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Beam-slab connection in precast bridge decks with pockets filled out with high-performance concrete and shear key

Ligação viga-laje em tabuleiros de pontes pré-moldadas empregando nichos preenchidos com concreto de alto desempenho e chave de cisalhamento

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Abstract

This paper deals with the beam-slab shear connection used in composite bridge decks with full precast concrete deck. The connection is shaped by steel bar bent in hoop form, which is inserted in the pocket in slabs, associated with shear key. The connection is formed filling out the pocket with cast-in-place steel fiber reinforced concrete. This paper evaluates the shear strength of the connection using push-out tests. The substitution of plane and smooth connection by connection with shear key and fibers increases the strength of connection by up to 250%. Expressions to design the interface between precast beam and slab using the connection with shear key are proposed. Those expressions consider the influence of concrete strength, connector diameter and the presence of fibers. Finally, an example of the design of the beam-slab connection in a highway precast concrete bridge deck shows the application of the proposed expressions. © 2005 IBRACON. All rights reserved.

Keywords: shear connection; precast concrete; steel fiber reinforced concrete; bridge deck design.

Resumo

Neste trabalho são estudados conectores de cisalhamento viga-laje para tabuleiros de pontes formados por viga e laje pré-moldadas de concreto. Os conectores são formados por vergalhões de aço dobrados em forma de laço inseridos em nichos existentes na laje pré-moldada. A ligação é realizada preenchendo os nichos com concreto reforçado com fibras metálicas. O objetivo do trabalho é avaliar a resistência dessa ligação com chave de cisalhamento através de ensaios de cisalhamento direto. Foi observado aumento de 250% na resistência da ligação quando a ligação plana e lisa foi substituída por outra com chave de cisalhamento e fibras. São propostas expressões para avaliação da resistência da ligação com chave de cisalhamento que consideram a resistência do concreto, o diâmetro do conector e a presença de fibras metálicas na ligação. Ao final é mostrado um exemplo de aplicação dessas expressões no dimensionamento da interface de um tabuleiro de ponte pré-moldada rodoviária. © 2005 IBRACON. All rights reserved.

Palavras-chave: ligação de cisalhamento; concreto pré-moldado; concreto reforçado com fibras metálicas; projeto de pontes.

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1 Introduction

The connection of precast elements with cast-in place concrete is one of the most common applications in precast construction, generally referred to as composite beam. Such a connection is successfully used in the construction of bridges where the longitudinal beams are precast and the slab is cast-in-place. Some of the main advantages of employing composite beams are the time of construction, which is shorter than a solution with cast-in-place concrete only, and the reduction of formwork. The use of precast slab maximizes these advantages (Figure 1).

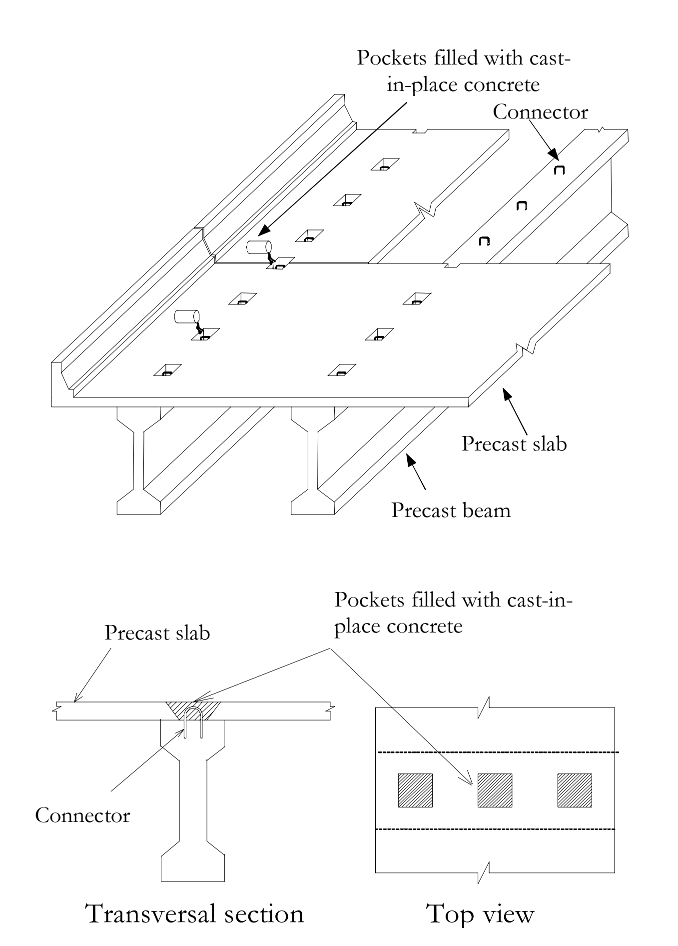


Figure 1 - Beam-slab connection in precast concrete bridge decks.

A connection frequently used between precast beam and slab consists of the combination of steel connectors with cast-in-place concrete. The steel connectors are bars bent in a hoop and placed in the precast beam during its molding. They are inserted in the shear pocket of the slab during the assembly of the structure, and the connection is later carried out by placing concrete in these pockets.

Although the combination of precast beam and slab has been frequently used in the construction of bridges, there are no reliable methods for design of the connection between both elements. The design can be made by analogy with the composite beams formed by steel beam and precast concrete slab, once in this structure type the stress in the interface is transferred in a discreet way by shear connectors [1 - 7]. However, the conclusions obtained from these tests cannot be directly applied to the combination of precast concrete beam and slab because for composite steel beams stud bolts were used as shear connectors. Another way to design the beam-slab connection is using experimental results that supply the strength of the shear connection for each specific case.

In this research, the connection showed in Figure 1 is modified by a shear key on the top face of the precast beam (Figure 2). Shear keys increase the shear strength of the interface because they provide an additional strength due to shear strength of the concrete in addition to the strength provided by transverse steel and friction in the contact surface [8, 9]. The use of high-strength concrete with steel fibers in the connection is also proposed. From push-out tests performed by authors, expressions to evaluate the strength of connection are proposed considering the influence of the strength of the cast-in-place concrete, the connector diameter and the addition of steel fibers. The fibers were just added in the connections with shear key, once in plane connections they do not present good performance because they are not anchored on both sides of the shear plane.

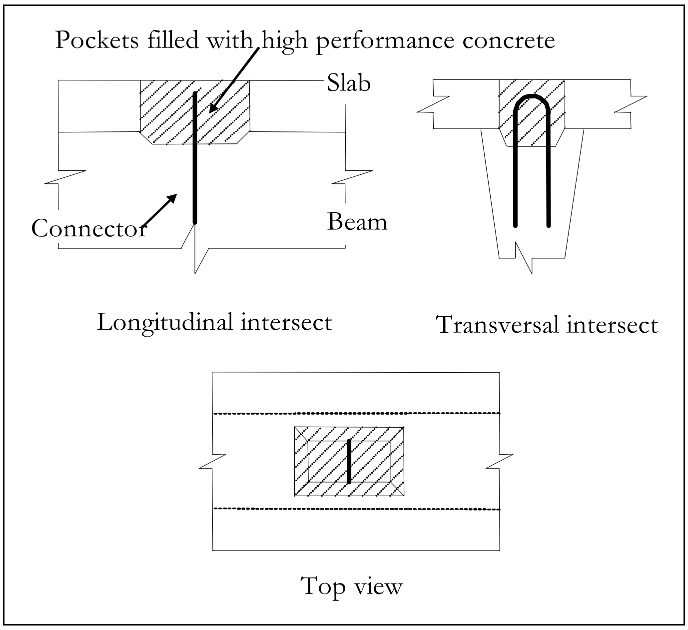


Figure 2 - Modified beam-slab connection due to shear key.

