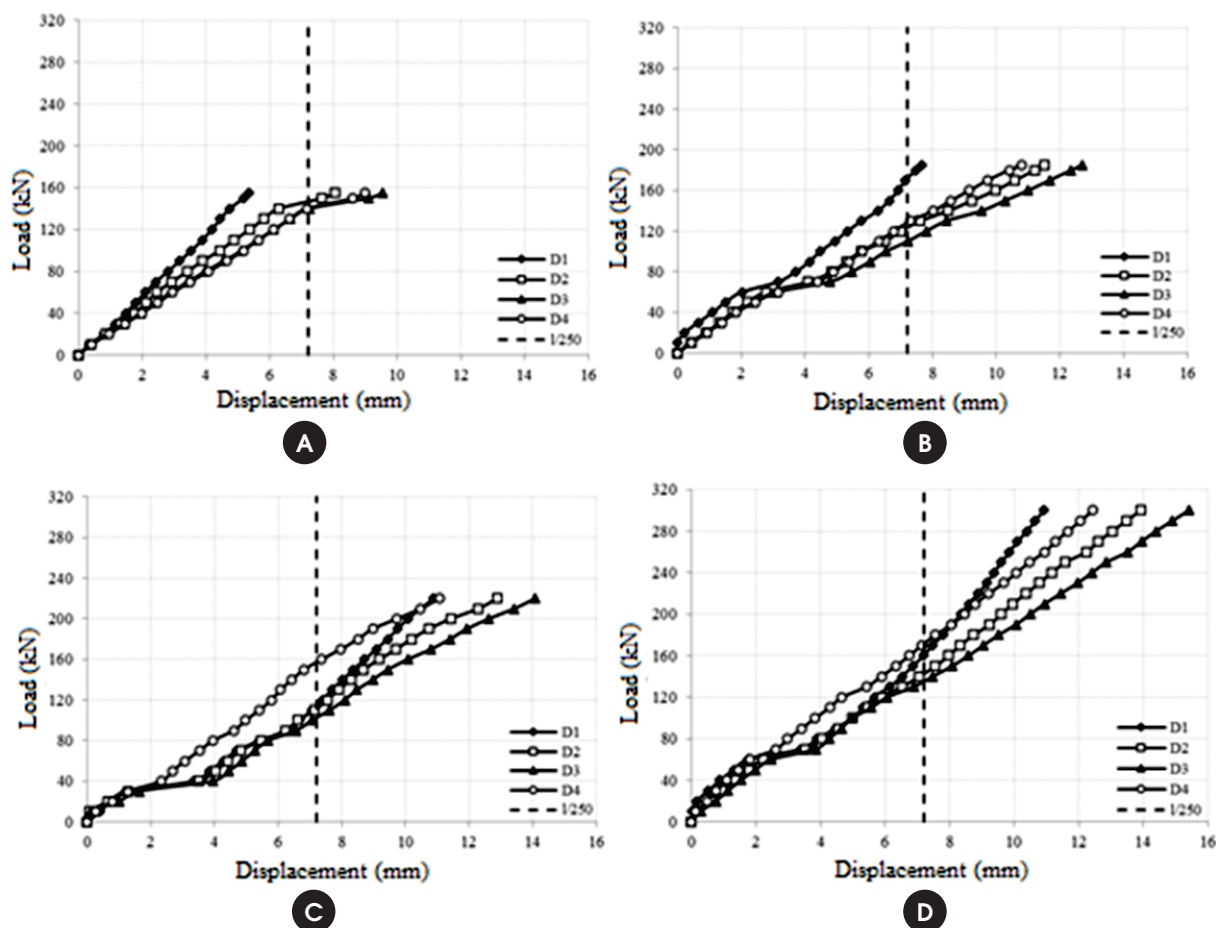
a preload of 0.5 kN in order to stabilize the system. The assembly of the test system was carried out with the aid of a forklift capacity of approximately 50 kN. The information about the strains were obtained using a modular data acquisition: ALMEMO ® 5690-2 m, from Ahlborn, compatible with the software AMRWinControl, who proceeded to read the gauges positioned in steel and concrete.

