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Behaviour under cyclic loading of strengthened beams

Comportamento de vigas reforçadas sob ação de carregamento cíclico

A. P. R. VAZ [a]
anapaularvaz@hotmail.com

I. A. E. SHEHATA [b]
ibrahim@coc.ufrj.br

L. C. D. SHEHATA [c]
lidiashehata@vm.uff.br

R. B. GOMES [d]
rbggomes@gmail.com

Abstract

This work presents a study on the behavior of reinforced concrete beams strengthened in bending by the addition of concrete and steel on their tension side and having expansion bolts as shear connectors at the junction between the beam and the jacket, subjected to a cyclic loading. The experimental program included tests on six full scale reinforced concrete beams, simply supported, initially with rectangular cross section 150 mm wide and 400 mm high, span of 4000 mm and total length of 4500 mm. All the beams, after receiving two cycles of static loading in order to create a pre-cracking condition, were strengthened in bending by partial jacketing and then subjected to cyclic loading until the completion of $2x10^6$ cycles or the occurrence of fatigue failure. Following the cyclic loading, the beams that did not fail by fatigue were subjected to a static load up to failure. The main variables were the beam-jacket interface condition (smooth or rough), the flexural reinforcement ratio in the beam and in the jacket, and cyclic load amplitude. On the basis of the obtained test results and the results of previous studies of similar beams tested only under static loading, the behavior of the strengthened beams is discussed and a proposal for the beam-jacket connection design is presented, for the cases of predominantly static and cyclic loading.

Keywords: flexural strengthening, partial jacketing, fatigue, beams, cyclic loading.

Resumo

Este trabalho apresenta estudo sobre o comportamento de vigas de concreto armado reforçadas à flexão, pela adição de concreto e barras de aço na região tracionada e chumbadores de expansão na ligação viga-reforço, submetidas a carregamento cíclico. O programa experimental incluiu ensaios em seis vigas de concreto armado em escala real, simplesmente apoiadas, inicialmente com seção transversal retangular com 150 mm de largura e 400 mm de altura, comprimento entre os apoios de 4000 mm e comprimento total de 4500 mm. Todas as vigas, depois de receber dois ciclos de carga estática, de modo a criar uma condição de pré-fissuração, foram reforçadas à flexão por encamisamento parcial e, em seguida, submetidas a uma carga cíclica até ao final de $2x10^6$ ciclos ou da ocorrência de ruptura por fadiga. Após a aplicação das cargas cíclicas, as vigas que não romperam por fadiga foram submetidas a uma carga estática até a ruptura. As principais variáveis foram a condição de interface de ligação entre viga e reforço (lisa ou rugosa), a taxa de armadura de flexão na viga e no reforço, e amplitude do carregamento cíclico. Com base nos resultados obtidos nos ensaios e em estudos anteriores de vigas semelhantes testadas apenas com carga estática, é feita uma discussão do comportamento dessas vigas reforçadas e apresentada uma proposta para dimensionamento da ligação viga-reforço, para os casos de carregamento predominantemente estático e cíclico.

Palavras-chave: reforço à flexão, encamisamento parcial, fadiga, vigas, concreto armado.

[a] *Saneamento de Goiás S/A, Departamento de Estruturas, Goiânia, GO, Brasil;*
[b] *COPPE, Universidade Federal do Rio de Janeiro, Programa de Engenharia Civil, Rio de Janeiro, RJ, Brasil;*
[c] *Universidade Federal Fluminense, Departamento de Engenharia Civil, Niteroi, RJ, Brasil;*
[d] *Universidade Federal de Goiás, Departamento de Engenharia Civil, Goiânia, GO, Brasil.*

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1. Introduction

Strenghtening reinforced concrete beams by adding concrete and steel bars presents the advantages of relatively low cost and no need for a highly qualified workforce, making it an interesting alternative when it is possible to increase the cross-section dimensions of the element to be strengthened.
The effectiveness of strenghtening by jacketing relies on the efficiency of the connection between the beam and the jacket. The roughness and cleanness of the surface that will receive the new concrete are essential factors for that efficiency. According to [2], the shear strength of an interface between two concretes increases with increasing roughness.
It is a consensus that an adequate curing of the new concrete is needed in order to minimize its initial shrinkage and ensure good bonding between concretes of different ages [3] and there is evidence that interfaces with greater roughness have shear strength less affected by differential shrinkage of the two concretes [4]. The shear strength of the interface between the two concretes can be increased with the use of reinforcement crossing it in two ways: dowel action, which corresponds to the flexural strength combined with axial tension, and by the production of normal stress at the interface, which is an indirect effect mobilized by the relative displacement between the joint. In case of cyclic loading, increasing the ratio of this reinforcement not only decreases the interface damage resulting from such loading, minimizing the loss of stiffness of the strengthened element, but also increases the number of cycles it can withstand.
One of the main factors that can reduce the interface shear strength is the effect of cyclic actions, which cause a decrease in the stiffness of the element, associated to a greater propagation of cracks, leading to strains in the structural elements larger than those verified under short-term static loading, and to different stress redistribution. In view of this, beams that, under static loading exhibit flexural failure, when subjected to cyclic loading, can present shear failure or failure by loss of bond between concrete and reinforcement [5]. It should be noted that the number of cycles the structure supports, as well as the degree of interface damage, is directly related to the amplitude of the cyclic loading to which the structure is subjected.
Although conventional reinforcement, which is attached to the element to be strengthened by means of adhesives, is usually used in practice, in this work, the use of the expansion bolts was chosen due to its easier fixation, without adhesives, leading to greater speed in the execution of strengthening.
Literature review carried out by Vaz [1] shows that there is not much research on the behavior of strengthened reinforced concrete beams by addition of concrete and steel bars and, among the researches reviewed, the ones described in [7], [8], [9], [10], [11] can be cited. From them, only one included beams with cyclic loading [8] and two included beams with expansion bolts at the beam-jacket connection ([10], [11]). In view of the practicality of using expansion bolts at the beam-jacket connection, an experimental study was developed aiming to contribute to the understanding of the behavior of beams strengthened with this technique when they are subjected to cyclic loading. This study, detailed in [1], is summarized here.

2. Experimental program

2.1 Characteristics of the beams and test methods

The main variables of the 6 tested beams were:

- ratio of tensile longitudinal reinforcement of the beams before (1,09% or 0,483%) and after strengthening (0,401%, 0,541%, 1,00% or 1,31%);
- the beam-jacket interface condition (rough or smooth);
- the cyclic load amplitude.

The beams with no strengthening had rectangular cross-sections 150mm wide and 400mm high and a total length of 4500mm. The beams were simply supported, with a distance of 4000mm between the centers of the supports (one roller and one pinned). The concentrated loading was applied at midspan. The beams were designed to have flexural failure, with yielding of longitudinal tensile steel, having sufficient transversal reinforcement to guarantee such a failure. Figure 1 and table 1 show the dimensions and reinforcement of the beams before strengthening.

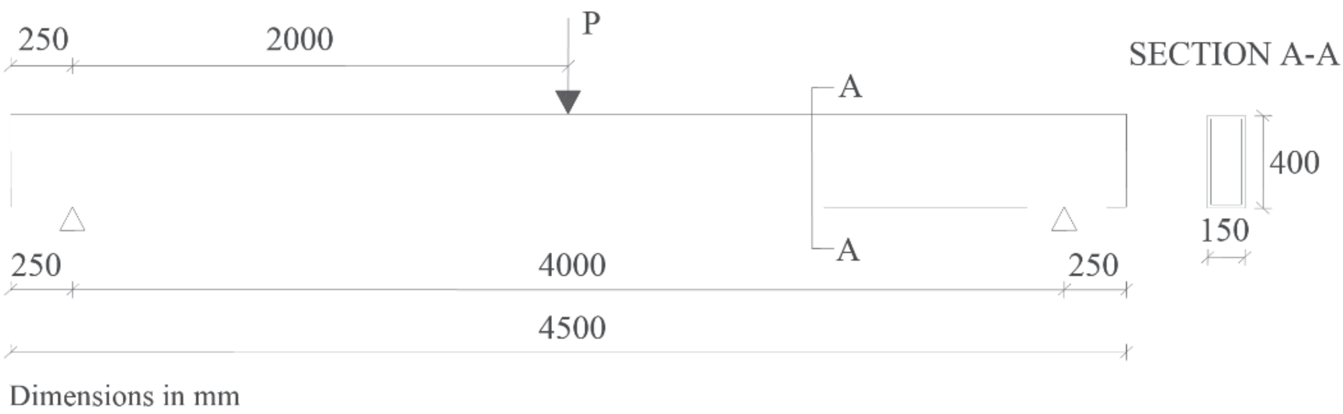

Figure 1
Geometrical characteristics of beams before strengthening

Table 1
Dimensions and reinforcement of the beams before strengthening

Beam	b (mm)	h (mm)	d (mm)	d′ (mm)	A_s (mm_2)	ρ (%)	A_s' (mm^2)	ρ′ (%)	A_{sw}/S (mm^2/mm)	ρ_{sw} (%)
V1 and V2	150	400	369	27	603	1.09	100	0.182	0.670	0.447
V3 to V6	150	400	386	27	280	0.483	100	0.174	0.670	0.447

A_{sw} – cross-section area of shear reinforcement in length s;
s – spacing of shear reinforcement;
A_s – cross section area of longitudinal tensile reinforcement;
A_s' – cross section area of longitudinal compression reinforcement;
ρ – geometrical ratio of longitudinal tensile reinforcement;
ρ′– geometrical ratio of longitudinal compression reinforcement;
ρ_{sw} – geometrical ratio of transverse reinforcement.