At the end of the paper, an example of the design of the beam-slab connection in a highway precast concrete bridge deck shows the application of the proposed expressions.

2 Experimental program

2.1 Description of tests

Twenty-five push-out tests were carried out, six with plane contact surface and nineteen with shear key. The variables analyzed in the tests were the shape of the contact surface, the compressive strength of concrete cast in the pockets, the diameter of the connector and the volume of fibers added to the connection. Table 1 shows the main characteristics of the specimens.

Table 1 – Details of push-out tests.

Mixture[(a)]	ϕ_s (mm)[(b)]	V_f (%)[(c)]	Specimen[(d)]
Mixture 1	10	-	PL-M1-10-0
	12.5	-	PL-M1-12.5-0
	8	-	PR-M1-8-0
	10	-	PR-M1-10-0
	0	-	C-M1-0-0
	8	-	C-M1-8-0
	10	-	C-M1-10-0
	12.5	-	C-M1-12.5-0
	12.5	1.50	C-M1-12.5-1.50
Mixture 2	8	-	C-M2-8-0
	8	0.75	C-M2-8-0.75
	8	1.50	C-M2-8-1.50
	10	-	C-M2-10-0
	10	0.75	C-M2-10-0.75
	10	1.50	C-M2-10-1.50
	12.5	-	C-M2-12.5-0
	12.5	0.75	C-M2-12.5-0.75
	12.5	0.75	C-M2-12.5-0.75-b
	12.5	0.75	C-M2-12.5-0.75-c
	12.5	1.50	C-M2-12.5-1.50
Mixture 3	12.5	-	PL-M3-12.5-0
	8	-	PR-M3-8-0
	8	-	C-M3-8-0
	10	-	C-M3-10-0
	12.5	1.50	C-M3-12.5-1.50

(a) The compressive strength of concrete used in connection ranged from 50 MPa to 100 MPa; (b) diameter of connector; (c) The steel fibers used were DRAMIX RL-45/30 BN with hooked ends; (d) In the nomenclature of the specimens, the first letter is the type of surface (smooth plane, rough plane or with shear key), the two following letters are the mixture used in connection, the following number is the diameter of the connector and the last number is the volume of fibers added to the connection.

The specimen used to apply only shear stresses to the connection is composed of three precast pieces: a central piece simulating the precast beam and two lateral pieces simulating the precast slab. The connector consisted of a steel bar bent in a hoop shape. The connection between the central piece and the lateral pieces was made of cast-in-place concrete in the existent pockets of the lateral pieces. Figure 3 shows the dimensions of the specimens used in these tests. These dimensions were chosen taking into account both the recommendations of BS 5400 [10] and the internal dimensions of the equipment available for the tests. This type of specimen is representative of the real situation, once the dimensions of the connection studied are similar to the dimensions of the connections usually found in precast concrete bridge decks.

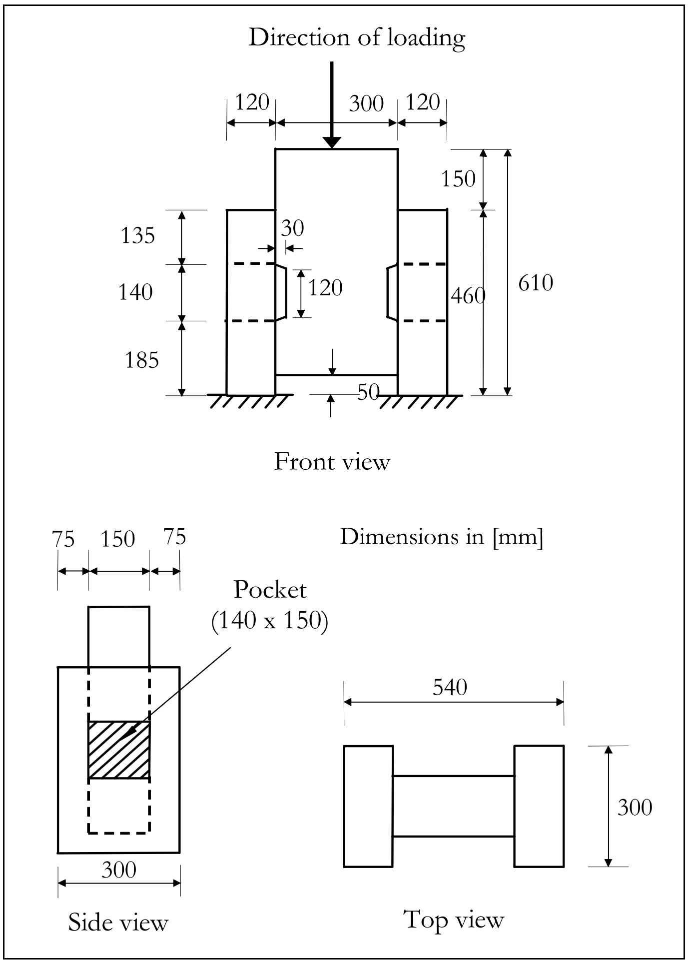
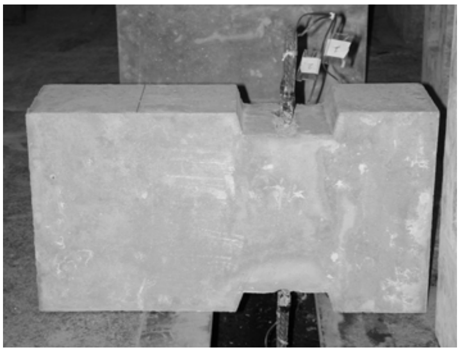
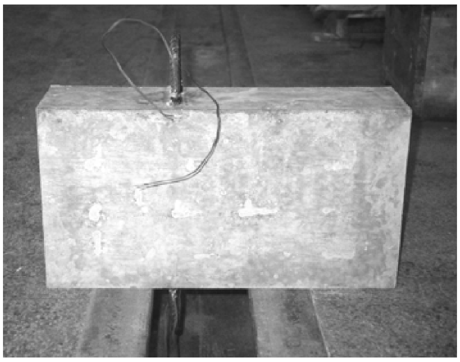


Figure 3 - Dimensions of the specimen used in push-out tests.

The specimens were made in two stages. In the first one, the lateral pieces with the pockets and the central piece with the connector were cast (Figure 4). After two days, one of the lateral pieces was positioned on the central piece and the connection was made with cast-in-place concrete. A thin layer of grease had been previously added to the surface of the central piece to avoid adherence between the precast pieces. After positioning the lateral piece, the pocket was caulked to prevent the slurry from escaping outside the area of connection, which could alter the results of connection strength. Soon after, the contact surface between the precast concrete and cast-in-place concrete was cleaned with compressed air to remove dirt and free particles. The pocket was wetted, avoiding free water that would cause a reduction of the cast-in-place concrete strength. The connection between precast pieces was finally carried out with concrete cast in the pocket (Figure 5). The cast-in-place concrete was cured for 24 hours using wet foam. After that period, the specimen was turned over, and the second connection was made in another lateral piece following the same procedure.



(a) Central piece with shear key



(b) Central piece with plane surface

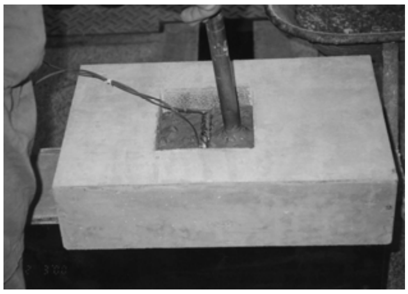
Figure 4 - Details of central pieces of the push-out specimens.