5. Results

5.1 Vertical displacements

The experimental vertical displacements (d) are shown in Figures 12 and 13, and Table 2 shows the maximum values of these displacements. Slabs of L1300 group showed similar curves, been differing only in the slab L1300-100, where there is a clear separation between these curves. However, slabs of L2000 group have kept constant this distance, even in slabs with thinner tables. Considering the L1300-100 slab, this was due to their greater stiffness, resulting in higher resistance and displacements. This same behavior was observed in slabs of L2000 group, but the greater spac-

Figure 13 – Slabs' vertical displacements: (a) L2000-30; (b) L2000-50; (c) L2000-80; (d) L2000-100



ing between the ribs resulted in greater vertical displacements. Almost all slabs exceeded the Brazilian regulatory limit for vertical displacements, where only the L1300-30 and L1300-50 slabs did not, due to their lower height not cracked concrete than others and hence lower load capacity. This behavior indicates that the table, or the beam flanges, can change the behavior of the slab and the increase of its thickness stiffens the slab so that its strength and vertical displacements increase, influencing its ductility, because there is an increase in the contribution of the concrete compressed leading to a rebalancing of the resistant forces (steel and concrete), allowing the longitudinal reinforcement strain until it reaches the yield stress of steel. Therefore, increasing the thickness of the table resulted in greater load capacity of the slabs, longitudinal reinforcement yielding and larger vertical displacements.

5.2 Flexural reinforcements' strains

The graphs of main and distribution flexural reinforcements' strains are shown in Figures 14 and 15. Table 3 shows the maximum strains of the steel of the flexural reinforcement of the test slabs and the respective sensors which registered them. The slabs showed increased strains of the reinforcement proportional to the increase

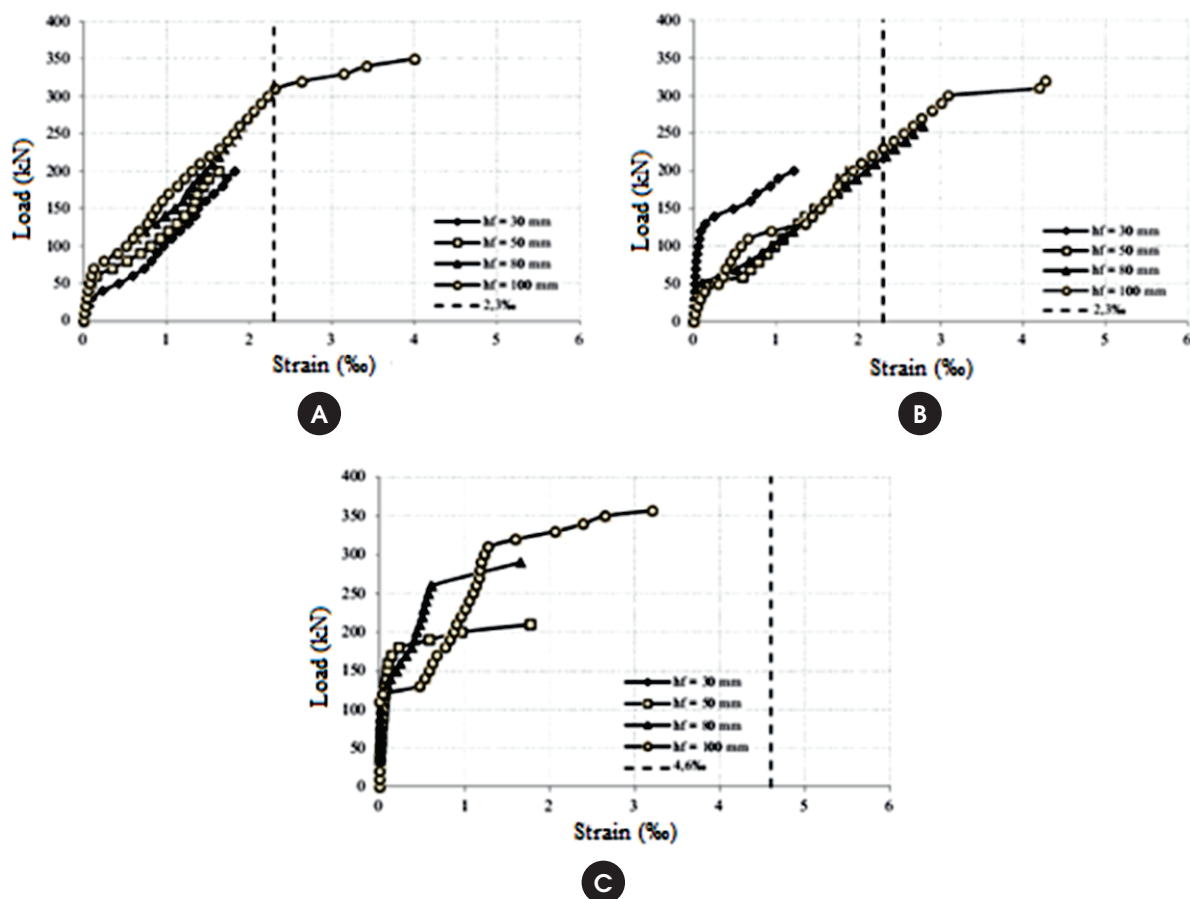
in thickness of the table, allowing the longitudinal reinforcement reached the yield strain, and the strains of the flexural reinforcement of the slabs L1300-80, L1300-100, L2000-50, L2000-80 and L2000-100 exceeded 17%, 82%, 8%, 91% and 143% the yield strain of the 12.5 mm (2.3 ‰) steel bar, respectively.

