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Geometric optimization associated with the use of high-strength concrete in viaducts

Otimização geométrica associada à utilização de concreto de alta resistência em tabuleiros de viadutos de múltiplas longarinas pós-tensionadas


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Abstract

In Brazil, there is a lack of infrastructure investment in roads, given that they facilitate the transportation of more than half the country's cargo volume. One of the main variables in road infrastructure is overpass construction and maintenance. Concrete overpasses with post-tensioned I-section beams have been extensively used in Brazil. This study discusses the economic aspects of the use of high-strength concrete (HSC) associated with the geometrical optimization of decks of post-tensioned multi-girder overpasses. Twelve overpass decks were dimensioned and divided into two groups. In Group A the characteristic concrete strength varied, but not the geometrical characteristics of the deck. In Group B, the characteristic strength and the geometrical characteristics of the deck varied. These were the configurations that presented the best results for each group of characteristic concrete strength. It was determined that the use of HSC significantly decreases the reinforcement ratio–especially shear reinforcement. In addition, although the HSC has a higher cost per m3, it is still considered a viable option owing to the reduction in the reinforcement ratio. Lastly, in addition to providing the benefits that are widely commented on in literature, using HSC can also provide more economical overpass structures compared to conventional concrete.

Keywords: bridges, concrete structures, viaducts, structures and design.

Resumo

O Brasil carece de investimentos em infraestrutura rodoviária, uma vez que este é responsável por mais da metade do volume de cargas transportado no território nacional. Quando se aborda o tema de infraestrutura rodoviária, uma das principais variáveis é, sem dúvida, a construção e a manutenção de viadutos. Viadutos em concreto de longarinas protendidas com seção I tem sido amplamente utilizadas no território nacional. Este trabalho tem como objetivo discutir os aspectos econômicos da utilização de concreto de alta resistência (CAR) associada à otimização geométrica de tabuleiros de viadutos de múltiplas longarinas pós-tensionadas. Para tanto, foram dimensionados doze tabuleiros de viaduto dividindo dois grupos. No Grupo A, a resistência característica do concreto variava e as características geométricas do tabuleiro não. Já no Grupo B a resistência característica variava, assim como as características geométricas do tabuleiro, haja vista que estas eram as configurações que apresentavam os melhores resultados para cada grupo de resistência característica do concreto. Sendo assim foi possível identificar que a utilização CAR diminui significativamente a taxa de armaduras, principalmente as armaduras de cisalhamento. Também observou-se que, apesar do CAR possuir um custo mais elevado por m³, ele ainda é considerado uma opção viável devido a redução da taxa de armaduras proporcionada. E por fim, a sua utilização além de proporcionar os benefícios amplamente comentados na literatura, ainda pode proporcionar estruturas de viadutos com custo de construção mais econômicos que o concreto convencional.

Palavras-chave: viadutos, pontes, CAR, otimização.