Before strengthening, the beams were pre-cracked. This procedure consisted in the application of static loading at the middle of the span until the strains of the bending reinforcement at midspan were around 2.0 ‰. Next, the beams were unloaded, and prepared to be strengthened.

On the lateral faces of the regions that would become beam-jacket interfaces, the surface concrete was removed (depth about 15mm) using a chisel, exposing the reinforcement (tensile longitudinal and transverse) and coarse aggregates. On the lower face, where, in practice, this would be more difficult to do, the cover was not totally removed and the surface was only chipped to make it rough. This procedure was used for beams V1R to V4R, while for beams V5R and V6R the beam surface was left as it was (smooth).

Although more sophisticated methods can be used in laboratory ([12] and [13]), the roughness index R was measured by the sand patch method. After roughening the lower surface of the beams, this method was used in three different regions along the lengths of the beams. Table 2 gives the values of R found and their mean R_m. For cases with R ≥ 1,5mm, according to [13], the surface can be classified as rough.

The strengthening consisted of a reinforced concrete jacket with a trapezoid-shaped cross-section, geometrically equal to the one used in beams of previous work ([10] and [11]), exempt that the width of the lower part of the V5R and V6R jacket measured 180mm instead of 150mm. This difference in V5R and V6R was due to the fact that, prior to strengthening, no concrete surface layer was removed from these beams in the region that would be the beam-jacket interface.

On the sides of the that region, 9.5 mm diameter expansion bolts similar to those used by Santos [10] and Simões [11] were installed,

Figure 2
Expansion bolts

Table 2
Roughness index R values and their mean R_m

Beam	R (mm)	R_m (mm)	Beam	R (mm)	R_m (mm)
V1	1.50	1.54	V3	1.52	1.54
	1.52			1.55	
	1.59			1.56	
V2	1.57	1.55	V4	1.47	1.52
	1.59			1.58	
	1.49			1.52	

and with the same 150mm spacing (figure 2). They served both for positioning the jacket reinforcement and improving the performance of the beam-jacket connection. The ratio of expansion bolts in the beam-jacket connection (ρ_w) was 0.329% for beams with rough surface and 0.298% for those with smooth surface (larger beam-jacket interface area). Considering the yield stress of expansion bolts of the 540MPa, leads to $\rho_w f_y$ values of 1.78 MPa or 1.61 MPa for rough and smooth surface, respectively.

Figure 3 gives details of the reinforcement in the jackets and Table 3 the reinforcement of the strengthened beams of this study and of similar beams of previous works ([10] and [11]).

After about 30 days from casting the jackets, cyclic loading was started, with a frequency of 2Hz or 3 Hz and loads ranging from about 25% to 50%, 30% to 60% or 35% to 70% of the theoretical bending failure load. The beams were subjected to cyclic loading until the occurrence of fatigue failure or completion of a total number of $2x10^6$ cycles. The beams that resisted to $2x10^6$ loading cycles, without having a fatigue failure, were unloaded and, then, subjected to a final static load up to failure.

For testing, the beams were simply supported (one roller and a pinned support) having a span of 4000mm. They were loaded at mid-span using a 500kN capacity jack connected to a load/displacement control system.

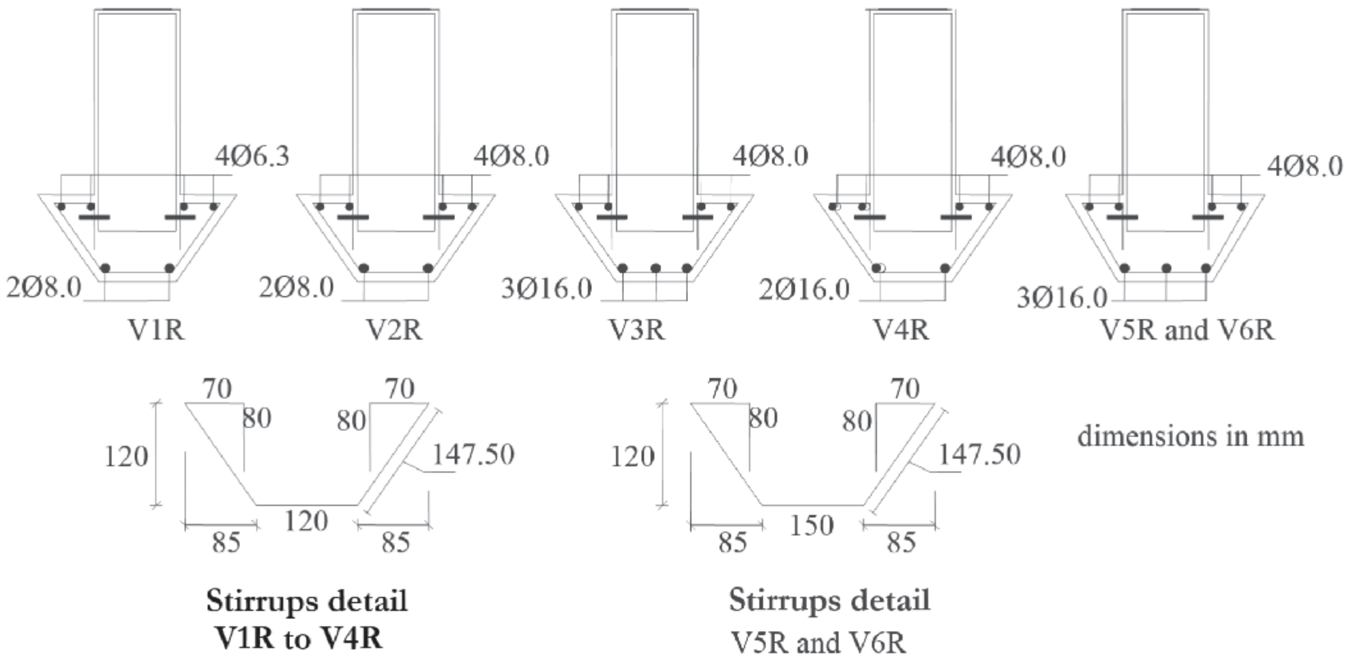

Figure 3
Reinforcement in the jackets of beams V1R to V6R

Table 3
Dimensions and reinforcement of the strengthened beams

Current work							
Main steel (ϕ in mm)			d_R	A_s (mm^2)	A_{sR} (mm^2)	ρ_R (%)	ρ_T (%)
	Beam	Jacket					
V1R	3ϕ16,0	4ϕ6,3 and 2ϕ8,0	374	603	225	0.401	1.48
V2R	3ϕ16,0	6ϕ8,0	372	603	302	0.541	1.62
V4R	2ϕ10,0 and 1ϕ12,5	4ϕ8,0 and 2ϕ16,0	402	280	603	1.00	1.47
V3R, V5R, V6R	2ϕ10,0 and 1ϕ12,5	4ϕ8,0 and 3ϕ16,0	409	280	804	1.31	1.77