Analyzing the graphs for external and central ribs you can it can be

Table 2 – Slabs' maximum vertical displacements

Slab	P_u (kN)	δ_{max} (mm)
L1300-30	200	1,81
L1300-50	210	3,56
L1300-80	290	7,38
L1300-100	360	11,32
L2000-30	160	9,54
L2000-50	220	12,69
L2000-80	330	14,08
L2000-100	370	15,41

Figure 14 – L1300 slabs' reinforcement strains: (a) flexure of external rib; (b) flexure of midrib; (c) distribution of the table



seen that in the group of slabs L1300 ribs showed seemed curves, and the midrib showed greater strain, while the group of slabs L2000 ribs present very disparate curves due to external ribs display strains close to the yield one for the longitudinal reinforcement rebars, for the slabs with $h_f=80$ mm and $h_f=100$ mm. This occurs because the slabs L1300 presented distance between the ribs 1 axes less than the normative limit, which is how it is calculated as a slab. However, this does not occur in the slabs L2000, because of this distance greater than 650 mm, which leads to be calculated as beams. Therefore, the central rib of the L2000 slabs absorb greater load, passing through the table lower intensity of load for the external ribs. So the Brazilian standard points to the need of analyzing this concrete table against flexure.

The curves for the distribution reinforcement's strains show that the slabs of both groups have certain similarity at the beginning and approximately after 200 kN these values tend to be proportional to the increase in the thickness of the tables. This is because the applied load is close to that for yielding of the longitudinal reinforcement, leaving the loading hat was previously absorbed by flexural reinforcement (now plasticized) to be redistributed by the concrete resistance mechanisms of the ribs and distribution rein-

forcement of the table, and this is redistributive capacity of the slab which allows a general increase in resistance with thicker tables, even for ribs without the shear reinforcement.