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1. Introduction

According to Eller [9], in Brazil more than half of the total cargo volume is transported by road.
The unsatisfactory conditions of this system have caused the Brazilian products to have costly shipping and vehicle maintenance, thus reducing their competitiveness. Therefore, owing to the constant need for investments in road conservation, the public resources are not sufficient to maintain the quality of the system [9].
Investments for recovery and duplication are being made, as well as concessions, but they are still not sufficient to satisfy the country's deficiency in road infrastructure. Martins, Soares, and Cammarata [11] estimate that it would be necessary to build 21,000 km of road network to obtain a significant reduction in the transportation time, the number of accidents, and the shipping costs in Brazil.
Overpasses and bridges, commonly named "special construction works," are one of the most important structures with regard to road infrastructure, given that they are costly and demand detailed planning.
Bridges built with multiple prestressed I-sections and precast girders are becoming common in Brazil, given that they are ideal for short-to-medium overpasses (20 to 60 m) owing to their moderate weight, structural efficiency, ease of manufacture, fast construction, and ease of maintenance [1].
With regard to the characteristic strength of concrete, special attention is drawn to the use of high-strength concrete (HSC). Currently, it is known that HSC has a characteristic compressive strength equal to or greater than 55 MPa – according to NBR 8953:2015 [7]. In the last few years, its use has become widespread owing to the technological breakthroughs in the executive processes, and the fact that technological and project control have required the concrete to have increasingly higher structural performance.
In addition to the improvement of mechanical properties, according to Mehta [13], one of the main characteristics of HSC is its durability as compared to conventional concrete – it is stronger when subjected to the action of aggressive agents. This improvement is caused by the low water–cement ratio that leads to a decrease in porosity and consequently the permeability, increasing the service life of the structure and reducing maintenance costs.
According to Aïtcin [2], after 1960, HSC has begun to be used in large quantities in noticeable structures. Until the 1990s, an improvement was made by including superplasticizer additives, followed by the use of silica.
Peinado et al. [14] commented that, in addition to increasing the concrete strength, and therefore, decreasing the use of cement, the use of active silica is recommended due to technical and environmental issues, given that it is a by-product of the production of metal alloys.
Rebmann [15] highlights that cement production is a highly energetic process, and the production of 1,000 kg of concrete is responsible for the emission of approximately 830 kg of CO_2 into the atmosphere, depending on the clinker content used.
According to the annual report by the National Union of the Cement Industry (Sindicato Nacional da Indústria do Cimento – SNIC), in 2013, 71 million tons of cement were produced in Brazil. Considering the average CO_2 emissions presented by Rebmann (830 kg/ton of cement), the cement production accounted for the emission of approximately 59 million tons of CO_2 into the atmosphere this year.
According to studies performed by the Federal Government, in the same year, 460 million tons of CO_2 were emitted in Brazil; therefore, cement production is responsible for approximately 12.8% of the CO_2 emissions in Brazil.
Hence, considering the environmental issues, there is an evident need for research in relation to each class of strength that is more appropriate for the dimensioning of each structure in order to reduce the use of cement in concrete and the use of steel per structure/structural element [14].
According to Dal Morin [8], in the mid-1990s the HSC combined with post-stressing was widely applied to overpasses with medium and long spans with the purpose of limiting the deflection, decreasing its own weight, and reducing the creep.
In bridge decks, cracks have been the subject of studies over the years. However, there are still many questions on how to effectively minimize this problem given that its occurrence leads to the corrosion of the reinforcement [12] and, according to Aïtcin [2], the penetration of chloride ions triggers this process early.
High-performance concrete contains noble materials such as superplasticizer additives and supplementary cementitious materials (active silica, fly ash, blast-furnace slag, etc.) that cause it to be costlier as compared to conventional cement. In turn, the improvements in its mechanical properties have direct benefits, such as the reduction in the reinforcement ratio.
Considering the factors mentioned so far (reduction in the use of cement and therefore, reduced CO_2 emissions, reduction in the

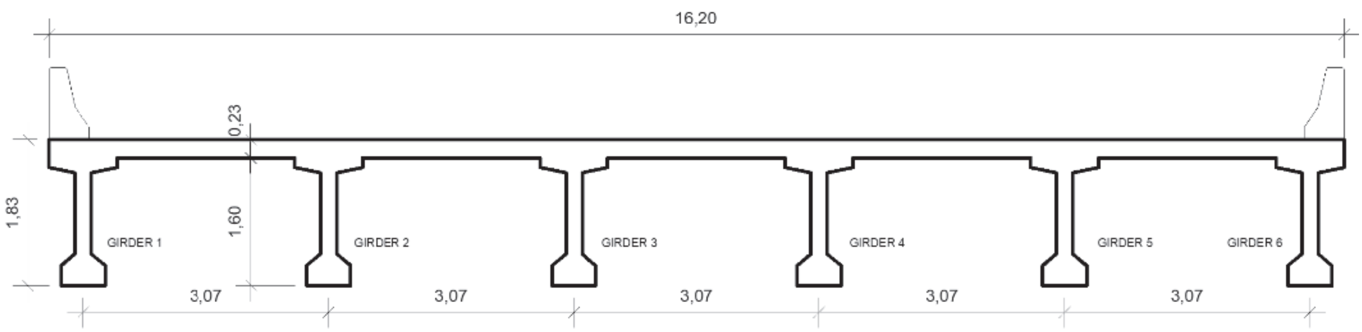

Figure 1
Cross section of the deck of Group A (dimensions in meters)

