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Required tie spacing to prevent inelastic local buckling of longitudinal reinforcements in RC and FRC elements

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ABSTRACT

Compressed reinforcement buckling in concrete columns can reduce ductility in structural elements. In order to avoid this, design codes propose maximum required tie spacing. Nonetheless, they do not incorporate the positive effect of concrete fibers in their formulation, whose capability of delaying buckling has been proved. For this reason, recommendations for maximum required tie spacing for elements made with concrete, with or without steel fibers, and with normal or high strength, are proposed in this article. In order to achieve this, the mixed model proposed by Pereiro-Barceló and Bonet was extended to consider elements made of HSC with and without steel fibers thanks to the results of an experimental campaign of HSC columns with and without fibers, under monotonic loading. In these tests, the buckling critical load in compressed reinforcement was experimentally determined in all the columns. In addition, a comparison of the proposed transverse concrete separation with respect to the recommendations proposed by the main existing codes was made.

1. Introduction

Compressed reinforcement buckling can reduce both strength and deformation capacity in reinforced concrete (RC) elements in buildings and bridges [1]. One of the functions of transverse reinforcement is to provide enough stiffness to prevent compressed longitudinal reinforcement from buckling. Inadequate transverse reinforcement arrangement can cause longitudinal reinforcement to buckle under high compressive strains when the concrete cover spalls, or with fiber-reinforced concrete, when concrete fibers are inefficient [2].

According to the elastic theory, Bresler and Gilbert [3] propose a relation between reinforcement separation s and the diameter of longitudinal reinforcement D. They make buckling stress equal the yield stress of longitudinal reinforcement. According to these authors, in order to develop maximum longitudinal reinforcement effectiveness, the tie spacing must allow buckling stress that equals the yield stress of longitudinal reinforcement to be achieved, even when the concrete cover has spalled. These authors also propose a relation between the diameter of the longitudinal and transverse reinforcement by assuming that the deflected shape of the buckled bar affects up to two tie intervals. Also according to the elastic theory, Scribner [4] assume that buckling affects up to three tie intervals. Neither research work takes into account that buckling behavior is strongly influenced by the shape of the stress-strain curve within the reinforcing bar inelastic range (both longitudinal and transverse bars). For this reason, Papia et al. [5] employ the reduced modulus theory to analyze the inelastic buckling of longitudinal reinforcement. Pereiro-Barceló and Bonet [2] extend the Papia et al. [5] model by considering the fiber-reinforced concrete cover and its degradation. Mau and El-Mabsout [6] analyze behavior in isolated bars through finite elements simulation and determine that for s/D to be superior to 16, post-buckling load capacity must be smaller than the yield load. Based on the same numerical simulation, Mau [7] observe that if s/D is lower than a critical value, the yield plateau has a negligible effect. Mau [7] also reports that critical relation s/D is located between 5 and 7 for the steel he used. For greater separations, steel bars can become unstable after longitudinal reinforcement yields.

Pantazopoulou [8] proposes a method to calculate buckling loads that depend on stirrup separation. This method is based on the forces equilibrium of the buckled bar. This author assumes the deformed shape of the buckled bar to be cosenoidal in shape. Pantazopoulou [8] points out that the tangent modulus of longitudinal reinforcement depends on whether yield stress has been achieved or not. If yield stress is not achieved, the elasticity modulus is used, otherwise the reduced modulus is used. Pantazopoulou [8] proposes a procedure based on expanding the concrete core to know the strain that stirrups undergo and, consequently, if they are yielded or not.

Dhakal and Maekawa [9] use an energy equilibrium method to calculate critical buckling load, which allows them to know transverse

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reinforcement separation. The stiffness of transverse reinforcements is considered by means of elasto-plastic springs (null axial stiffness after achieving yield stress). These authors eliminate the stirrups closer to half the buckling length because they tend to enter the plastic zone rather than those that are far away. After conducting a parametric study about the number of stirrups to be eliminated, they propose minimum stiffness for transverse reinforcement, which is related to transverse reinforcement separation. Kashani et al. [10] proves that Dhakal and Maekawa [9] buckling model is in good agreement with the experimental results. Dhakal [11] extends the proposed method by Dhakal and Maekawa [9] to consider the fiber-reinforced concrete cover. This method bears in mind the concrete cover discretely at the stirrup location, but does not consider the cover degradation that results from overestimating buckling stress. Other authors [12–15] use buckled bar energy equilibrium methods to obtain critical load expressions depending on stirrup separation. Both transverse reinforcements [12,13,15] and concrete cover [14] are considered distributedly along instability length. With these methods, tie spacing can be obtained to not allow, for example, the bar to buckle before yield stress is achieved.

Several authors propose constitutive curve of steel bars including buckling [16–18], even considering corroded steel bars [19,20], and finite element models to obtain the constitutive curves including buckling [21].

As Pereiro-Barceló and Bonet [2] point out, the aforementioned analytical models followed to estimate the critical buckling load of compressed bars are not valid for the whole range of transverse reinforcement separations and do not consider progressive fiber concrete cover degradation. Only the mixed model proposed by Pereiro-Barceló and Bonet [2] is valid for any tie spacing because it contemplates transverse reinforcement discretely and the fiber concrete cover distributedly, and it also considers fiber concrete cover degradation.