Earlier works							
Main steel (ϕ in mm)			d_R	A_s (mm^2)	A_{sR} (mm^2)	ρ_R (%)	ρ_T (%)
	Beam	Jacket					
VR1 [10]	3ϕ16,0	6ϕ8,0	372	603	302	0.541	1.62
VR2 [11]	2ϕ10,0 and 1ϕ12,5	4ϕ8,0 and 2ϕ16,0	402	280	603	1.00	1.47
VR3 [11]	2ϕ10,0 and 1ϕ12,5	4ϕ8,0 and 3ϕ16,0	409	280	804	1.31	1.77

b = 150mm; h = 470mm; d´ = 27mm; A_s´ = 100mm²
Stirrups (jacket): ϕ5,0 c/150 mm

During the static loading tests, concrete strains were measured at four levels of a section at 130mm from the midspan (figure 4), by means of a demec gauge with 100 mm length gage and a smaller division of 0.001mm. Strains of the longitudinal tensile reinforcement were measured using electrical resistance strain gauges stuck on the bars of the beams and the jackets, at midspan and at a section 960mm from midspan (figure 5). The vertical displacements of the beams were measured using two strain gauge displacement transducers, at sections 150mm apart from the midspan and at each side of the loading region. The relative horizontal

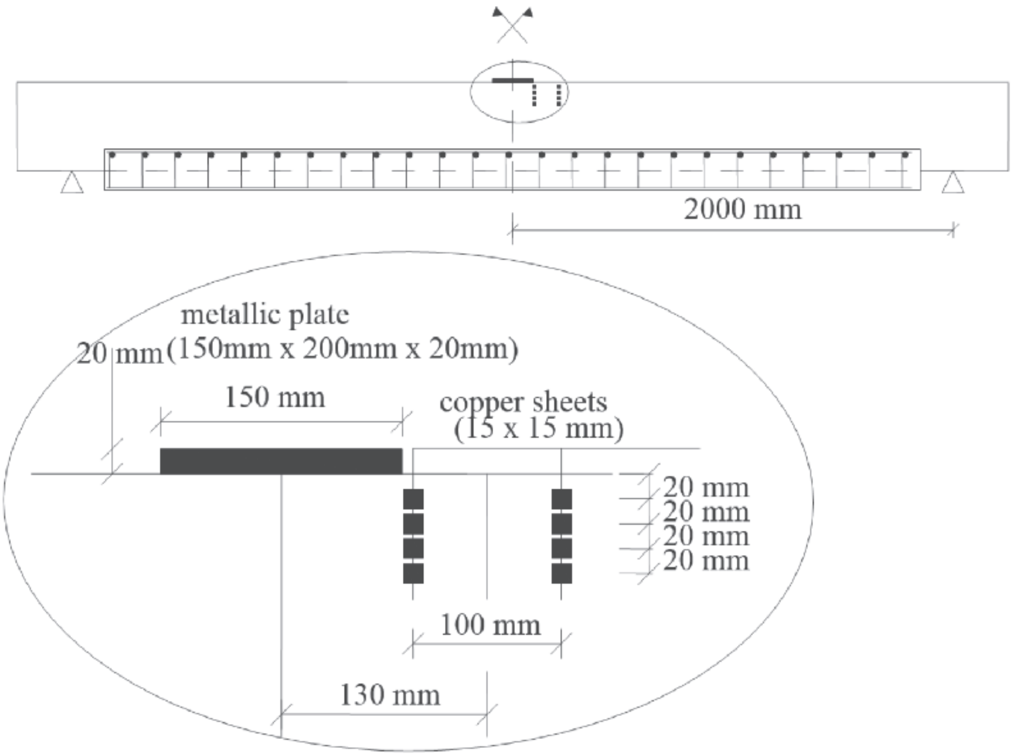

Figure 4
Position of bases for demec gauge measurements

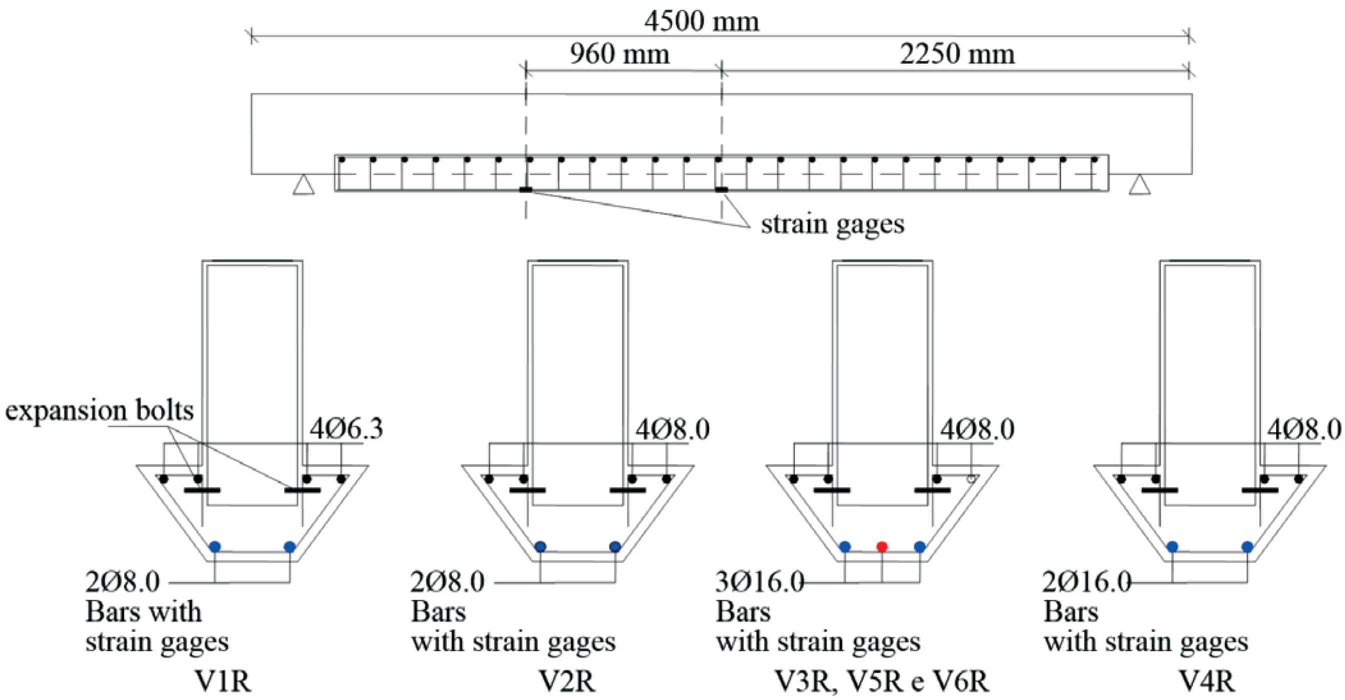

Figure 5
Cross-section of strengthened beams and strain gauges positions on longitudinal tensile reinforcement of the jackets

displacements at the beam-jacket connection were measured with strain gauge displacement transducers placed at the section at 960mm from midspan and at the end of the jacket using aluminium devices (figure 6).

2.2 Materials

CA-50 and CA-60 steel bars were used as reinforcements of beams and jackets. The transverse reinforcement of the beams and of the jackets had 8.0 mm and 5.0 mm diameter, respectively. The longitudinal compression reinforcement of the beams consisted of 8.0 mm diameter bars, as well as the longitudinal reinforcement of jackets together with 6.3 mm bars. The longitudinal tensile reinforcement of beams V1 and V2 and some of the jackets had 16.0 mm diameter. Bars of 10.0 mm and 12.5 mm diameter were used as longitudinal tensile reinforcement of beams V3 and V6. Samples of each type of bar were tested and the average values of yield stress and tensile strength obtained were, respectively, 655 MPa and 739 MPa (5.0 mm), 596 MPa and 767 MPa (6.3 mm), 607 MPa and 748 MPa (8.0 mm), 522 MPa and 641 MPa (10.0 mm), 555 MPa and 688 MPa (12.5 mm), 562 MPa and 686 MPa (16.0 mm).