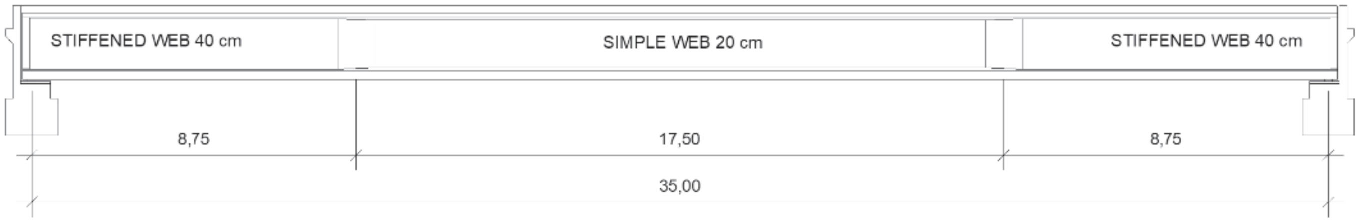

Figure 2
Longitudinal elevation of the deck (dimensions in meters)

deflection, reduction in creep, and the higher durability of the structure), the use of HSC is advantageous. This study aims to discuss the influence of the improvements obtained by using HSC on the cost of overpass decks.

2. Materials and methods

To evaluate the influence of the changes in concrete strength on the total cost of the deck, 12 decks with a 35-m span and 16.2-m width were separated into two groups. Group A consists of six decks with the same geometrical configurations and different characteristic concrete strengths (35, 45, 55, 65, 75, and 90 MPa). In turn, Group B consists of six decks with different geometrical configurations as well as different characteristic concrete strengths (the same as Group A).

Trentini [18] created software that determines the optimal pre-dimensioning in relation to the lowest cost for this type of deck– available at *"http://www.pcv.uem.br/programas/".* The selected geometrical configurations were obtained from this study, hence, the combination of the independent variables of Group A (Figure [1] and Figure [2]) is the one that provides the lowest cost for the deck with the characteristic concrete strength equal to or greater than 35 MPa, as shown below:

N_{lg} = 6 girders;
H_{lg} = 1.60 m;
H_{lj} = 23 cm;
E_{lg} = 8.75 m.

For Group B, each deck has a single geometrical configuration, i.e., each deck has the combination of independent variables that results in the lowest cost for its concrete strength class based on the study by Trentini [18].

2.1 Premises for the design

During the design the load prescribed in NBR 7188:2013 [5] was applied – moving road load and pedestrians on bridges, overpasses, pedestrian overpasses and other structures – considering the following, "the standard moving road load TB-450 is defined by a 450 kN vehicle type with 6 wheels, P = 75 kN, three cargo axles with 1.5 m spacing and occupation area of 18.0 m², surrounded by a constant load with uniform distribution p = 5 kN/m²."

Because the two-dimensional structure of the deck forms a grid, the variation of the position of the vehicle type according to NBR 7188:2013 [5] in the plane should be considered. Knowing the influence of the load points, it is possible to minimize the hypotheses of the structure calculations, thus evidencing the most unfavorable position for each dimensioning situation. Fauchart [10] elaborated on a method for the calculation of the transverse influence for this type of structure.

Fauchart's process is applied to multi-girder decks without intermediate cross beams. Furthermore, the girders should be fixed and have a constant inertia. In this process, the longitudinal work of the slabs is not considered [17].

To distinguish which part of the load is oriented to which girder, the cross-section of the deck is loaded, while observing the cross-sectional influence line that results from Fauchart's process. The vehicle type is positioned at the points of maximum positive and negative influence along the beam, and the load is multiplied by the value of the corresponding influence.