In three specimens, the surface of connection was plane and smooth, that is, there was no treatment on the contact surface of the central piece to make it rough, while in other three specimens the surface of connection was plane and rough. In these specimens, the roughness was made by gluing small wooden strips with 5 mm thickness by 20 mm width, spaced by 20 mm (Figure 6). These dimensions were chose in order to meet the minimum roughness recommended by NBR 9062 [11] of 5 mm depth every 30 mm.

The C-M1-0-0 specimen was made without a connector in the connection. The aim of this test was to quantify the contribution of the concrete to the connection strength. An adhesive was applied to the surface of the precast piece prior to casting the concrete in the pocket to guarantee adherence between the shear-key and the precast central piece. This procedure should guarantee that the failure would occur in the shear-key and not by the loss of adherence between the concrete cast in the pocket and the precast concrete.

The tests were carried out with deformation control seven days after the second connection was cast, using a universal test machine (Figure 7). The rate of load increment before the peak strength of connection varied from 0.001 mm/s to 0.006 mm/s. The smallest rate was used in specimen without fibers in the connection, while the largest rate was used in specimens with fibers. After the connection failure was well characterized, the rate of increment was gradually increased until the end of the test.

(a) Lateral piece with shear pocket



(b) Molding of the connection between central piece and lateral piece

Figure 5 – Detail of lateral piece and mounding of the connection.

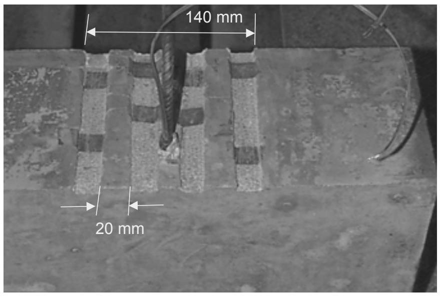


Figure 6 - Specimen with plane and rough surface (5 mm x 20 mm).

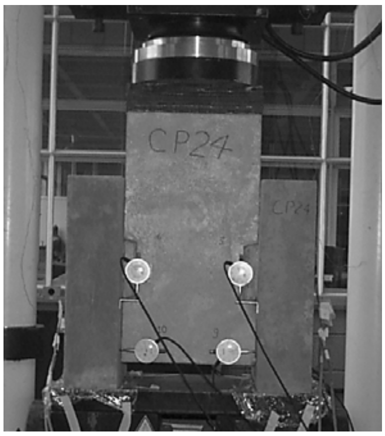


Figure 7 - Testing the specimen.

Table 2 - Mixture proportion per cubic meter of concrete without fiber.

Mixture	Cement (kg)	Silica Fume (kg)	Sand (kg)	Coarse aggregate (kg)	W/C factor	High-range water-reducing (%)
1	380	38	631	1132	0.56	1.50[a]
2	460	46	616	1104	0.42	1.80[a]
3	640	64	563	1011	0.32	2.20[a]
4	345	-	759	1173	0.55	0.37[a]

[a] *Percentage by weight of cement.*

Table 3 - Compressive and tensile strength of concrete with and without fibers.

Mixture	Steel fiber by volume (%)	Compressive strength – f_{cm} (MPa)	Splitting tensile strength – $f_{ct,sp}$ (MPa)	Flexural tensile strength – $f_{ct,fl}$ (MPa)
1	0	45.0	3.2	3.96
	0.75	52.2	5.1	4.04
	1.50	48.9	6.0	4.01
2	0	73.3	3.5	5.83
	0.75	73.1	5.0	5.99
	1.50	73.1	8.1	7.80
3	0	93.7	4.7	6.80
	0.75	99.3	6.2	6.93
	1.50	101.5	8.9	8.59

2.2 Characteristics of materials employed in the tests

Three concrete mixtures were used to make the connection between the precast elements, with a compressive strength ranging from 50 MPa to 100 MPa. The mix proportions per cubic meter of concrete are given in Table 2. In this table, mixtures 1, 2 and 3 were used to cast the connections and mixture 4 was used to cast the precast elements. By using high-early-strength Portland cement, it was possible to carry out a test seven days after casting the second connection. Silica fume was added to the concrete of connection using a proportion of 10% of the cement weight. River sand and crushed basalt stone with 19 mm in maximum size as coarse aggregates were used. A high-range water-reducing admixture was also added to improve the workability of the mixture. The admixture amount was defined to obtain a concrete of connection with good workability that allowed easy execution of the connection of precast elements. Therefore, the amount of high-range water-reducing admixture shown in the Table 2 was increased due to addition of fibers to the mixture.

The steel fibers employed were DRAMIX RL-45/30 BN with hooked ends. They were 30 mm long with a diameter of 0.62 mm, resulting in an aspect ratio of 48. The minimum tensile strength of these fibers was 1250 MPa. Two volumes of fibers were used, i.e., 0.75% and 1.50%, which corresponded to 60 kg and 120 kg of fibers per cubic meter of concrete, respectively. The fibers were added at the end of the mixture process of all other components; subsequently mixing was continued for one more minute.

The addition of fibers to the concrete did not alter its compressive strength, but it improved its tensile splitting strength and flexural strength obtained from third point load tests (Table 3). The fibers increase the ductility and this increase is more significant in high strength concrete. Further information about the influence of fibers on the behavior of concrete can be found in references [12] and [13].

The reinforcement steel employed in the precast elements and the connectors had well-defined yield strength. The average yield strength of 553 MPa and an average modulus of elasticity of 210 GPa had been determined by tension tests.

2.3 Results of push-out tests

Typical curves of the applied load versus the average slip between precast elements are shown in Figure 8 for specimens with shear key, smooth plane surface and rough plane surface. From this figure, it is observed that shear key increased by 250% the strength of the connection when compared with the plane and smooth surface. The roughness in the contact surface of the connection increases by 165% the strength of the connection and increases the dissipated energy by the peak load 22 times when compared with the specimens with smooth surface.

Table 4 shows the compressive strength of the concrete and the normal stress of the shear-off plane using the shear-friction model. This stress is obtained from the product of the rate of the transverse steel to shear-off plane (ρ) by the yielding strength of the steel (f_y). In this model, a rough interface is idealized by several small teeth without friction. When a horizontal load is applied, one portion tends to slide from the other portion. However, because of the small teeth, the portions tend to separate and the transversal reinforcement is tensioned, producing a normal stress at the interface. The product of that normal stress by an apparent friction coefficient provided the shear strength of the interface [14]. The table also shows the ultimate shear stress of the connection (τ_u), obtained from the strength of the connection divided by the area of the pocket (140 mm x 150 mm).

Table 4 - Concrete strength and ultimate shear at connection.

Specimen	Compressive strength f_{cm} (MPa)		ρf_y (MPa)	τ_u (MPa)
	Precast concrete	Cast in place concrete		
PL-M1-10-0	52.30	54.75	4.13	3.41 [a]
PL-M1-12.5-0	58.50	75.45	6.46	5.21 [a]
PR-M1-8-0	55.30	53.75	2.65	9.17
PR-M1-10-0	52.30	54.75	4.13	9.07
C-M1-0-0	53.58	55.10	0	9.34
C-M1-8-0	47.40	54.00	2.65	10.99
C-M1-10-0	47.40	54.00	4.13	11.91
C-M1-12.5-0	51.06	48.01	6.46	12.38
C-M1-12.5-1.50	51.60	55.75	6.46	17.06
C-M2-8-0	73.49	83.80	2.65	13.63
C-M2-8-0.75	84.61	88.60	2.65	15.24
C-M2-8-1.50	73.08	80.00	2.65	16.44
C-M2-10-0	67.63	72.81	4.13	15.07
C-M2-10-0.75	66.79	71.87	4.13	17.29
C-M2-10-1.50	66.79	72.07	4.13	18.28
C-M2-12.5-0	73.49	83.80	6.46	16.64
C-M2-12.5-0.75	53.58	80.92	6.46	22.00
C-M2-12.5-0.75-b	53.58	80.92	6.46	21.59
C-M2-12.5-0.75-c	84.61	88.60	6.46	24.74
C-M2-12.5-1.50	67.63	71.35	6.46	21.88
PL-M3-12.5-0	51.50	97.60	6.46	6.17 [(a)]
PR-M3-8-0	55.30	91.20	2.65	10.26
C-M3-8-0	51.06	96.76	2.65	14.76
C-M3-10-0	51.06	96.76	4.13	16.46
C-M3-12.5-1.50	51.60	96.35	6.46	20.08

(a) The ultimate load in the plane and smooth surface was taken when the adhesion between the precast concrete and the concrete cast in pocket was failed.