The concrete mix was chosen aiming a compressive strength of 30 MPa at 28 days. For each concrete batch cylindrical specimens

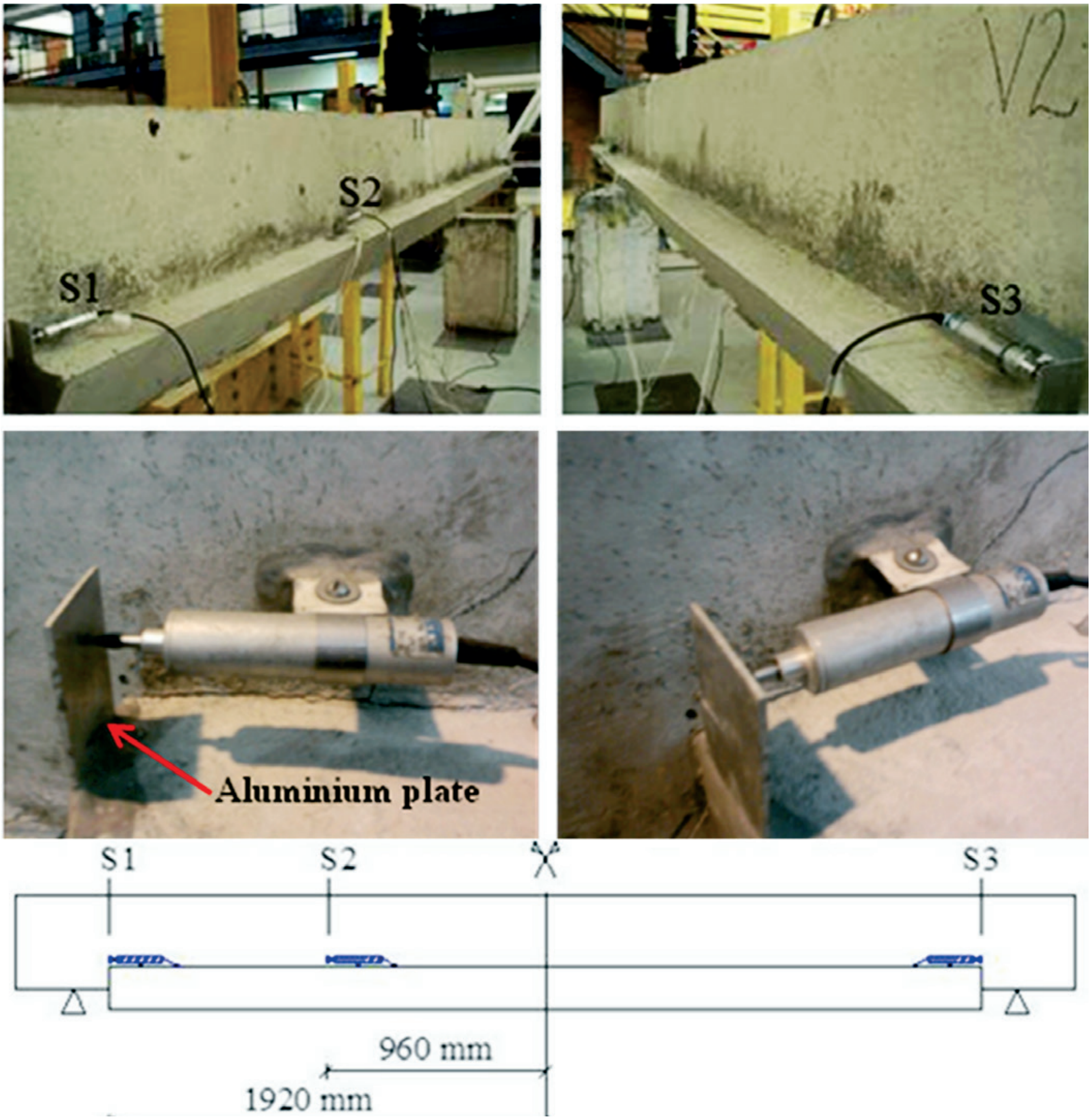

Figure 6
Position of displacement transducers for measurement of relative longitudinal displacements between beam and jacket

were moulded for the compression, tensile and modulus of elasticity tests. The mean values obtained in the tests related to concretes of the beams and jackets were, respectively: 34.0 MPa and 33.0 MPa for compression strength, 3.60 and 3.57 MPa for splitting tensile strength and 26.0 GPa for modulus of elasticity.

3. Results and discussion

Table 4 summarizes the experimental results of the tested beams. The theoretical bending failure load values of the strengthened beams, $P_{u,theo}$, used in this table were determined using f_c and f_y obtained in the material tests (γ_c=γ_s=1) and are given in table 5.

3.1 Cracks and load

In the strengthened beams, some cracks on the jackets were observed during the two static loading cycles before the cyclic loading. During the cyclic loading, other flexural cracks appeared in the jacket up to about the first 100 000 cycles for the beams V1R, V2R, V3R and V5R, 70 000 cycles for the V4R and 5000 cycles for the V6R. Besides theses cracks, in V4R and V6R, with a load variation between 25% to 50% of $P_{u,theo}$ and higher $\rho_R f_y$ values (5,69MPa and 7,39MPa), shear cracks appeared in the regions near the supports and horizontal cracks in the beam-jacket connection. Beam V6R, that failed by fatigue of the expansion bolts at beam-jacket

Table 4
Loading and results of the tested beams

Initial static loading (before strengthening)								
Beam	ρ_R (%)	ρ_T (%)	P_{cr} (kN)	P_{max} (kN)	δ_i (mm)	$\varepsilon_{s,i}$ (‰)	$\varepsilon_{s,ires}$ (‰)	$\delta_{res,i}$ (mm)
V1 V1R	0.401	1.48	25	60	8.73	1.78	0.383	2.31
V2 V2R	0.41	1.62	25	60	8.88	1.72	0.368	2.00
V3 V3R	1.31	1.77	20	35.4	8.58	1.99	0.527	2.43
V4 V4R	1.00	1.47	15	35.3	8.96	2.09	0.602	2.95
V5 V5R	1.31	1.77	20	35.6	7.66	2.08	0.497	2.09
V6 V6R	1.31	1.77	20	35.5	7.68	1.97	0.481	2.15

Cyclic loading (strengthened)					
Beam	$P_{min}/P_{u,teo}$ (%)	$P_{max}/P_{u,teo}$ (%)	$\varepsilon_{s,max}$ (‰)	$\varepsilon_{s,res}$ (‰)	δ_{res} (mm)
V1 V1R	33	64	2.36	–	–
V2 V2R	19	40	1.28	0.352	1.74
V3 V3R	22	42	1.57	0.345	2.52
V4 V4R	24	58	1.64	–	4.54
V5 V5R	21	42	1.37	0.301	2.21
V6 V6R	27	54	1.49	–	–

Final static loading (strengthened)								
Beam	$P_{u,exp}$ (kN)	$\delta_{u,exp}$ (mm)	$\varepsilon_{su,exp}$ (‰)		Maximum relative displacement (mm)			Failure mode
			Beam	Jacket	S1	S2	S3	
V1 V1R	–	–	–	–	–	–	–	Steel fatigue 1.865.825 cycles
V2 V2R	193	33.8	46.7	50.52	–	0.081	0.058	Flexure
V3 V3R	180	18.1	2.24	2.59	0.754	0.051	9.85	Shear at beam-jacket interface
V4 V4R	186	25.3	–	–	0.641	0.04	0.444	Flexure
V5 V5R	173	28.6	2.02	2.20	1.34	0.843	6.57	Shear at beam-jacket interface
V6 V6R	–	–	–	–	–	–	–	Bolts fatigue 875.280 cycles

ρ_R – tgeometric ratio of longitudinal tensile reinforcement of jacket; ρ_T – total geometric ratio of longitudinal tensile reinforcement of strengthened beam; δ_i – vertical displacement corresponding to the maximum load P_{max} in the initial static test; $\delta_{res,i}$ – residual vertical displacement at the end of the initial static test; δ_{res} – residual vertical displacement at the end of cyclic loading; $\delta_{u,exp}$ – vertical displacement at failure load of beams that did not have fatigue failure;
$\varepsilon_{s,i}$ – longitudinal reinforcement strain of the beam corresponding to the maximum load in the initial static test; $\varepsilon_{s,res}$ – residual strain of longitudinal reinforcement of the beam at the end of initial static test; $\varepsilon_{s,max}$ – maximum strain of longitudinal reinforcement of the beam during cyclic loading; $\varepsilon_{u,exp}$ – maximum strain of longitudinal reinforcement of the beam or jacket at failure load of the beams that did not have fatigue failure;
P_{cr} – load corresponding to the first visible cracks during initial static test; $P_{u,exp}$ – experimental failure load of the beams that did not have fatigue failure.