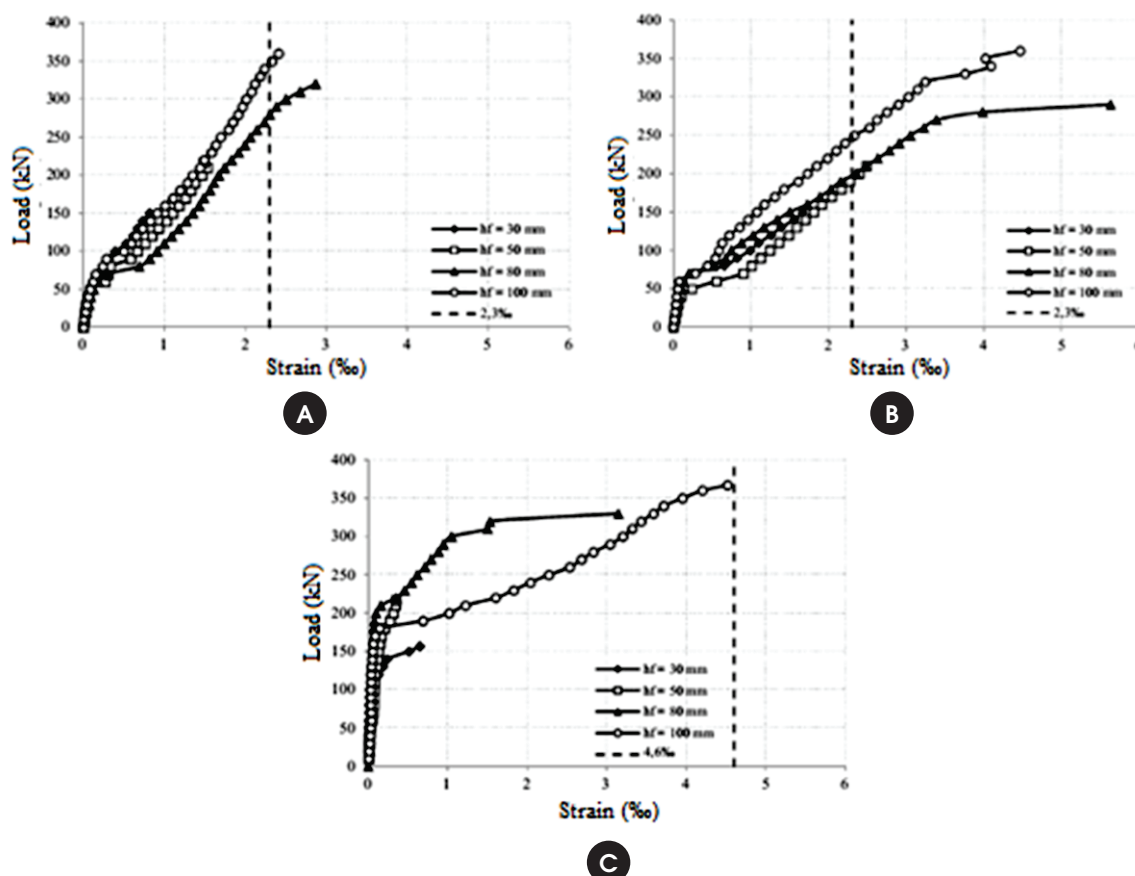
5.3 Concrete's strains

The graphs of strains in concrete are shown in Figures 16 and 17. Table 3 shows the maximum strains of the concrete of the tested slabs and the respective sensors that recorded them. Unlike what occurred with the reinforcement's strains, the slabs did not show an increase in the values of the concrete strains proportional to the increase of thickness of the concrete table, or at least it was not clear this relationship, but all strains were much lower than the limit 3.5 ‰ for bended elements. In both groups of slabs curves were very close only for the initial stages of loading, tending to distance from each other as they neared the collapse.

5.4 Cracking pattern

The cracking pattern was similar for all slabs, which took its beginning with excessive cracking of the ribs. Initially, flexural cracks arose

Figure 15 – L2000 slabs' reinforcement strains: (a) flexure of external rib; (b) flexure of midrib; (c) distribution of the table



with posterior inclination near the supports, which set the imminence of shear failure, and after certain load the shear failure of the external rib was observed in most slabs, followed by the midrib, where the failure surface crossed the table leading to its detachment. This type of failure occurred due to the central rib provide well-defined T cross section, while the outer ribs presented L cross-section, result-

ing in increased rigidity for the middle rib, remaining more integrate than the outer rib, even under high loads, while the detachment of the table confirms this resistant activity monolithically with the rib. On the upper surface of the slab there was a longitudinal crack- ing pattern between rib and table, which indicates the presence of

Table 3 – Slabs' maximum steel and concrete strains

Slab	$\varepsilon_{s,max}$ (‰)	$\varepsilon_{c,max}$ (‰)
L1300-30	1,8 (Es3)	-0,46 (Es1)
L1300-50	2,1 (Es1)	-0,48 (Es1)
L1300-80	2,7 (Es1)	-0,39 (Es1 and Es3)
L1300-100	4,2 (Es1)	-0,82 (Es2)
L2000-30	1,6 (Es1)	-0,35 (Es1)
L2000-50	2,5 (Es1)	-0,44 (Es1)
L2000-80	5,6 (Es1)	-0,67 (Es2 e Es3)
L2000-100	4,5 (Es2)	-1,02 (Es1)