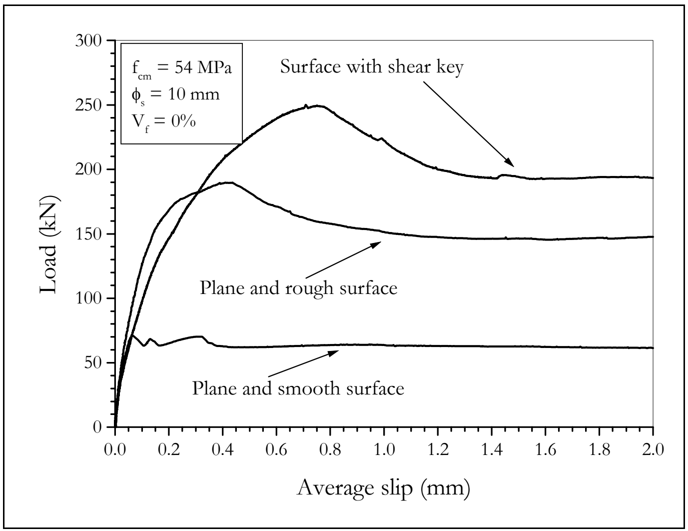


Figure 8 - Load-slip curves of the connection with 10 mm connector.

3 Expressions for evaluation the strength of beam-slab connection

Expressions for evaluation the strength of the beam-slab connection with shear key were obtained from the results of the push-out testes. The expressions were obtained from linear regressions correlating the ultimate shear stress, divided by the square root of the compressive strength of the concrete cast in the pocket, with the normal stress to the shear-off plane (ρf_y). Then, expressions similar to Bakhoum [15] were obtained based on shear-friction model. The following expression was obtained for the connections without fibers:

$$\tau_u = 1.270\sqrt{f_{cm}} + 0.798\rho f_y \leq 1.8\sqrt{f_{cm}} \quad (1)$$

where f_{cm} is the average compressive strength of the concrete cast in the pockets. For the connections with fibers, the following expression was obtained:

$$\tau_u = 1.388\sqrt{f_{cm}} + 1.415\rho f_y \leq 2.6\sqrt{f_{cm}} \quad (2)$$

where f_{cm} is the average compressive strength of the concrete cast in the pockets. The expression (2) is valid only for fiber volumes of up to 1.5%. In both expressions, the ultimate shear stress results in MPa. Other information on the procedure for obtaining those expressions can be found in reference [16].

Expressions (1) and (2) were applied to evaluate the strength of the specimens tested. The values of ratio of the strength obtained from the expression (1) and the experimental results of the specimens without fibers were calculated. The average of these values is 1.000 with standard deviation of 0.038. For specimens with fibers, the values of ratio of the strength obtained from the expression (2) and the experimental results have an average of 1.018 with standard deviation of 0.081.

Expressions (1) and (2) are function of the average strength of the materials, providing the average strength of the connection. These expressions need to be modified to take into account the safety of the connection for application in design. One of the methodologies to take the safety in structures is the semiprobabilistics methods. These methods use concepts of probability with deterministics values. The loads, with probabilistics or deterministics values, are magnified for partial γ factors while the strengths, with probabilistics values, are also lessened by partial factors [17].

The first modification proposed in those expressions is the substitution of the average strengths of the materials by its characteristic values. The design strength of the connection, for the ultimate limit states, is obtained lessening the characteristic strengths of the materials for the partial factors recommended by NBR 8681, that is, $\gamma_c = 1.4$ e $\gamma_s = 1.15$ [17]. Two other factors, ϕ and γ_{fad}, that take in account the probability of failure of the expressions (1) and (2) and the reduction of the connection strength due the fatigue of the material are introduced, respectively.

The factor ϕ takes in account the probability of failure of the empirical expressions, that is, the probability of the strength of the connection is less than the value calculated by those expressions. To obtain the value of that factor the concept of reliability index was used (β). This index represents the safety margin of an event to have success, that is, it indicates the distance of the structures with specific load and strength configuration to the ultimate state. The reliability index is calculated from the failure probability of the event, in this case defined by the strength of the connection available for the empirical expressions is greater than experimental strength. There are not enough experimental results to define a probability distribution function, than it was admitted that the strength of the connection follows a normal distribution.

If it is desired that the strength of the connection evaluated by the empirical expressions has a probability of 95% to be the same or less than the value obtained from tests (p_f = 5%), the factor ϕ is calculated by:

$$\phi = 1 - \beta.\delta \qquad (3)$$

where β = 1.65 (p_f = 5%) and $\delta = S/M$ are the variation coefficient. *S* and *M* are the standard deviation and the average, respectively, of the relationship between the strength calculated by the empirical expressions and the values obtained from the tests. Therefore ϕ = 0.94 is obtained for the connection without fibers and ϕ = 0.87 for the connection with fibers. Those values are greater than some values recommended in other papers (ϕ = 0.83) for determining the characteristic strength of the concrete elements from tests, and it can be obtained from the expression (3) using δ = 0.1 and β = 1.7 [18]. In this paper, it was opted to use the value of ϕ = 0.83 for connections with fibers as well for connections without fibers. Figure 9 shows the comparison between some values obtained from tests and values obtained from the expression (2) and from this expression multiplied by the factor ϕ. It was observed that the values estimated by the empirical expression multiplied by factor ϕ are always less than the experimental results (as was expected).

The structures of bridges are subjected to thousands cycles of load and unload during its service life, then it is fundamental to take account the fatigue to design these structures. There are no experimental results that evaluate the strength reduction of this beam-slab connection due to the fatigue. FIP [19] published some recommendations for the design of connection between concretes cast in different ages. The reduction of the static strength proportioned by concrete for the half is suggested when the connection is subjected to repeated loads. This recommendation is valid for connections based on shear-friction model that was used in deduction of the empirical expressions. Therefore, it was opted to use γ_{fad} = 2 in connections without fibers.

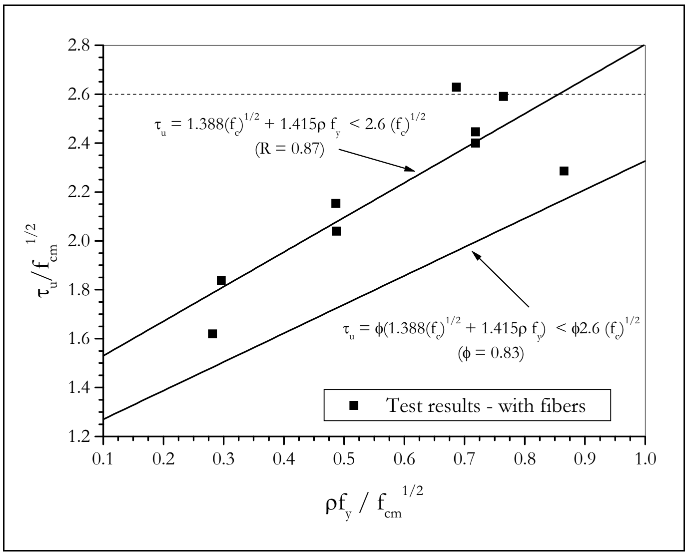


Figure 9 - Strength of the connection with shear key and fibers from empirical expressions.