Table 5
Experimental and theoretical failure loads of strengthened beams of [1] and previous work [10] and [11]

					Static loading Rough beam-jacket interface			Cyclic loading Rough beam-jacket interface			Cyclic loading Smooth beam-jacket interface		
Beams	ρ_R (%)	ρ_T (%)	ρ_{Rfy} (MPa)	ρ_{Tfy} (MPa)	$P_{u,exp}$ (kN)	$P_{u,teo}$ (kN)	$P_{u,exp}/P_{u,teo}$	$P_{u,exp}$ (kN)	$P_{u,teo}$ (kN)	$P_{u,exp}/P_{u,teo}$	$P_{u,exp}$ (kN)	$P_{u,teo}$ (kN)	$P_{u,exp}/P_{u,teo}$
V1R	0.401	1.48	2.29	8.46	–	–	–	–	–	–	Steel fatigue		
VR1 [10]	0.541	1.62	2.97	8.92	186	156	1.19	–	–	–	–	–	–
V2R	0.541	1.62	3.12	9.33	–	–	–	193	168	1.15	–	–	–
VR2 [11]	1.00	1.47	6.11	8.98	205	192	1.07	–	–	–	–	–	–
V4R	1.00	1.47	5.69	8.29	–	–	–	186	177	1.05	–	–	–
VR3 [11]	1.31	1.77	8.29	10.8	229	230	1.00	–	–	–	–	–	–
V3R	1.31	1.77	7.39	9.98	–	–	–	180	212	0.85	–	–	–
V5R	1.31	1.77	7.39	9.98	–	–	–	–	–	–	173	216	0.80
V6R	1.31	1.77	7.39	9.98	–	–	–	–	–	–	Bolts fatigue		

☐ Flexure ☐ Shear at the beam-jacket interface ☐ Fatigue

connection, presented the greatest number of shear cracks during the cyclic loading. In V1R, with lower longitudinal reinforcement ratio in the jacket ($\rho_R f_y$=2,29MPa) and a load variation between 32% and 64% of $P_{u,theo}$, these horizontal and shear cracks did not appear. It had failure by fatigue of the main steel of the jacket and showed only bending cracks and with greater width.

During the final static test, beams V2R and V4R, that had bending failure, presented similar cracking patterns. The beams V3R and V5R, which had shear failure at the beam-jacket connection, presented smaller number of flexural cracks than the other beams. The theoretical and experimental values of bending strength of the strengthened beams of this and previous researches ([10] and [11]), calculated with experimental values of concrete compressive strength and steel yield stress and parabola-rectangle diagram for normal compression stresses in concrete, are presented in table 5. The experimental maximum values of normal stresses and the variation of these stresses in the longitudinal tensile reinforcement at the beginning of cyclic loading, obtained from the measured steel strains, and also the calculated ones are given in table 6.

Table 5 shows that only the beams of this study that had shear failure at the beam-jacket connection had a $P_{exp}/P_{u,theo}$ $P_{exp}/P_{u,theo}$ ratio smaller than one (0.85 and 0.80) and that the beam with the smooth beam-jacket interface had the lowest ratio. For the beams of previous work [11] similar to those of this study, but subjected to a static loading only, with shear failure at the beam-jacket connection, this ratio was 1.07 and 1.00. Comparing V3R with VR3 [11] (ρ_T = 1.77% and ρ_R = 1.31%, with rough beam-jacket interface and shear failure at this connection), it is found that the V3R had a resistance capacity 21% smaller. However, this reduction cannot be attributed only to cyclic loading, since V3R had $\rho_R f_y$ 11% lower. Taking this into account, the reduction of strength due to cyclic loading becomes 12%.

In beams with ρ_T = 1.62% and ρ_R = 0.541% (V2R of this study and VR1[10]), which had bending failure, the cyclic loading did not affect the resistance capacity, since the difference between the experimental failure loads of beams with only static loading and with

Table 6
Maximum normal stresses and variation of these stresses in the longitudinal tensile reinforcement at the beginning of cyclic loading, obtained from measured and calculated steel strains

Beams	ρ_{Rfy} (MPa)	ρ_{Tfy} (MPa)	$P_{min}/P_{u,teo}$ (%)	$P_{máx}/P_{u,teo}$ (%)	$\sigma_{s,max}$ (MPa)	$\sigma_{s,max,calc}$ (MPa)	$\Delta_{\sigma s}$ (MPa)	$\Delta_{\sigma s,calc}$ (MPa)	$\sigma_{sR,max}$ (MPa)	$\sigma_{sR,max,calc}$ (MPa)	$\Delta_{\sigma sR}$ (MPa)	$\Delta_{\sigma sR,calc}$ (MPa)
V1R	2.29	8.46	32	64	299	366	155	183	477	494	231	247
V2R	3.12	9.33	20	40	187	234	100	117	225	317	102	159
V4R	5.69	8.29	25	56	265	304	152	167	330	380	174	209
V3R	7.39	9.98	21	42	172	213	87	107	253	269	116	135
V5R	7.39	9.98	21	42	173	213	98	107	225	269	111	135
V6R	7.39	9.98	27	54	227	275	130	137	293	347	157	174

static loading after the cyclic one corresponds almost to the difference between the values of $\rho_T f_y$ of these beams.
The V1R (ρ_T = 1.48% and ρ_R = 0.401%) had the highest $P_{min}/P_{u,theo}$ and $P_{max}/P_{u,theo}$ ratios of the tested beams (32% and 64%) and, consequently, larger variations of normal stress in the reinforcement in the jacket (causing fatigue in that reinforcement) and of vertical displacement. This beam had $\sigma_{sR,max}$= 477MPa ≈ 0,79 f_y and $\Delta\sigma_{sR}$=231MPa. In figure 7, it can be seen that, for the number of cycles verified in V1R (1 865 825), there was a variation of normal stress in the reinforcement greater than the limit given by the relationship between $\Delta\sigma_s$ and the number of cycles N of ABNT NBR 6118:2014 [14]. Due to the low steel ratio used in the jacket, the shear stress variation at the beam-jacket interface was low and there was no slipping at that connection.

3.2 Reinforcement strains

For the static load before the cyclic one, in general, the ratios between the strains of the reinforcement in the jacket (bottom layer) and in the beam varied between 1.2 and 1.4, values that would be expected according to a state II analysis, but in beam V1R these ratios ranged from 1.5 to 1.7.
During cyclic loading, except for V2R, the measured maximum and minimum strains of the longitudinal tensile steel, as a function of number of cycles, for beams that did not have fatigue failure did not show a stabilization tendency. Beam V6R, that had fatigue failure of the expansion bolts, had a differentiated behavior, presenting a sudden decrease in the strains of the longitudinal reinforcement of the jacket and, at the same time, a sudden increase in the strains of the longitudinal reinforcement of the beam, when N was equal to about 600 000 cycles (figure 8).
Table 7 lists the values of the maximum strains of the longitudinal tensile reinforcement of jackets and beams measured during cyclic loading, as well as the variation of these strains and the residual strains at the end of loading. These values depend on P_{min}/P_{theo} and P_{max}/P_{theo} and on ρ_T.
The strains of longitudinal reinforcement in the jacket measured during the final static loading at midspan of V2R are compared with the ones of beam VR1[10] in figure 9, and those of beams V3R and V5R compared with the ones of V3R [11] in figure 10. The curves of beams VR1[10] and V2R are practically coincident for load values up to 50kN; for higher loads, the curve of V2R, with $\rho_T f_y$ about 5% higher, show smaller strains.
In figure 10, it can be seen that, for the same load, beam V5R, with smooth beam-jacket interface, presented smaller strains than

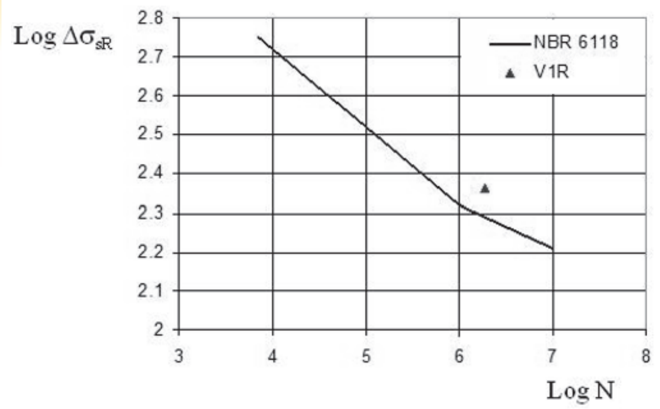

Figure 7
Comparison between the relationship between N and $\Delta\sigma_{sR}$ according to ABNT NBR 6118:2014 [14] and the one of the beam with steel fatigue failure

V3R, with rough surface, and that the strains curve of V3R is close to that of VR3 [11], and although the two beams had the same kind of failure, the excessive slipping at the beam-jacket interface of V3R prevented the longitudinal reinforcement of the jacket from having strains higher than ε_y* (strain corresponding to f_y when a steel bilinear normal tensile stress-strain diagram with plateau is considered) and caused V3R to fail at a lower load than VR3 [11], for which strains larger than the yielding one were measured (about 8.5‰).