The result of this process is called a "load train", where each girder has its own graph of influence and, therefore, its train with independent load. The girders are then considered as fixed beams subjected to a load train.

During the construction process of this structure, the precast girders are lifted and placed on the supports. Thus, the weight of the girder is supported only by itself as the slab has not been built yet. Only after applying concrete and curing the slab, the additional loads can be considered to act on the composite section of the girder and the slab. The dimensioning presented in this study considers that any load acts on the section of the composite girder with slab, and therefore, none of the differences provided by these models are significant for what is proposed in this study.

The characteristic forces calculated so far need to be combined using the equations and coefficients proposed by NBR 8681:2003 – Structural actions and safety – Procedure, to become design forces [6].

To comply with the limits of durability demanded by item 13.4 of NBR 6118:2014 – Concrete structure design – Procedure [3], it is important that the parts with limited prestressing obey the limits of crack formation (ELS-F item 3.2.2) for the frequent combination and at the decompression limit state (ELS-D item 3.2.5) for the almost permanent combination, where the prestressing force P_∞ should be enough to obey both limits.

The present study adopts an estimated loss of prestressing force of 25% (10% immediate and 15% progressive) for the purpose of simplification. When dealing with the dimensioning of pro-stressed sections, it is important to consult item 9.6.3, Loss of prestressing force, of NBR 6118:2014 [3].

The dimensioning of the flexural longitudinal reinforcements is performed, considering it to be at the conventional state of rupture by

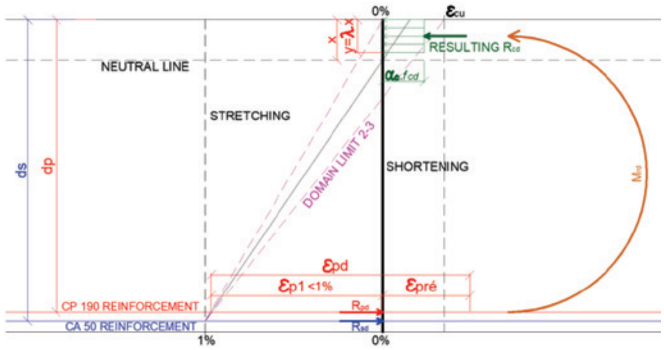

Figure 3
Specific deformation in ELS

excessive plastic stretching, i.e., it is stretched by 1%. Hence, the deformations in the section of the girder at the ultimate limit state (ULS) are distributed as presented in Figure 3 and the internal moment of resistance of the section (M_{rd}) is calculated using Equation [1].

$$M_{rd} = R_{cd} \cdot \left(dp - \frac{\lambda}{2} \cdot x\right) + R_{sd} \cdot (ds - dp) \tag{1}$$

For the calculation of the shear reinforcements, model I of ABNT NBR 6118:2014 [3] provided in item 17.4.2.2 is used. In turn, for the calculation of the torsional reinforcements, item 17.5 of the same standard is used, considering θ = 45°.
It is also necessary to verify if the maximum stress variation in the stirrups during the frequent combination does not exceed 85 MPa, as per item 23.5.5 of NBR 6118:2014 [3]. In this verification, the standard allows the consideration of only half of the contribution of concrete.
The vertical curves of the prestressing cables generate favorable forces for the shear stress that acts on the girders; however, for the purpose of simplicity, this effect will not be considered in this study.
In contrast to the girders, Fauchart's process is not applied to obtain the forces on the slab, as it neglects the longitudinal distribution of the load on the structure.
The uniformly distributed forces, such as the weight of the slab itself, can be analyzed as applied on a fixed beam, because the entire cross-section of the overpass has the same load. Nevertheless, because the non-uniform load distribution occurs in a different way in each direction, it is important to use a theory that best represents this situation. Rüsch [16] developed a set of practical tables for the dimensioning of bridge slabs using the plate theory, which considers the work of the loads in the cross-sectional direction as well as in the longitudinal direction of the deck. Therefore, Rüsch's tables will be used to obtain the forces on the slab.
After calculating the characteristic forces, the equations and coefficients of NBR 8681:2003 [6] are applied to obtain the calculated forces. The dimensioning of the slab is performed using the classical theory of the beam subjected to simple flexure, assuming the unitary width *b* and Equations (2) and (3).