The addition of fibers reduces stiffness degradation and increases fatigue strength of the connections. The fatigue strength is here defined as the greatest load that can be applied to the connection so that it does not fail after a million cycles of load and unload. This strength is usually defined as a percentage of the monotonic strength of the connection. Most papers in the technical literature deal with the fatigue of the fiber reinforced concrete submitted to the bending loads, some of them when subjected to compression loads and few when submitted to shear loads. In general, fibers do not increase the fatigue strength of the concrete when submitted to compression loads [20]. When subjected to bending or shear loads, fibers allow the concrete to reach fatigue strength of up to 90% of its monotonic strength [21, 22]. In reference [23], it is verified from bending tests that concretes with compressive strength of 120 MPa support a million cycles with fatigue strength up to 73% of its monotonic strength. This conclusion was obtained from tests with fibers of aspect ratio greater than that used in this paper. However, other papers showed that the aspect ratio of the fibers has little influence on the fatigue strength [24]. Therefore, γ_{fad} = 1.4 was chosen for connections with fibers ($\gamma_{fad} \approx 1/0.73$).

The design strength of connection with shear key can be evaluated by the following expression:

$$F_{lig,d} = A_n \tau_{u,d} \qquad (4)$$

where A_n is the area of the pocket. The ultimate shear stress in connections with shear key but without fibers is:

$$\tau_{u,d} = \phi\left(\frac{1.270}{\gamma_{fad}}\sqrt{\frac{f_{ck}}{\gamma_c}} + 0.798\rho\frac{f_{yk}}{\gamma_s}\right) \leq 1.8\frac{\phi}{\gamma_{fad}}\sqrt{\frac{f_{ck}}{\gamma_c}} \quad (5)$$

The ultimate shear stress in connections with shear keys and fibers up to volume of 1.5% is:

$$\tau_{u,d} = \phi\left(\frac{1.388}{\gamma_{fad}}\sqrt{\frac{f_{ck}}{\gamma_c}} + 1.415\rho\frac{f_{yk}}{\gamma_s}\right) \leq 2.6\frac{\phi}{\gamma_{fad}}\sqrt{\frac{f_{ck}}{\gamma_c}} \quad (6)$$

There are not enough test results to obtain an empirical expression for plane and rough connections. In this case, the expressions developed by Tassios and Vintzeleou for rough interfaces between precast concrete elements are recommended [25]. The authors proposed expressions based on shear-friction model and in dowel action of the bars transverse to the interface. More details on those expressions and their application to the plane and rough connection used in this paper can be found in reference [26].

Table 5 shows the strength of some plane and rough connections obtained by those expressions varying the diameter of the connector. This table also shows the values obtained by FIP [19] for intentionally rough surface. The compressive strength of the concrete cast in the pocket is constant at 65 MPa and the pocket dimension is 180 mm x 180 mm. Figure 10 shows the strength of the connection in function of the transverse steel ratio using expressions (5) and (6).

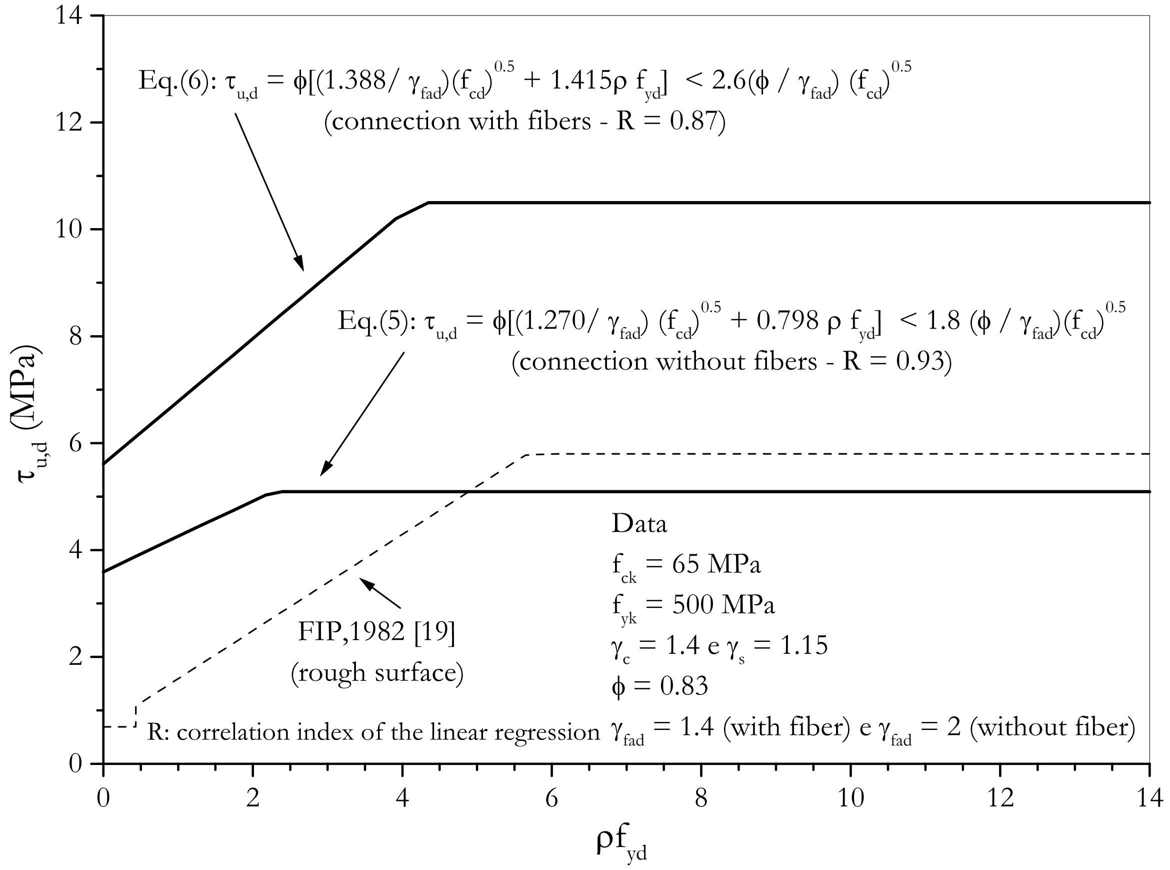


Figure 10 - Design strength of the connection with shear key in function of transverse reinforcement ratio.