3.3 Longitudinal force, shear stress and slipping at the beam-jacket connection

The longitudinal force T_R, and, from it, the shear stress at the beam-jacket interface τ, was calculated using the strains measured in the longitudinal reinforcement of the jacket at midspan. Table 8 shows $T_{R,max}$ and ΔT_R during cyclic loading, obtained from measured and calculated strains. The sum of the jacket longitudinal reinforcement forces at midspan of each beam obtained from the measured strains, for different levels of the final static loading, is in table 9 and figure 11 gives these longitudinal forces as a function of the applied load for beams V3R, V5R and V6R, together with the ones of VR3 [11], with the same tensile longitudinal reinforcement ratio. This figure shows the variation that the measured strains may have as a result of cracking, since, from equilibrium condition at

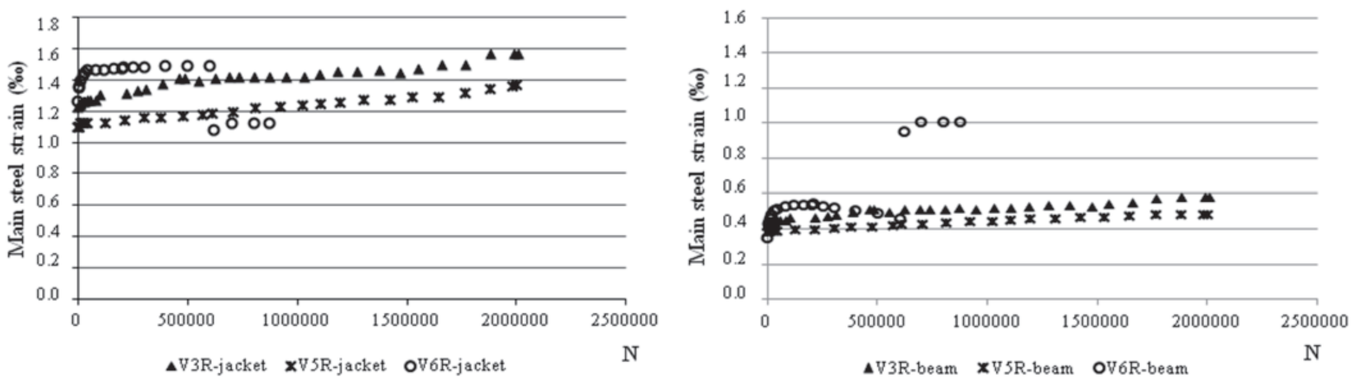

Figure 8
Maximum main steel strain of jacket and beam, at midspan, as a function of the number of cycles of beams with same reinforcement ratio, with rough (V3R) and smooth interface (V5R and V6R)

Table 7
Maximum strain of the longitudinal tensile reinforcement of jacket and beam and strain variation during cyclic loading and residual strain at the end of loading

Beam	ρ_R (%)	ρ_T (%)	$P_{min}/P_{u,teo}$ (%)	$P_{max}/P_{u,teo}$ (%)	$\varepsilon_{sR,max}$ (‰)	$\Delta_{\varepsilon sR}$ (‰)	$\varepsilon_{sR,res}$ (‰)	$\varepsilon_{s,max}$ (‰)	$\Delta_{\varepsilon s}$ (‰)	$\varepsilon_{s,res}$ (‰)
V1R	0.401	1.48	32	64	2.36+	1.15+	–	1.49+	0.777+	–
V2R	0.541	1.62	20	40	1.28	0.532	0.352	1.04	0.539	0.244
V4R	1.00	1.47	25	56	1.64+	0.863+	–	1.41+	0.842+	–
V3R	1.31	1.77	21	42	1.57	0.576	0.345	1.09	0.501	0.235
V5R	1.31	1.77	21	42	1.37	0.769	0.301	1.08	0.579	0.229
V6R	1.31	1.77	27	54	1.49*	0.836*	–	1.24*	0.781*	–

+ < 100.000 cycles | * 600.000 cycles

midspan, for a given load, beams with same longitudinal steel ratio must have the same value of T_R.
Table 10 lists the values of horizontal force at the beam-jacket connection (static loading) and the variation of that force during cyclic loading for beams that had shear failure at the beam-jacket connection, and also for V4R.
On the basis of the $T_{R,max}$ and ΔT_R values for beams V5R and V6R given in table 10, and considering that the longitudinal force at beam-jacket connection is resisted only by the expansion bolts (24 bolts in all, disregarding the two bolts at midspan), the equations (1.0a) and (1.0b) that give the expansion bolt shear force variation, $\Delta T_{R,ch}$, as a function of the number of cycles could be deduced. The one that gives $\log\Delta T_{R,ch}$ as a function of logN (1.0a) is of the type commonly used for shear connectors [15] and the one that gives $\Delta T_{R,ch}$ as a function of logN (1.0b) is its equivalent. The same expression was assumed for smooth and rough interfaces, in view of the little difference between the strengths of beams V3R and V5R, and $T_{R,ch}$ is given in newtons.

$$\log \Delta T_{R,ch} = -0{,}0811 \log N + 4{,}204 \quad \text{(1a)}$$

or

$$\Delta T_{R,ch} = 16000\, e^{-0'187 \log N} \quad \text{(1b)}$$

These expressions can be written in the form of shear stress variation in the bolts (in MPa), that is

$$\log \Delta T_{R,ch} = -0{,}0811 \log N + 2{,}354 \quad \text{(2a)}$$

or

$$\Delta \tau_{ch} = 226\, e^{-0'187 \log N} \quad \text{(2b)}$$

It should be noted that, for N=1, this variation is approximately equal to $0.4f_y$ and that, according to Tresca failure criteria (which is more conservative than the Von Mises one), the shear stress limit would be $0.5f_y$. This lower resistance can be explained by the stress concentration due the existence of thread in the expansive bolts and by the fact that the expansive bolts are not subjected to pure shear.

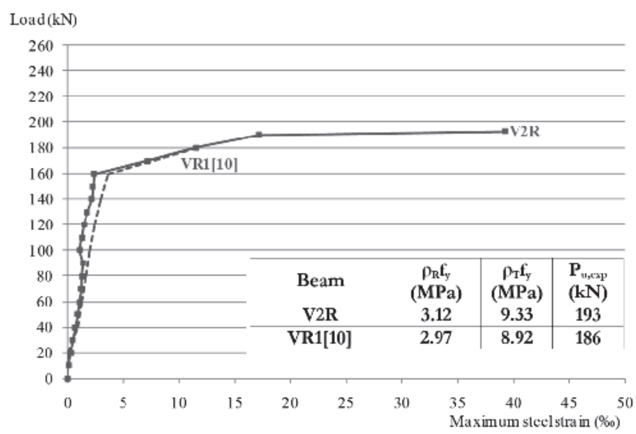

Figure 9
Load - maximum longitudinal steel strain curves for beams VR1 [10] and V2R

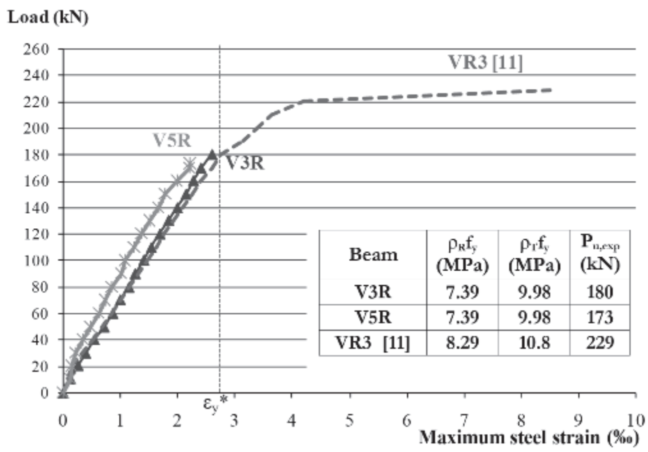

Figure 10
Load - maximum longitudinal steel strain curves for beams VR3 [11], V3R and V5R

Table 8
Jacket main steel maximum force $T_{R,max}$ and force variation ΔT_R under cyclic loading, obtained from measured and calculated steel strains