Design codes propose separations for transverse reinforcement to prevent compressed reinforcements from buckling [22–25]. Nevertheless, these recommendations do not take into account the favorable effect of steel fibers in concrete [2,26,27].

Consequently, this article proposes design recommendations to determine the required tie spacing in elements made of normal strength concrete (NSC), fiber-reinforced normal strength concrete (FRNSC), high strength concrete (HSC) and fiber-reinforced high strength concrete (FRHSC). To achieve this, the model of Pereiro-Barceló and Bonet [2] was used. This model was calibrated for NSC and FRNSC elements. In order to extend the model application field, an experimental campaign of HSC and FRHSC columns was performed.

2. Determining critical buckling stress

This section briefly describes simplified expressions to determine the critical buckling stress proposed by Pereiro-Barceló and Bonet [2]. These expressions are used in Section 4 to propose the recommendations of the required separation of transverse reinforcements for NSC, FRNSC, HSC and FRHSC elements.

2.1. Critical buckling stress

The mixed model of Pereiro-Barceló and Bonet [2] provides the buckling critical stress of passive reinforcements in NSC and FRNSC elements. It contemplates stirrups discretely and the concrete cover continuously. The model is based on two fundamental parameters: the stiffness of transverse reinforcement α_s and the distributed stiffness of concrete cover α_c. The expression of α_s is shown in Expression (1):

$$\alpha_s = \frac{E_{sw} \cdot A_{sw}}{L_{ef}} \tag{1}$$

where:

E_{sw}: the tangent modulus of transverse reinforcement. In order to know this modulus, it is necessary to determine if reinforcement is yielded or not. For this purpose, it is necessary to relate the transverse strain to the longitudinal strain through the dilatancy parameter [28–31].

A_{sw}: the transverse reinforcement area.

L_{ef}: the effective transverse reinforcement length, which depends on the reinforcement arrangement and the type of load (concentric or eccentric) [2].

The distributed stiffness of cover α_c was calibrated experimentally for NSC and FRNSC. The value of $\alpha_c = 70$ MPa was experimentally obtained. This value can be guaranteed until a longitudinal reinforcement strain of $\varepsilon_{crit,\eta \leqslant 1}$ (Expression (2)) is achieved, which depends on $f_{R,1}$ (residual tensile strength that corresponds to a Crack Mouth Opening Displacement (CMOD) of 0.5 mm in the flexural tensile strength test (UNE EN 14651:2007 [32])). Beyond this strain, the fiber reinforced concrete cover is too degraded and a null value of α_c is considered ($\alpha_c = 0$ MPa).

$$\varepsilon_{crit,\eta \leqslant 1}\,(‰) = 0.66 f_{R,1}\,(\text{MPa}) + 7.15 \tag{2}$$

Once the α_s and α_c values are known, buckling critical stress σ_{crit} can be obtained through the following expression:

$$\sigma_{crit} = c_c \frac{\pi^2 E_r I}{s^2 A} \tag{3}$$

where:

s: Transverse reinforcement separation.

E_r: The reduced modulus of the longitudinal reinforcement proposed by Papia et al. [5].

I: The inertia moment of longitudinal reinforcement.

A: The transverse reinforcement area.

c_c: The critical adimensional stress c_c (4)-(7). c_c is the relation between the critical buckling stress of the bar and the critical buckling stress of the bar hinged between two consecutive rigid stirrups ($\frac{\pi^2 E_r I}{s^2 A}$).

If $k_{cs} = 0$

$$c_c = 4 \cdot \left(1 - \frac{1}{1 + 0.09\gamma^{0.58}}\right) \tag{4}$$

If $0 < k_{cs} \leqslant 30$

$$c_c = a_1 \cdot e^{b_1 \cdot \log_{10}\gamma} + c_1 \quad \text{when} \quad c_c \geqslant c_{c_{\eta=1.4}}(\gamma) \tag{5}$$

where:

$$a_1 = 0.35 k_{cs}^{0.5} - 0.0066$$
$$b_1 = \frac{1.15 k_{cs} + 0.035}{k_{cs} + 0.029}$$
$$c_1 = \frac{-0.0116 k_{cs} + 0.062}{k_{cs} + 0.036}$$

$$c_c = a_2 \cdot e^{b_2 \cdot \log_{10}\gamma} + c_2 \quad \text{when} \quad c_c < c_{c_{\eta=1.4}}(\gamma) \tag{6}$$

where:

$$a_2 = \frac{5.5 k_{cs}^3 + 99.3 k_{cs}^2 + 189 k_{cs} + 91.2}{k_{cs}^3 + 93 k_{cs}^2 + 417 k_{cs} + 25.4}$$
$$b_2 = \frac{1.14 k_{cs}^2 + 1.26 k_{cs} + 0.08}{k_{cs}^2 + 1.535 k_{cs} + 0.404}$$
$$c_2 = \frac{-0.02 k_{cs}^2 - 0.375 k_{cs} - 1.07}{k_{cs}^2 + 5 k_{cs} + 0.325}$$

If $k_{cs} > 30$

$$c_c = \left(\frac{s}{\pi}\right)^2 \sqrt{\frac{12\alpha_c}{E_r I}} \tag{7}$$

where:

$$\gamma = \alpha_s s^3 / E_r I \tag{8}$$