$$x = \frac{1}{\lambda} \cdot d \left(1 - \sqrt{1 - \frac{M_d}{\frac{\alpha_c}{2} \cdot b \cdot d^2 \cdot f_{cd}}}\right) \tag{2}$$

$$A_s = \frac{M_d}{f_{yd}\left(d - \frac{\lambda}{2} \cdot x\right)} \tag{3}$$

It is also necessary to verify that the fatigue in the slab reinforcement is similar to the one performed in the girder. In NBR 6118:2014 [3], item 23.5.5 establishes the limit for the variation of stress during the frequent combination of 190 MPa for longitudinal, straight, flexural reinforcements with a diameter less than 16 mm.
There is the possibility that the reinforced slab can resist the acting force only with the longitudinal reinforcement, and a verification is performed according to item 19.4.1 of ABNT NBR 6118:2014 [3]. If the use of supplementary reinforcement is necessary, it is calculated according to item 17.4.2.2 of NRB 6118:2014 [3].

2.2 Cost function

The total cost of the deck C_T is determined by the sum of the costs of the construction and assembly materials.
Peinado et al. [14] presents, in their study, the cost of the different concrete classes provided by a concrete manufacturer company from the region of Maringá, Paraná and dated April 2014. For all unitary prices used in the calculation of the total cost to have the same date, the value presented by the authors was corrected by the National Construction Cost Index (INCC – Índice Nacional de Custo da Construção), which was 13.15% in the period of April 2014 to November 2015. The values in Table 2 already include 9.28 $/m³ for workforce and 10.61 $/m³ for pumping.
The cost of the girder casts $C_{f,lg}$ varies according to the number of repetitions, with 15.74 $/m² for two, 12.04 $/m² for two to five, 9.11 $/m² for five to eight, and 7.51 $/m² for more than eight repetitions, all of which were obtained from the SINAPI table of june 2015.

Table 1
Geometrical characteristics of the decks of Group B

f_{ck} (MPa)	N_{lg}	H_{lg} (m)	H_{lj} (m)	E_{lg} (m)
35	6	1.6	0.23	8.75
45	6	1.5	0.22	7.00
55	6	1.5	0.22	7.00
65	6	1.5	0.22	7.00
75	6	1.6	0.20	8.75
90	6	1.6	0.20	8.75

Table 2
Cost of concrete in function of the characteristic strength

Concrete strength class	Cost in April/2014 ($/m³)	Cost in November/2015 ($/m³)
C35	89.92	101.75
C45	99.47	112.55
C55	108.58	122.86
C65	115.90	131.14
C75	122.15	138.21
C90	130.15	147.27

Table 3
Unit costs

Variable	Unit of measurement	Cost	Source
$C_{c,lg}$	\$/m³	101.75~147.27	Peinado et al (2014, p. 7) + 13.25%
$C_{f,lg}$	\$/m²	16.32~7.81	SINAPI, PR, 06/2015 + 3.95%
$C_{a,lf,lg}$	\$/kg	1.70	SINAPI, PR, 06/2015 + 3.95%
$C_{a,lt,lg}$	\$/kg	1.70	SINAPI, PR, 06/2015 + 3.95%
$C_{a,vct,lg}$	\$/kg	1.70	SINAPI, PR, 06/2015 + 3.95%
$C_{p,lg}$	\$/kg	2.79	Hejos Construções Civis 2015
$C_{cj,p,lg}$	\$/un	100.76	Hejos Construções Civis 2015
$C_{i,lg}$	\$/un	54.39/ton	Hejos Construções Civis 2015
$C_{c,lj}$	\$/m³	101.75~147.27	Peinado et al (2014, p. 7) + 13.25%
$C_{a,lj}$	\$/kg	1.70	SINAPI, PR, 06/2015 + 3.95%