Beam	ρ_R (%)	ρ_T (%)	P_{min} (kN)	P_{max} (kN)	$T_{R,max}$ (kN)	$T_{R,max,calc}$ (kN)	$T_{R,max,calc}/T_{R,max}$	Δ_{TR} (kN)	$\Delta_{TR,calc}$ (kN)	$\Delta T_{R,calc}/\Delta_{TR}$
V1R	2.29	8.46	50	100	79.2	89.8	1.13	40.2	44.8	1.11
V2R	3.12	9.33	34	68	58.0	73.1	1.26	30.4	36.5	1.20
V4R	5.69	8.29	45	100	186	201	1.08	102	111	1.09
V3R	7.39	9.98	45	90	179	196	1.10	84.6	98.2	1.16
V5R	7.39	9.98	45	90	167	196	1.18	87.2	98.2	1.13
V6R	7.39	9.98	58	116	218	253	1.16	120	127	1.06

Figure 12 shows the relationship between the shear resistance at the connection, τ_R, and ρ_w f_y given by the expression τ_R = 0,4 ρ_w f_y and those of beams VR2 and VR3 of [11] and V3R and V5R. All of them had longitudinal shear failure at the beam-jacket interface during monotonically increasing load, but VR2 and VR3 [11] had not been previously subjected to cyclic loading. Beam V4R was also included, although it had bending failure, because shear failure at the beam-jacket connection was imminent. The τ_R values of V4R, V3R and V5R are those of residual shear strength after cyclic loading. This figure shows that, with the exception of V4R, which did not have failure at the beam-jacket connection, the expression τ_R = 0,4 ρ_w f_y leads to values of τ_R smaller than or approximately equal to the ones of the beams analyzed, for both beams tested only statically and for those subjected to cyclic loading before the static one (residual resistant shear stress).

Figures 12 and 13 suggest that the beam-jacket connections provided with expansion bolts can be designed considering

$$\Delta\tau \leq 0{,}4\, \rho_w f_{yd} [e^{-0{,}187 \log N}] \quad (3)$$

where N is the expected loading cycles, $\Delta\tau$ is obtained from the longitudinal forces values at the connection T_{Rmax} and T_{Rmin} calculated at stage II, for maximum service load (permanent loads + frequent variable loads) and minimum (permanent loads), respectively, and $f_{yd} = f_y/1{,}15$.

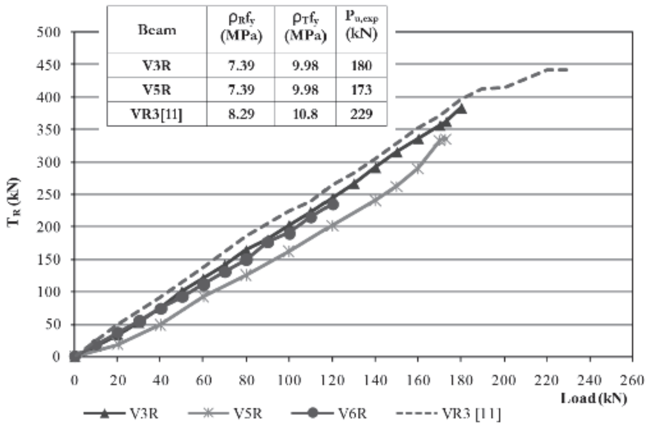

Figure 11
Load - T_R curves of beams VR3 [11], V3R, V5R and V6R, with same reinforcement ratio

The maximum displacements between beam and jacket, in the

Table 9
Longitudinal force and nominal shear stress at the beam-jacket interface during final static loading

Load (kN)	T_R (kN)						τ (MPa)					
	V1R+	V2R	V4R+	V3R	V5R	V6R	V1R+	V2R	V4R+	V3R	V5R	V6R+
0	0	0	0	0	0	0	0	0	0	0	0	0
20	8.83	17.9	30.7	33.2	19.1	36.3	0.017	0.0330	0.0571	0.0618	0.0321	0.0611
40	26.3	36.3	70.6	75.2	48.5	73.9	0.049	0.0680	0.131	0.140	0.0814	0.124
60	43.6	56.0	110	120	92.2	110	0.081	0.104	0.206	0.224	0.155	0.186
80	60.0	72.2	151	163	125	150	0.112	0.134	0.282	0.305	0.210	0.252
100	71.5	82.4	185	202	162	190	0.133	0.153	0.346	0.376	0.272	0.320
120	–	104	–	244	202	234	–	0.194	–	0.454	0.339	0.395
140	–	133	–	291	241	–	–	0.247	–	0.541	0.405	–
150	–	142	–	315	262	–	–	0.267	–	0.586	0.439	–
160	–	144	–	335	290	–	–	0.299	–	0.624	0.487	–
170	–	161	–	356	333	–	–	0.299	–	0.662	0.561	–
173	–	161	–	361	335	–	–	0.299	–	0.673	0.563	–
180	–	161	–	383	–	–	–	0.299	–	0.712	–	–
193	–	161	–	–	–	–	–	0.299	–	–	–	–

+ static loading before cyclic loading (beams with failure during cyclic loading and V4R, where there was damage in the strain gauges)

Table 10
Values of $T_{R,max}$ and Δ_{TR} corresponding to static and cyclic loading

			Only static loading	During cyclic loading	Static load after cyclic
Beam	ρ_R (%)	ρ_T (%)	$T_{R,max}$ (kN)	Δ_{TR} (kN)	$T_{R,max}$* (kN)
VR2 [11]	1.00	1.47	370	–	–
V4R	1.00	1.47	–	111++	348
VR3 [11]	1.31	1.77	490	–	–
V3R	1.31	1.77	–	98.2++	401
V5R+	1.31	1.77	–	98.2++	385
V6R+	1.31	1.77	–	127+	–

+ ssmooth interface; $T_{R,max}$* after cyclic; ++ no failure during cyclic loading; + fatigue failure with 875.280 cycles.

beams V2R and V4R, that had bending failure, were 0.260 mm and 0.641 mm, respectively. In beams V3R and V5R, with shear failure at the beam-jacket connection, displacements of 0.754 mm and 1.34 mm were registered in the position S1 and 9.85 mm (V3R) and 6.57 m (V5R) in the position S3. Figure 14 shows the relationship between load and slipping at the beam-jacket connection of beams VR3 [11] and V3R and V5R, where plateaus indicate the effect of the expansive bolts used in the connection. Observing in figure 14 the curves of beams with rough beam-jacket interface, V3R, and VR3 [11], it can be seen that, for the same relative displacement value, VR3 [11], which had no cyclic loading, had a higher load.

4. Conclusions

There are few experimental studies on shear strength of concrete connections under cyclic loading and, in the literature review [1], no study on connections with expansion bolts was found. According to the MC 2010 (FIB, 2013), in the design of interfaces subjected to cyclic loading, it is recommended a reduction to about 40% of the static resistance, if cracks are likely to occur at the connection.
In view of the advantages of the bending strengthening by adding concrete and steel bars and expansion bolts at the beam-jacket connection, the experimental study described here was developed aiming to investigate the behavior of beams strengthened according to this technique, under unidirectional cyclic loading with different amplitudes.
The comparison between similar strengthened beams, with rough beam-jacket interface, tested only statically with those that had cyclic loading before the static one showed that cyclic loading had no negative influence on the strength capacity of beams with bending failure (V2R, V4R).
The beams with shear failure at the beam-jacket connection, unlike the similar ones tested under only static loading, the larger relative displacements verified at the beam-jacket connection of the beams tested with cyclic loading prevented the longitudinal tensile reinforcement from having strains larger than that corresponding to the beginning of yielding. From the beams that differed only by

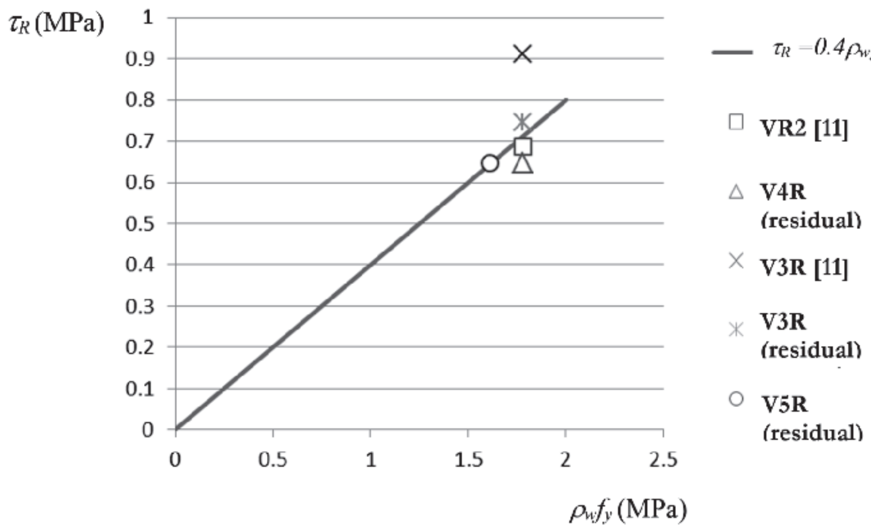

Figure 12
Relationship between τ_R and ρ_{wfy} according to expression $\tau_R = 0{,}4\rho_{wfy}$ and those of beams VR2 and VR3 [11] and the residual shear stress of V4R, V3R and V5R