Table 4 – Ultimate loads and failure modes of the slabs

Slab	P_{ys} (kN)	P_u (kN)	Experimental failure mode	Theoretical failure mode
L1300-30	–	200	CSE**	C*
L1300-50	–	210	CSE**	C*
L1300-80	220	290	CCE***	C*
L1300-100	240	360	CCE***	C*
L2000-30	–	160	CSE	C*
L2000-50	180	220	CCE	C*
L2000-80	270	330	CCE	C*
L2000-100	350	370	CCE	C*

*C (Shear); **CSE (Shear without yielding); ***CCE (Shear with yielding).

negative moment, and this cracking in some slabs reached the transverse section of the rib. This crack in the upper surface is provided by the standard for ribbed slabs with distance between its axes beyond 650 mm, but below this there is no normative value

analysis with respect to tables but the tables of L1300 slabs showed excessive cracking parallel to the ribs. The cracking patterns of the outer ribs of the slabs are shown in Figure 18. These ribs presented higher degree of cracking and it was where most failures occurred.

Figure 16 – L1300 slabs' concrete strains: (a) flexure of external rib; (b) flexure of midrib; (c) distribution of the table

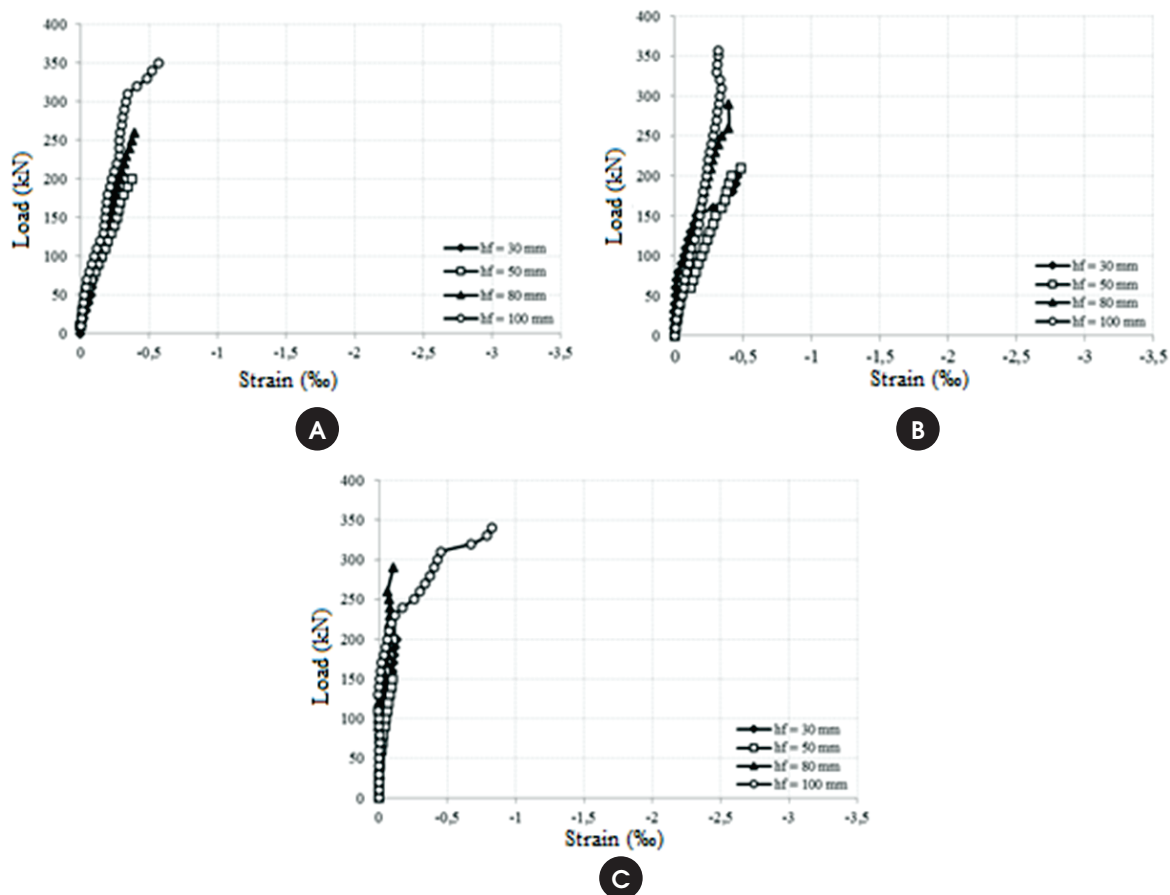


Table 5 – Comparison between theoretical and experimental results

Slab	NBR 6118	ACI 318	EC 2	Experimental	Experimental/Theoretical		
	V_{NBR} (kN)	V_{ACI} (kN)	V_{EC2} (kN)	P_u (kN)	P_u/V_{NBR}	P_u/V_{ACI}	P_u/V_{EC2}
L1300-30	240	264	151	200	0,83	0,76	1,32
L1300-50	240	264	151	210	0,88	0,80	1,39
L1300-80	240	264	151	290	1,21	1,10	1,92
L1300-100	240	264	151	360	1,50	1,36	2,38
L2000-30	260	264	151	160	0,62	0,61	1,05
L2000-50	260	264	151	220	0,85	0,83	1,45
L2000-80	260	264	151	330	1,27	1,25	2,18
L2000-100	260	264	151	370	1,42	1,40	2,45

5.5 Ultimate loads and failure modes

Table 4 shows the failure modes observed and estimated. In group L1300 is observed that 50% of the slabs presented shear failures without flexural reinforcement yielding and the other with yielding due to the increase of the compressed area which led to the increase of strain in the longitudinal reinforcement in order to balance the external bending moment. However, with respect to L2000 group, only with slabs $h_f = 30$ mm showed no yielding in the longitudinal reinforcement, while the others showed it, since there is also an increase of the compressed area and the increase in the length of the table width cooperating, adding considerable ductility to the observed failure mode.

There is a proportionality between the increase thickness of the cooperating concrete table with the increase final load. The gain was such that it reached 80% in L1300 group relative to the slab with tables of 30 mm thickness and equal to 131% in the L2000 with respect to the slab with table of 30 mm thickness, showing that the gain in this group is greater than in the previous one, because there is the sum of the cooperating thickness and length of the tables.

With the increase in bearing capacity due to the increase of the tables thickness values all the slabs failure with intense crack-

ing. Failure by shear with or without yielding of the longitudinal reinforcement, leading to a collapse preceded by visible displacements and the development of inclined shear crack between the support and load points. In all slabs the shear failure was substantial, demonstrating the collaboration of resistant mechanism, especially the aggregates interlock and longitudinal reinforcement dowelling as well as the height of the non-cracked concrete area of the slabs with thicker tables.

5.6 Codes' prescriptions analysis

Table 5 shows the results estimated by the NBR 6118 [2], AC1318 [12] and Eurocode 2 [13] codes, and the experimental results. Note that these codes do not consider the contribution of the table (h_f), only the effective depth (d) and the rib width (b_w) as variables related to the geometry that contribute to the shear strength, differ in their estimates due to such verification as a slab or beam or the values of the resistant stress. Brazilian and American codes overestimated the shear strength for slabs with $h_f = 30$ mm and $h_f = 50$ mm, while the European code introduced more conservative values in relation to the experimental results. The best results were obtained for the slabs with table thickness

Figure 17 – L2000 slabs' concrete strains: (a) flexure of external rib; (b) flexure of midrib; (c) distribution of the table