The costs obtained from the SINAPI table, PR, June 2015 were adjusted to the same date of November 2015, considering the correction index of 3.95%.
The cost of the cast to manufacture the pre-slabs is not evaluated in this function, as the comparison of costs is performed between overpasses with the same longitudinal span and the same cross-sectional width. Therefore, it is the same for all solutions and can be disregarded.
Some very specific services, such as prestressing, set of anchorage, and the lifting of girders are not included in conventional quotation tables. Therefore, this cost was provided by a construction company from the region of Maringá, specialized in this type of construction.
The cost of the lifting of precast girders is calculated as a function of the weight of the girders, assuming that the cost increases linearly with the lifted weight.
Additional costs such as corrugated metal sheaths and cement laitance injection were not considered because they are very low as compared to the total cost of the construction.

3. Results and discussions

Initially, the twelve decks were dimensioned, following the premises discussed in section 2.1. After the dimensioning, the costs of each combination were calculated using the unit costs presented in section 2.2.

3.1 Group A

Group A presents the same geometrical configuration and different characteristic concrete strengths. Table [4] presents the cost of each one of the six combinations. Figure [4] shows the variation of the cost as a function of the characteristic concrete strength, f_{ck}. It is possible to observe a small percentage variation in the cost (0.31% is the greatest variation between two combinations). Because all decks in Group A have the same geometrical configuration, i.e., they have the same concrete volume, the same cast area, the same lifting cost, and the same quantity of active reinforcement, the variation in the cost is in the cost of the concrete and the cost of the passive reinforcements.
To better visualize this dependency, Figure [5] shows the cost of the concrete and the cost of the passive reinforcement as a function of the characteristic concrete strength for Group A.
As observed in Figure [5], the cost of the passive reinforcement decreases when higher values are used for the characteristic concrete strength because, as seen in Figure [3], for higher concrete strength values, the height of the compressed concrete y is lower, which increases the lever arm of the flexural reinforcement in the calculation of the internal moment of resistance of the section and,

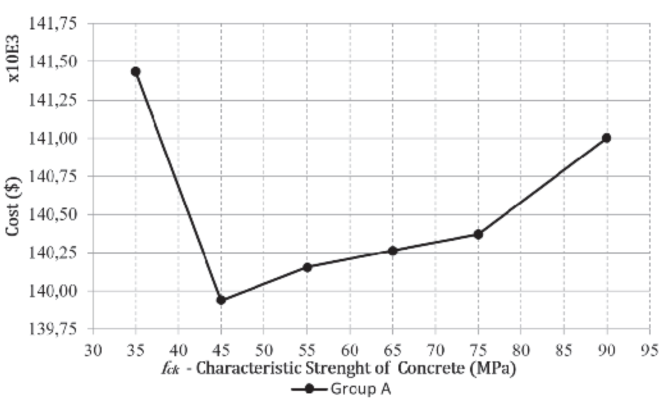

Figure 4
Cost as a funcion of f_{ck}, Group A

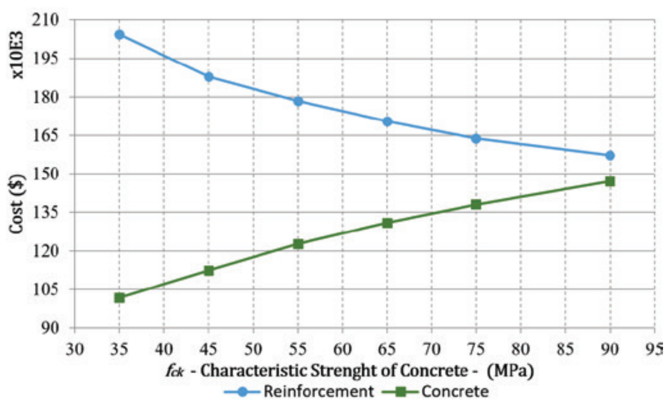

Figure 5
Cost of reinforcement as a funcion of f_{ck}, Group A

Table 4
Cost of the deck in function of the characteristic strength of concrete, Group A

f_{ck} (MPa)	N_{lg}	H_{lg} (m)	H_{lj} (m)	E_{lg} (m)	Cost ($)
35	6	1.6	0.23	8.75	141,436.12
45	6	1.6	0.23	8.75	139,937.98
55	6	1.6	0.23	8.75	140,155.21
65	6	1.6	0.23	8.75	140,264.41
75	6	1.6	0.23	8.75	140,372.06
90	6	1.6	0.23	8.75	141,000.89