4 Design example

4.1 Description

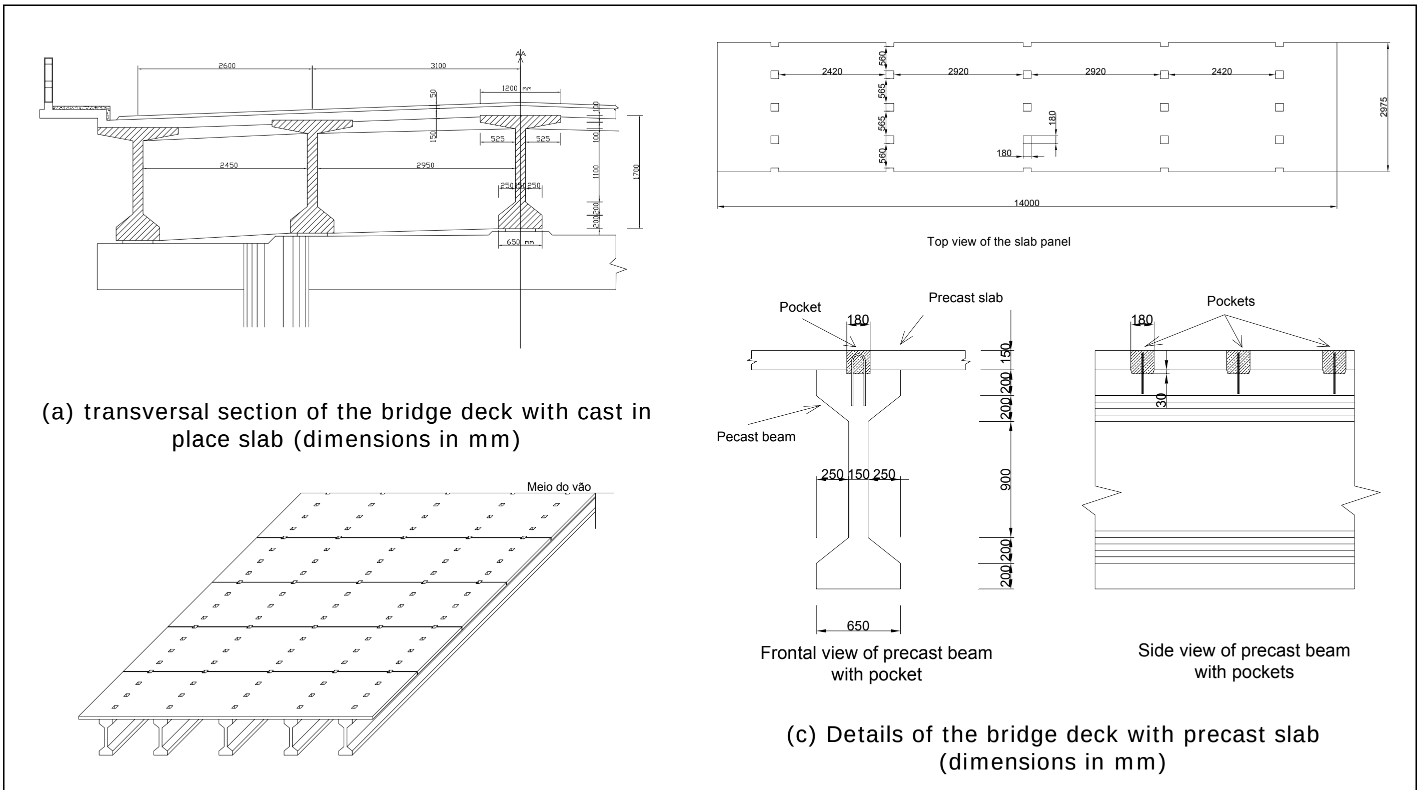
This section shows the design example of composite beam made with the association of precast beam and slab. It is the typical highway bridge structure of class 45 defined in Brazilian Standard [27]. It is formed by seven simply supported spans with 29 m each and has a total width of 14 m. The superstructure is made by five precast prestressed concrete girders (f_{ck} = 30 MPa), four transverse crossbeams (two on support and two on span) and cast-in-place concrete slab. Each span was designed separately, that is, the continuity of the slab was not admitted. This structure was originally designed with cast-in-place concrete slab, but to illustrate the design it was adopted the same dimensions of cast-in-place slab to the precast slab. Figure 11 shows some details and the geometry of this bridge.

The objective of this example is to show the application of the empirical expressions obtained in the previous sections. Thus, it is necessary to know the stresses in the interface originating from the external loads, which are functions of the construction process.

Table 5 - Strength of beam-slab connection (pocket with 180 mm x 180 mm and f_{ck} = 65 MPa).

Connection type	Volume of fibers (%)	Diameter of connector (mm)([b])	Design strength of connection - $F_{lig,d}$ (kN)	FIP([c])(kN)
Shear Key	0.75	1 ϕ 12.5 ρ = 0.00759	307.0	118.7
		1 ϕ 10 ρ = 0.00485	261.9	83.9
		1 ϕ 8 ρ = 0.00310	233.0	61.8
Shear Key	0	1 ϕ 12.5 ρ = 0.00759	164.9	118.7
		1 ϕ 10 ρ = 0.00485	161.6	83.9
		1 ϕ 8 ρ = 0.00310	145.3	61.8
Plane and rough([a])	0	1 ϕ 12.5	100.8	109.6
		1 ϕ 10	80.7	74.9
		1 ϕ 8	79.9	52.7

(a) In the plane and rough surface, it was used the smallest concrete strength on connection, in this case the precast concrete strength admitted of 30 MPa. It was used, also, γ_{fad} = 2; (b) steel with f_{yk} = 500 MPa; (c) strength obtained from the FIP [17] for intentionally rough surface (category 2) submitted to fatigue: $\tau_{ud} = 0.2f_{ctd} + 0.9\rho f_{yd} \le 0.125f_{cd}$



(a) transversal section of the bridge deck with cast in place slab (dimensions in mm)

(b) perspective

(c) Details of the bridge deck with precast slab (dimensions in mm)

Figure 11 - Full precast concrete bridge deck.

After erection of the girders and the molding of the transverse crossbeams, the concrete panels are assembled on the girders and the high performance concrete cast in the pockets carries out the connection. In that construction process, the transverse crossbeams are not connected to the slab. The second prestress phase in the girders is made after the concrete of the pocket obtain the necessary strength to assure the shear transfer for the interface. In accord with that sequence, the loads that act in the composed section are the second prestress phase of the girders, the self- weight of the non-structural elements (g_3) and the traffic load (q). Those actions should be transferred by the interface. In this example, only the self-weight of the non-structural elements and the traffic load were considered. The stress due the second prestress phase act in opposite direction decreasing the total stress in the interface. For simplicity, it was neglected.

4.2 Forces in the connections

The internal forces in the girders due to traffic load were obtained using the Brazilian Standard NBR-7188 [27]. Table 6 shows the internal forces in the middle section span of the extremity girder due the load g_1 (self-weight of the precast beam), g_2 (self-weight of the slab and the transverse crossbeams), g_3 (self-weight of the pavement and the non-structural elements) and traffic (q). Those internal forces were obtained from the cast-in-place concrete slab construction process. In full precast concrete deck construction process, the superior flange of the precast beam was reduced for best support of the slab. However, it did not alter significantly the internal forces due to self-weight load.

Table 6 - Bending moment and shearing-force in the midspan of the extremity girder.

Load	Bending moment (kN.m)	Shearing-force (kN)
g_1	1495	205.1
g_2	1606	220.3
g_3	510	70.0
q	3143	446.8

The shear stress in the interface between beam and slab were available using the following simplified expression:

$$\tau = \frac{V}{0.9bd} \quad (7)$$

where *V* is the shearing-force, *b* is the width of the interface and *d* is the distance from the extreme compression fiber to centroid of tensile force of the composite beam. That expression is function of the shearing-force in the section. Therefore, the shear stress on the interface follows a similar distribution of the shearing-force along the span. Table 7 shows the shear stress in the interface in each one of the design sections of the more loaded girder, that is, the extremity girder. The interface width is 180 mm. In construction process with precast concrete slab the shear stress in the interface is transmitted by discreet connections along the span, while in solution with cast-in-place concrete slab the shear stress is transmitted by completely superior face of the precast beam. Then, the width of the interface was defined equal to the pocket width that was admitted approximately equal to the web width of the precast beam. The distance *d* of the composite beam (1.73 m) was obtained from the distribution of the prestressed reinforcement in the middle span and it was admitted constant along the span.