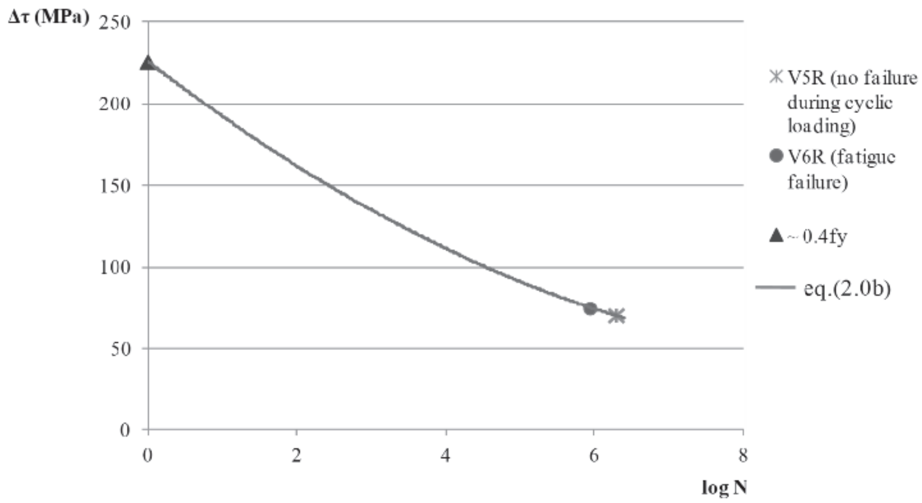

Figure 13
Shear stress variation in beam-jacket interface as a function of the number of cycles given by equation (2.0b)

the condition of the beam-jacket interface (V3R and V5R), the one with smooth interface had resistance practically equal to the one with rough interface.
Tests of strengthened beams and direct shear tests of connections between concretes with expansions bolts previously carried out with no cyclic loading showed that the design of the connections should consider

$$\tau \leq 0{,}4\,\rho_w f_{yd} \quad (4)$$

On the other hand, to cover the cases of static and cyclic loads, it was verified that it is possible to consider for shear stress variation at the connections

$$\Delta_\tau \leq 0{,}4\,\rho_w f_{yd}[e^{-0'187\,\log N}] \quad (5)$$

As far as the beam-jacket relative displacements is concerned, it was verified that, for the same load, beam with cyclic loading has greater displacement than the similar beam with only increasing monotonic loading.

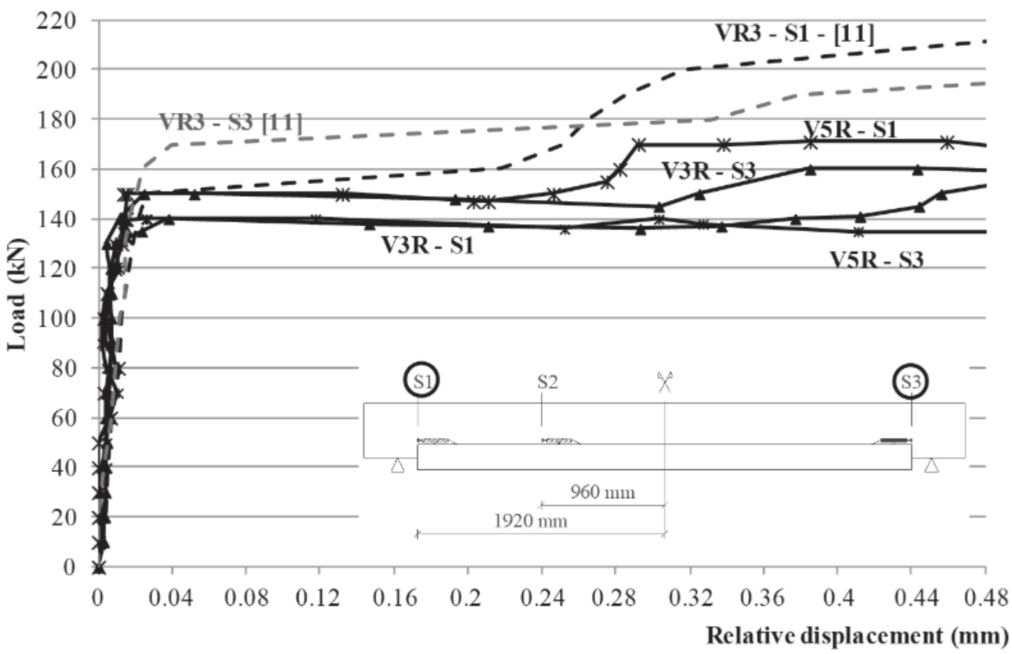

Figure 14
Load - relative displacement at the beam-jacket interface curves of VR3[11], V3R[1] and V5R[1]

5. References

[1] VAZ, A. P. R.. Comportamento de vigas reforçadas sob ação de carregamento cíclico. Tese de D. Sc., COPPE/ UFRJ, Rio de Janeiro, RJ, 2012.

[2] GOHNERT, M. Horizontal shear transfer across a roughened surface. Cement and Concrete Composites, v. 25, n.3, (Apr), pp. 379-385, 2003.

[3] CLIMACO, J. C. T. S.; REGAN, P. E. Evaluation of Bond strength between old and new concrete in structural repairs. Magazine of Concrete Research, v. 53, n.6, (Dec), pp. 377-390, 2001.

[4] BEUSHAUSEN, H., ALEXANDER, M. G. Bond strength development between concretes of different ages. Magazine of Concrete Research, v. 60, n.1, (Feb), pp. 65-74, 2008.

[5] MATTOCK, A. H. Shear transfer in concrete having reinforcement at an angle to the shear plane, ACI Special Publication SP-42, American Concrete Institute, pp. 17-42, 1974.

[6] LIEW, S. C., CHEONG, H. K.. Flexural behavior of jacketed RC beams. Concrete International, v. 13, n. 12, (Dec), pp. 43-47, 1991.

[7] PIANCASTELLI, E. M. Comportamento do reforço à flexão de vigas de concreto armado, solicitando a baixa idade e executando inclusive sob carga. Dissertação de M. Sc., UFMG, Belo Horizonte, Minas Gerais, 1997.

[8] CHEONG, H. K.; MacALEVEY, N. Experimental behavior of jacketed reinforced concrete beams. ASCE Journal of Structural Engineering, v. 126, n. 6, (Jun), pp. 692-699, 2000.

[9] BORJA, E. V. Estudo do comportamento de vigas de concreto armado reforçadas à flexão e esforço cortante. Dissertação de M. Sc., UFPE, Recife, Pernambuco, 2001.

[10] SANTOS, E. W. F. Reforço de vigas de concreto armado à flexão por encamisamento parcial. Dissertação de M. Sc., COPPE/UFRJ, Rio de Janeiro, RJ, 2006.

[11] SIMÕES, M. L. F. Reforço à flexão de vigas de concreto armado por encamisamento parcial. Dissertação de M. Sc., COPPE/UFRJ, Rio de Janeiro, RJ, 2007.

[12] SANTOS, P. M. D., JÚLIO, E. N. B. S., SILVA, V. D. Correlation between concrete-to-concrete bond strength and the roughness of the substrate surface, Construction and Building Materials, v. 21, n.8, (Aug), pp. 1688-1695, 2007.

[13] FÉDÉRATION INTERNATIONALE DU BÉTON, FIB Model Code for concrete structures 2010, 2013. Lausanne, Switzerland.

[14] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS, 2014. ABNT NBR-6118:2014: Projeto de Estruturas de Concreto – Procedimento. Rio de Janeiro, RJ, Brasil.

[15] XIE, E., VALENTE, M. I. B. Fatigue strength of shear connectors, Research Report, universidade do Minho, Guimarães, Portugal, 2011.