Table 5
Cost of the deck in function of the characteristic strength of concrete, Group B

f_{ck} (MPa)	N_{lg}	H_{lg} (m)	H_{lj} (m)	E_{lg} (m)	Cost ($)
35	6	1.6	0.23	8.75	141,436.11
45	6	1.5	0.22	7.00	136,713.18
55	6	1.5	0.22	7.00	137,008.97
65	6	1.5	0.22	7.00	138,724.59
75	6	1.6	0.20	8.75	140,051.89
90	6	1.6	0,20	8.75	140,789.30

consequently, decreases the amount of reinforcement required. For the calculation of the shear reinforcement, the effect of the concrete strength is even greater (model I of NBR 6118:2014 [3] item 17.4.2.), and therefore, could also provide a smaller amount of reinforcement.
To observe this behavior, Figure [6] is presented, where the costs of the flexural and shear reinforcements of the girders from Group A are separated.
Hence, Figure [6] shows higher savings with the shear reinforcement than with the flexural, given that the comparison of the results for 35 MPa and 90 MPa provided a cost difference of 21.96% for the flexural reinforcement and of 36.00% for the shear reinforcement.

3.2 Group B

The cost of each combination of the decks from Group B is presented in Table [5].

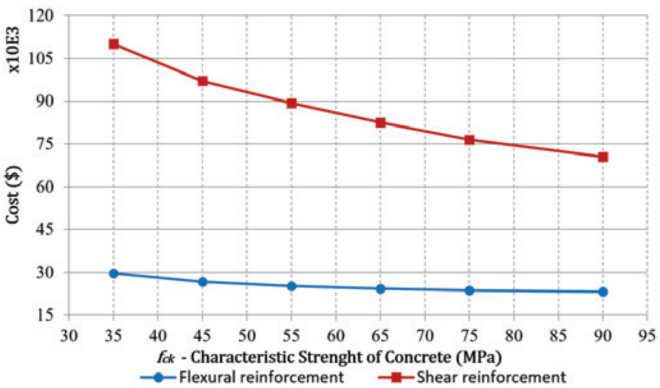

Figure 6
Cost flexural reinforcement and shear of the girder according f_{ck}, Group A

To better interpret the differences between the costs of the Groups A and B, Figure [7] presents the cost of both groups as a function of the characteristic concrete strength.
Hence, Figure [7] shows that, by combining the independent variables of dimensioning, it is possible to find more economical solutions.
On analyzing Table [5], it can be noted that for the higher concrete strengths, the optimal results contain beams with larger cross-sections. This occurs because the shear reinforcements make up a great part of the total cost of the deck and there is still the tendency that, by resisting a greater part of the shearing force with the concrete section, more economical results will be obtained. When lower concrete strength values are used, the increase in the cross-section does not cause a significant increase in the shear strength and, additionally, it increases the weight of the structure. In turn, in combinations that include stronger concretes, the increase in the cross-sectional area causes a more beneficial effect in the shear strength that balances the increase in the weight of the structure itself.

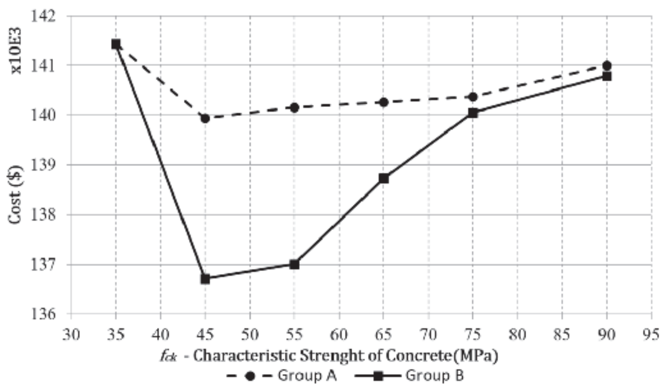

Figure 7
Cost as a funcion of f_{ck}, Group A and B

4. Conclusions

The aim of this study was to evaluate the economic aspects of the use of HSC associated with the geometrical optimization of decks from post-stressed multi-girder overpasses.