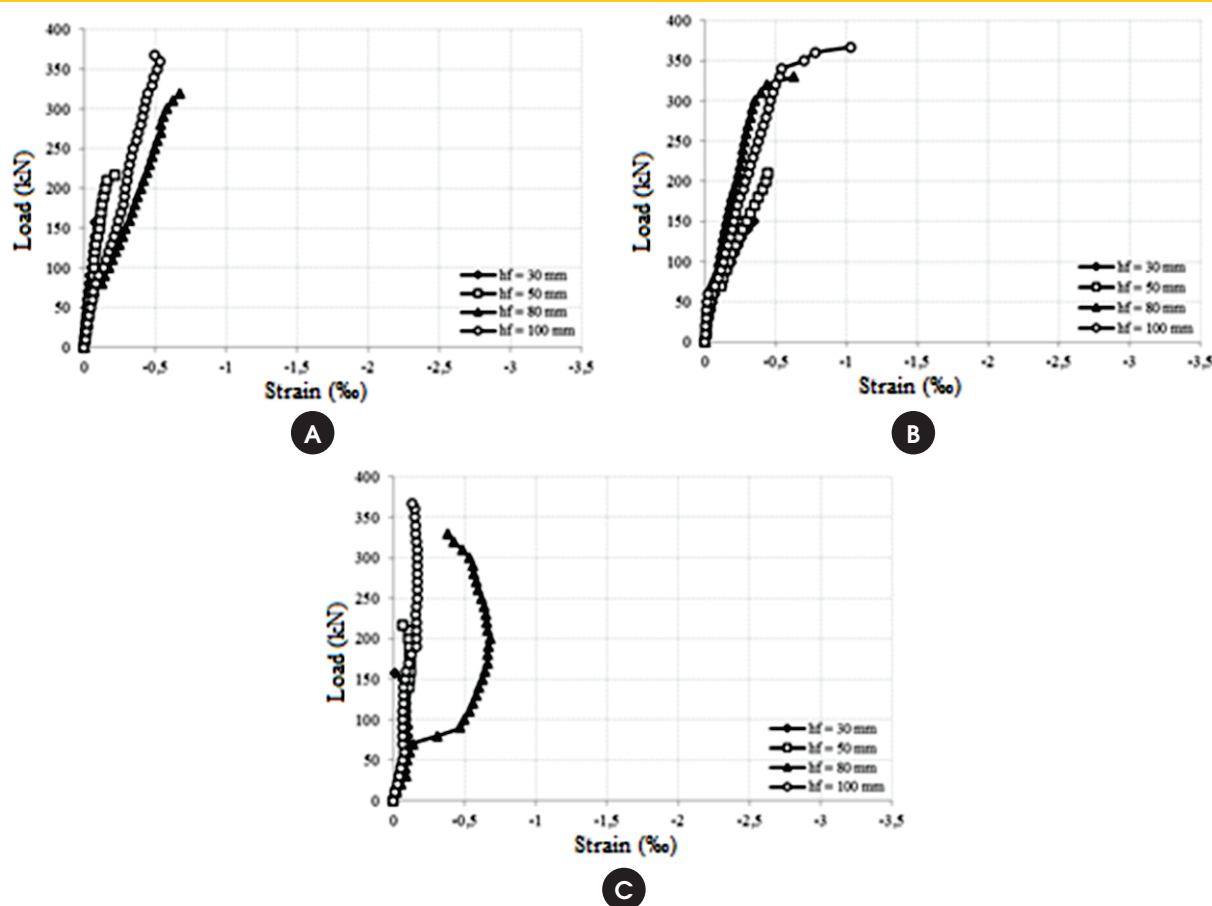