After obtaining the shear stress in the interface, it was chosen the connection type and the distance between pockets in the interface. There are several ways to design the connection. Here, the distance and dimensions of the pockets was adopted and then the diameter of the connector, the compressive strength of the concrete and the connection type was defined that assure a connection strength greater than a shear stress in the interface. The initial choice of the distance and dimensions of the pockets was done with the intention of standardizing the dimensions of the precast slab that would facilitate its manufacture. A distance of 720 mm was adopted between pockets that result in, approximately, four pockets between each design section. Therefore, each slab panel could have, for example, width of 2.90 m. One of the pocket dimensions was adopted approximately equal to the web width of precast beam (180 mm). The other dimension was also adopted equal to 180 mm, once in the available tests results the two dimensions of the pocket were always close, do not have experimental evidence that the deduced empirical expressions can be directly applied to the pockets with other ratio between the sides different from one. Table 7 shows the ultimate load on each connection, between two adjacent design sections, as well as the maximum value of its variation in function of the action of the traffic load. The force applied in the connection was obtained multiplying the shear stress on the interface by the width of the interface and the distance between the pockets. Close to the support, between design sections 0 and 1, the shearing-force was reduced due to the arch action that appears close to support. Therefore, the shear stress used to estimate the force applied in the connection close to the support was that located in a design section distant from the support of the height of the precast beam, that is, 1.70 m (τ = 1674.5 kN/m^2).

4.3 Design of the connection

The design is carried out to attend the ultimate limit state with normal combination of actions (greatest monotonic force in the connection - F_{lig} - and the strength non reduced by the fatigue) as well as the limit state of fatigue with the greatest variation of the force applied in the connection (ΔF_{lig}) and the strength reduced by the fatigue.

Table 8 shows the shear force and the possibilities of connectors that can be used when the connection is plane and rough. Table 9 shows the diameters of the connectors when the connection is carried out with shear key, with or without fibers. Figure 12 illustrates the case of connection with shear key but without fibers. That figure shows the shear force and the design strength of the interface admitting the pockets distant of 540 mm. This detail aimed just to attend the shear force distribution on the interface along the span of the beam. In a real design, the standardization of the connection should also be taken into account.

In this design example, the use of the plane and rough connection requested the connectors with large diameter that make unfeasible the production of the connection. With pocket of 180 mm x 180 mm and connector of 16 mm diameter, the maximum possible distance between pockets was 540 mm. With connector of 16 mm diameter and pocket enlarged for 180 mm x 270 mm, the maximum possible distance was 675 mm. The design of the connection was defined by the limit state of fatigue and the reinforcement obtained from ultimate limit state was smaller.

Table 7 - Shear stress on interface of the extremity composite beam.

Section	5	4	3	2	1	0
V_{g3} (kN)	0	14.0	28.0	42.0	56.0	61.8
$V_{q,max}$ (kN)	158.0	205.8	258.8	316.6	379.3	407.5
$V_{q,min}$ (kN)	-158.0	-114.5	-76.0	-43.3	-14.7	0
τ (kN/m^2)	563.8	784.3	1023.3	1279.5	1553.2	1674.5
F_{lig} (kN)[(a)]	73.1	101.6	132.6	165.8	201.3	217.0
ΔF_{lig} (kN)[(b)]	146.1	148.1	154.8	166.4	182.2	188.4

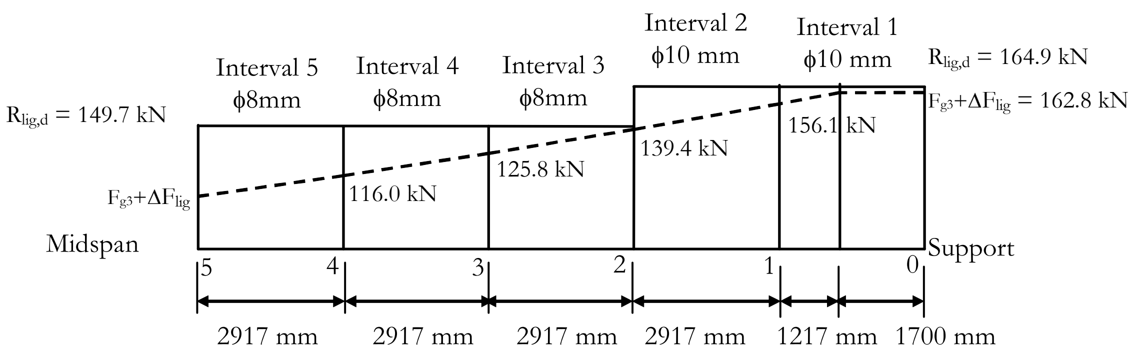
[(a)] *F_{lig}: maximum shear force in each connection obtained from $F_{lig} = \tau \times b \times e$, where e = 720 mm is the distance between the pockets and b = 180 mm is the interface width; (b) ΔF_{lig}: variation of the force on connection.*

Table 8 – Possibilities of connectors in the beam-slab connection with plane and rough surface.

Pocket dimension (mm)	Distance (mm)	Design shear force on the connection(kN)					Diameter of connector and design strength of the connection				
		Interval					Interval				
		1	2	3	4	5	1	2	3	4	5
Ultimate Limit State[(a)]											
180 x 180	720	303.8	281.8	232.1	185.8	142.4	Non compatible				
	540	227.8	211.4	174.1	139.3	106.8	ϕ16 mm (222.8 kN)	ϕ16 mm (222.8 kN)	ϕ12,5 mm (178.9 kN)	ϕ10 mm (148.5 kN)	ϕ8 mm (125.5 kN)
	360	151.9	140.9	116.1	92.9	71.2	ϕ12,5 mm (178.9 kN)	ϕ10 mm (148.5 kN)	ϕ8 mm (125.5 kN)	ϕ8 mm (125.5 kN)	ϕ8 mm (125.5 kN)
180 x 270	810	341.8	317.1	261.1	209.0	160.2	Non compatible				
	675	284.8	264.2	217.6	174.1	133.5	ϕ16 mm (279.4 kN)	ϕ16 mm (279.4 kN)	ϕ12,5 mm (228.1 kN)	ϕ10 mm (190.8 kN)	ϕ8 mm (163.2 kN)
	540	227.8	211.4	174.1	139.3	106.8	ϕ12,5 mm (228.1 kN)	ϕ12,5 mm (228.1 kN)	ϕ10 mm (190.8 kN)	ϕ8 mm (163.2 kN)	ϕ8 mm (163.2 kN)
Limit State of Fatigue[(b)]											
180 x 180	720	188.4	182.2	166.4	154.8	148.1	Non compatible				
	540	141.3	136.7	124.8	116.1	111.1	ϕ16 mm (146.0 kN)	ϕ16 mm (146.0 kN)	ϕ16 mm (146.0 kN)	ϕ16 mm (146.0 kN)	ϕ16 mm (146.0 kN)
	360	94.2	91.1	83.2	77.4	74.1	ϕ12,5 mm (109.2 kN)	ϕ12,5 mm (109.2 kN)	ϕ12,5 mm (109.2 kN)	ϕ10 mm (86.6 kN)	ϕ10 mm (86.6 kN)
180 x 270	810	212.0	205.0	187.2	174.2	166.6	Non compatible				
	675	176.6	170.8	156.0	145.1	138.8	ϕ16 mm (174.6 kN)	ϕ16 mm (174.6 kN)	ϕ16 mm (174.6 kN)	ϕ16 mm (174.6 kN)	ϕ16 mm (174.6 kN)
	540	141.3	136.7	124.8	116.1	111.1	ϕ16 mm (174.6 kN)	ϕ16 mm (174.6 kN)	ϕ16 mm (174.6 kN)	ϕ12,5 mm (134.1 kN)	ϕ12,5 mm (134.1 kN)

[(a)] *$F_{lig,d} = \gamma_f F_{lig}$, with γ_f = 1.4. To obtain the strength of the connection γ_{fad} = 1.0 , γ_c = 1.4 e γ_s = 1.15 was considered.*

[(b)] *$F_{lig,ser} = \Delta F_{lig}$. To obtain the strength of the connection γ_c = 1.4 , γ_s = 1.0 e γ_{fad} = 2.0 was considered. Only the concrete strength was affected by fatigue.*



Dimensions of pockets : 180 mm x 180 mm
Distance of pockets: 540 mm
Concrete strength of concrete cast in pocket: f_{ck} = 65 MPa
Volume of fibers: 0%

Obs: This detail aimed just to attend the shear force distribution on the interface along the span of the beam. In a real design, the standardization of the connection should also be taken into account

Figure 12 - Diagram of shear force and strength on the interface along the span - connection with shear key, without fibers and distance between pockets of 540 mm.