The main benefit of the use of HSC is the reduction of reinforcement, where the shear reinforcement is the one associated with the best savings. Special attention is drawn to the fact that by changing the f_{ck} from 35 MPa to 90 MPa, there is a 36% reduction in the shear reinforcement of the girder that also implies a reduction in the time required to build the structure.

This study also showed that not only alterations in the characteristic concrete strength, but also in the geometrical configurations of the deck are able to provide a better use of the variables that, in this case, provided a 3.34% reduction in the cost of the 45-MPa deck as compared to the 35-MPa deck.

Despite all beneficial effects provided by the use of HSC, i.e., the reduction in the use of cement, the reduction in CO_2 emissions, the reduction in deflections, the reduction of creep, and the greater durability of the structure associated with the geometrical optimization, the use of HSC also provides an overall reduction in the costs of the construction of the structure.

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5. List of notation

N_{lg} is the number of girders;
H_{lg} is the height of the girders;
H_{lj} is the slab height;
E_{lg} is the length of the web stiffening;
f_{ck} is the characteristic strength of concrete;
M_{rd} is the internal bending moment;
R_{cd} is the total reaction of compressed concrete;
d_p is the distance between the upper face of the composite girder with the slab and the center of the protension force;
x is the neutral line height;
R_{sd} is the reaction of passive steel;
d_s is the distance between the upper face of the composite girder with the slab and the center of the passive steel;
λ is the coefficient of approximation of the height of the reaction of compressed concrete during the ULS, 0.8 for $f_{ck} \le 50$ MPa or 0.8 - (f_{ck} - 50)/400 for $f_{ck} > 50$ MPa;
α_c is the coefficient of approximation of the compressive stress of the concrete during the ULS, 0.85 for f_ck≤50 MPa or 0.85·[1.0 - (f_{ck} - 50)/200] for $f_{ck} > 50$ MPa;

b is the width of the beam;
d is the depth of the compressed edge to the centroid of reinforcement;
f_{cd} is the concrete design strength;
f_{yd} is the steel design strength;
A_s is the steel area (for b width used);
$C_{c,lg}$ is the cost of the pumped ready mixed concrete, including the application and densification – girder;
$C_{f,lg}$ is the cost of the cast for the concrete structures, including the manufacturing, assembly, and disassembly – girder;
$C_{a,lf,lg}$ is the cost of the reinforcement, CA-50 steel, including the sectioning, 10% loss, bending and placement – longitudinal reinforcement resistant to the flexion of the girder;
$C_{a,lt,lg}$ is the cost of the reinforcement, CA-50 steel, including the sectioning, 10% loss, bending and placement – longitudinal reinforcement resistant to the torsion of the girder;
$C_{a,vct,lg}$ is the cost of the reinforcement, CA-50 steel, including the sectioning, 10% loss, bending and placement – cross-sectional reinforcement resistant to the shearing force and torsion of the girder;
$C_{p,lg}$ is the cost of the prestressing reinforcement, CP-190 RB steel, including the sectioning, placement and prestressing – girder;
$C_{cj,p,lg}$ is the cost of the set for the anchorage of prestressing reinforcement, including the setup – girder;
$C_{i,lg}$ is the cost of the lifting of each girder;
$C_{c,lj}$ is the cost of the pumped ready mixed concrete, including the application and densification – slab;
$C_{a,lj}$ is the cost of the reinforcing bars, CA-50 steel, including the sectioning, 10% loss, bending and placement – longitudinal reinforcement resistant to the flexion and shearing of the slab.