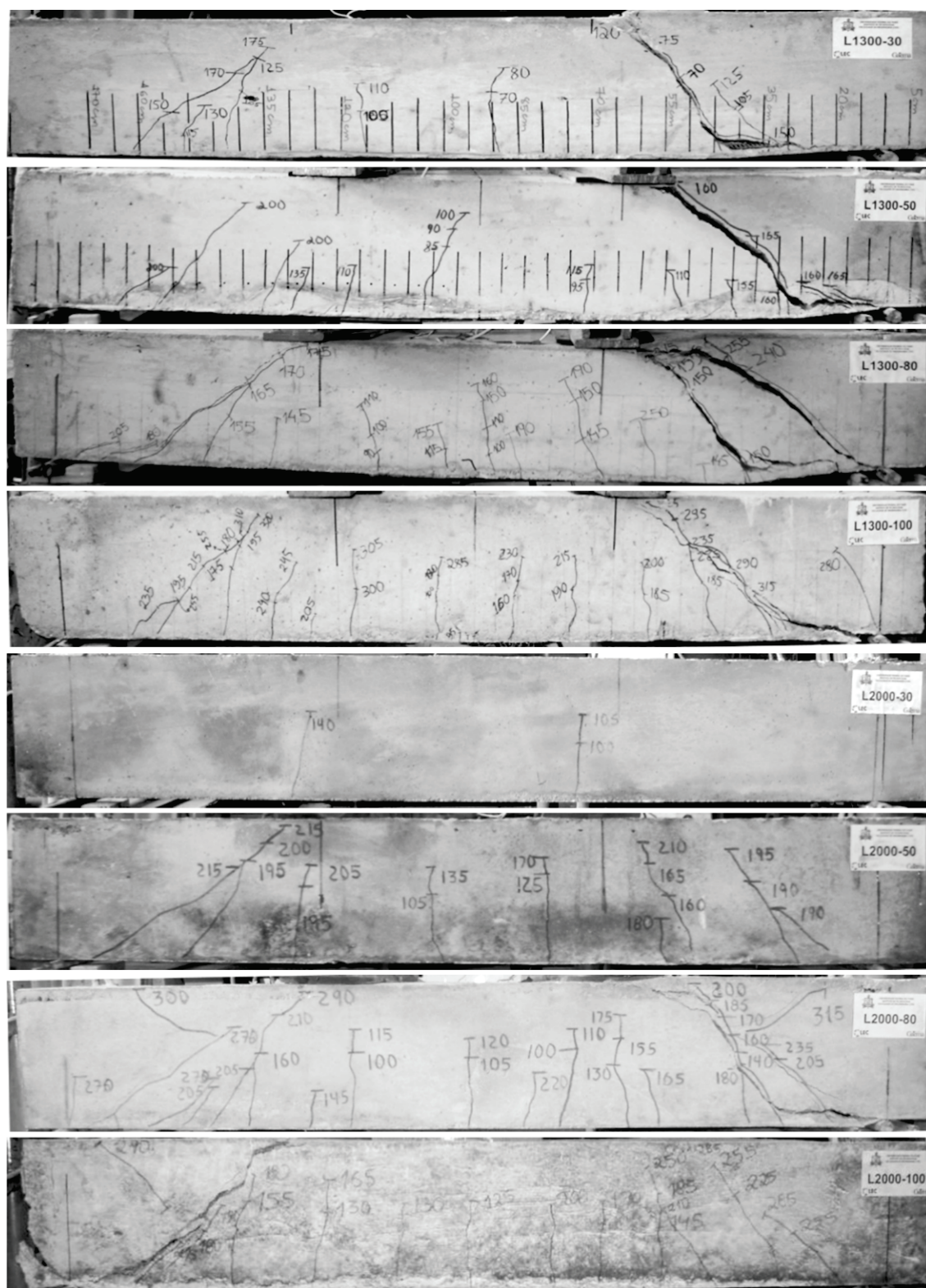