Table 9 - Diameter of connectors in beam-slab connection with shear key.

<table>
<tr><th rowspan="3">Pocket dimension (mm)</th><th rowspan="3">Distance (mm)</th><th colspan="5">Diameter of connector and design strength of the connection – Ultimate Limit State[(a)]</th><th colspan="5">Diameter of connector and design strength of the connection – Limit State of Fatigue[(b)]</th></tr>
<tr><th colspan="5">Interval</th><th colspan="5">Interval</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td colspan="12">Connection with shear key and fibers</td></tr>
<tr><td rowspan="3">180 x 180</td><td>720</td><td colspan="5">ϕ8 mm (305.7 kN)</td><td colspan="5">ϕ8 mm (240.7 kN)</td></tr>
<tr><td>540</td><td colspan="5">ϕ8 mm (305.7 kN)</td><td colspan="5">ϕ8 mm (240.7 kN)</td></tr>
<tr><td>360</td><td colspan="5">ϕ8 mm (305.7 kN)</td><td colspan="5">ϕ8 mm (240.7 kN)</td></tr>
<tr><td rowspan="3">180 x 270</td><td>810</td><td colspan="5">ϕ8 mm (432.8 kN)</td><td colspan="5">ϕ8 mm (331.5 kN)</td></tr>
<tr><td>675</td><td colspan="5">ϕ8 mm (432.8 kN)</td><td colspan="5">ϕ8 mm (331.5 kN)</td></tr>
<tr><td>540</td><td colspan="5">ϕ8 mm (432.8 kN)</td><td colspan="5">ϕ8 mm (331.5 kN)</td></tr>
<tr><td colspan="12">Connection with shear Key and without fibers</td></tr>
<tr><td rowspan="3">180 x 180</td><td>720</td><td>ϕ12.5 mm (303.4 kN)</td><td colspan="4">ϕ8 mm (261.7 kN)</td><td colspan="5">Non compatible</td></tr>
<tr><td>540</td><td colspan="5">ϕ8 mm (261.7 kN)</td><td colspan="5">ϕ8 mm (149.7 kN)</td></tr>
<tr><td>360</td><td colspan="5">ϕ8 mm (261.7 kN)</td><td colspan="5">ϕ8 mm (149.7 kN)</td></tr>
<tr><td rowspan="3">180 x 270</td><td>810</td><td colspan="5">ϕ8 mm (378.0 kN)</td><td>ϕ10 mm (226.5 kN)</td><td colspan="4">ϕ8 mm (207.8 kN)</td></tr>
<tr><td>675</td><td colspan="5">ϕ8 mm (378.0 kN)</td><td colspan="5">ϕ8 mm (207.8 kN)</td></tr>
<tr><td>540</td><td colspan="5">ϕ8 mm (378.0 kN)</td><td colspan="5">ϕ8 mm (207.8 kN)</td></tr>
</table>

[(a)] *To obtain the strength of the connection γ_{fad} = 1.0, γ_c = 1.4 e γ_s = 1.15 were considered in expressions (5) and (6).*

[(b)] *To obtain the strength of the connection γ_c = 1.4, γ_s = 1.0, γ_{fad} = 1.4 (connection with fibers) and γ_{fad} = 2.0 (connection without fibers) was considered.*

The change of plane and rough connection by other with shear key altered the detail of the connection. The connection without fibers resisted the shear stress on the interface in all situations, except for pocket with

180 mm x 180 mm and distance of 720 mm. The necessary connector diameter in the connection was 10 mm. The addition of fibers to the concrete cast in the pockets provided a connection able to resist all situations analyzed with a connector of only 8 mm diameter. In connections without fibers and pockets most spaced, the design is defined by limit state of fatigue. With the addition of fibers, the fatigue did not limit the design of the connection.

The use of the shear key in connection allowed a larger distance between the pockets in interface. In addition, it reduced the necessary diameter of the connector in the connection, what can facilitate the assembly process in the work. It is observed, also, that the pocket of 180 mm x 180 mm proved to be appropriate due to the addition of fibers to the concrete. This became needless the increase of the pocket size to reduction the diameter of the connector. It is just observed that if very large distance between pockets is used, there will be a force concentration in the connection, which can result in failure of the concrete of the precast beam. This failure can be verified considering the action of the self-weight g_3 and the traffic load (q) from the adaptation of truss model [28].

It is important to point out that the design presented takes into account the full variation of the force in the connection due to traffic load. According to Brazilian Standard NBR-6118: [29], this variation can be reduced for the factor ψ_1 that takes into account the applied load type and the type of structural element. For highway bridge girders, this code recommends for this factor the value 0.5. Carrying out the design of the connection with this factor is verified that the use of shear key also reduces the diameter of the connector. On the other hand, the addition of fibers does not alter the detail of the connection, that is, as the connection with shear key and fiber as the connection with shear key but without fiber attend all situations analyzed with a connector of only 8 mm diameter.

5 Conclusions

The results obtained in the paper show that the execution of the shear key on the connection increases significantly the strength of the connection when compared to the connections with plane and smooth surface. The assurance of minimum roughness on the surface of the precast beam also increases its strength when compared to the smooth surface. The main advantage of the connection with shear key over the plane and rough connection is the possibility of the use of steel fibers, which contribute to the strength and the dissipated energy until the failure of the connection.

The main quantitative conclusion obtained from the push-out tests is that the execution of shear key on the connection increased its strength y 250% when compared with the plane and smooth connection. The assurance of minimum roughness on the contact surface increased the strength of connection by 165%, also compared with smooth surface. Those values were obtained from the tests with diameter connector of 10 mm and concrete cast in the pocket with 50 MPa compressive strength.

Empirical expressions were proposed from the tests to evaluate the strength of the connection with shear key. Those expressions were modified to appreciate the safety in the structure and then they were applied to design the interface of a highway composite bridge with full precast concrete deck. This analysis showed that it is possible to use the precast concrete slab solution from the appropriate choice of the connection on the interface. In the design example, the use of shear key with fibers was more appropriate for allowing a larger distance between the pockets combined with the use of connectors with smaller diameter in the connection. Comparing the alternative connection to the usual connection with plane and rough surface, with pockets of 180 mm x 180 mm and distance of 540 mm, it is observed that in the interval of larger shear force (interval 1 - close to the support) the reinforcement is reduced from 1ϕ 16 mm to 1ϕ 8 mm. In addition, the presence of shear key and the addition of fibers to the concrete increased the pocket distance by up to 720 mm.

The use of expressions proposed in the paper is an advance, once there are no appropriate criteria for the design of this structure type. Therefore, the precast beam and slab solution can be applied appropriately in situations where the velocity of the construction is a decisive factor, as, for example, in roads of heavy traffic.

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