Figure 18 – Cracking pattern of the slabs' external ribs



of 80 mm and 100 mm, although conservative, but in general the NBR 6118 [2] and ACI 318 [12] codes were more accurate than the Eurocode 2 [13]. Thus, the codes' estimates are conservative without consideration of the tables' participation in the shear strength of the slabs.

6. Conclusion

For the tested slabs could be observed that increasing the thickness of the table provides greater shear strength, but the L2000 group had higher failure loads also because of the increased cooperating width. The midrib flexural reinforcement, as expected, showed greater strains due to the layout of the test system which led higher loading demand on this rib. However, even with such configuration it was possible to note that the other ribs, as well as the table, cooperated to the ultimate resistance. The flexural reinforcement strains were proportional to the increase of the table height due to the rebalancing of resistant forces against the increased concrete compressed area, leading to larger strains and allowing its yielding. The limit strain 3.5‰ for concrete crush in flexure was not observed. On the upper surface of all the slabs there was a crack pattern in the table-rib joint indicating the presence of negative bending moment, which confirms the Brazilian code's prescription to check the table against flexural effects for distance between ribs greater than 650 mm. However, in slabs with this distance less than 650 mm the Brazilian code does not recommend this analysis, and the slabs of L1300 group presented cracks due to negative bending moments. The estimates for the ultimate resistance showed that the NBR 6118 [2] and ACI 318 [12] codes presented very similar values and overestimated the bearing capacity for slabs with thin tables, unlike what happened with the Eurocode 2 [13], which presented more conservative values by adopting a lower resistant stress.

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