Volume 8, Number 5 (October 2015) p. 749-754 • ISSN 1983-4195
http://dx.doi.org/10.1590/S1983-41952015000500011

REVISTA IBRACON DE ESTRUTURAS E MATERIAIS
IBRACON STRUCTURES AND MATERIALS JOURNAL

Response to the Discussion of "Concrete structures. Contribution to the safety assessment of existing structures, Rev. IBRACON Estrut. Mater. 2015, vol.8, n.5, pp. 744-748, proposed by Santos, D. M.; Stucchi, F. R. and Beck, A.T."

Resposta à Discussão de "Estruturas de concreto. Contribuição à análise de segurança em estruturas existentes, Rev. IBRACON Estrut. Mater. 2015, vol.8, n.5, pp. 744-748, proposta por Santos, D. M.; Stucchi, F. R. and Beck, A.T."

D. COUTO [a b]
M. CARVALHO [b]
A. CINTRA [b]
P. HELENE [b c]

RESPONSE | RÉPLICA

[a] *University of Campinas, Campinas, SP, Brazil;*
[b] *PhD Engenharia, São Paulo, SP, Brazil;*
[c] *University of São Paulo, São Paulo, SP, Brazil.*

Received: 30 Sep 2015 • Accepted: 30 Sep 2015 • Available Online: 13 Oct 2015

This article's authors are grateful to the Colleagues for giving this opportunity to expand the discussion of this important topic. The intention of the authors is to contribute to reducing unnecessary expenditure between the various stakeholders in a concrete structure, clarifying the real differences between the usual security analysis of new concrete structures (to be built) and verification of safety in existing works (already built, retrofit works, buildings in use or under construction). Unlike stated in the discussion, the aim is to avoid unnecessary structural strenghtening and contribute to sustainability, without jeopardizing the structural safety.

The authors agree on the importance of inspection and diagnosis of the existing structure as previous and indispensable subsidy to adopt new safety verification criteria as noted in the abstract, in the introduction and in item 4.1.3 of the article under discussion. The authors thank the Colleagues for sharing the same vision. As an example, in item 4.1.3 of the article, it says clearly and objectively: *"[...] the security check can proceed to the third step, which is the careful observation of the finished structure giving geometrical measures position armor, armor rate, eccentricity tolerances, level and plumb, thickness slabs, or checking the accuracy of execution of the structure"*.

About the article's Section 2, the Colleagues criticize the values of the γ_c (γ_{c1}, γ_{c2}, γ_{c3}) partial factors, arguing that these values are difficult to obtain and need to be assessed in a probabilistic manner, defending that it is incorrect to evaluate these coefficients with cores results.

Surely there must have been a misunderstanding of Colleagues because the available literature on this topic is abundant [1][2][3]. Following it is transcribed pan excerpt of the ***fib*** Model Code 2010 (commentary) which clarifies the issue, noting that there are small adjustments between the notations of the ABNT NBR 6118:2014 and the ***fib*** Model Code 2010, but the concepts and meanings are the same.

Commentary to item 4.5.2.2.3 of fib Model Code 2010, p. 62

Indicative values are γ_{Rd1} = 1.05 for concrete strength and γ_{Rd1} = 1.025 for steel strength. In some cases – such as punching in the ULS, where concrete crushing is governing the behavior – models may be affected by larger uncertainty, which can be accounted for by adding a specific factor in the verification formulas). For taking into account geometrical uncertainties an indicative value is γ_{Rd2} = 1.05 (regarding the variability of the size of the concrete section or the position of the reinforcing steel).

For concrete strength this leads to $\gamma_{Rd,c} = \gamma_{Rd,c1} \cdot \gamma_{Rd,c2} = 1.05 \cdot 1.05 = 1.10$ and for steel strength $\gamma_{Rd,s} = \gamma_{Rd,s1} \cdot \gamma_{Rd,s2} = 1.025 \cdot 1.05 = 1.08$.

More over:

$$\gamma_m = \frac{R_k}{R_d} = \frac{\mu_R(1-k\cdot\delta_R)}{\mu_R(1-\alpha_R\cdot\beta\cdot\delta_R)} = \frac{1-k\cdot\delta_R}{1-\alpha_R\cdot\beta\cdot\delta_R} \quad (1)$$

considering a normal distribution, or

$$\gamma_m = \frac{R_k}{R_d} = \frac{\exp(\mu_{\ln R} - k\cdot\delta_{\ln R})}{\exp(\mu_{\ln R} - \alpha_R\cdot\beta\cdot\delta_{\ln R})} = \exp(-k\cdot\delta_{\ln R} + \alpha_R\cdot\beta\cdot\delta_{\ln R}) \quad (2)$$

considering a lognormal distribution.

Commonly the 5% fractile is used for the characteristic value, yielding k = 1.64. moreover, most commonly the following values are used:

α_R = 0.8 being the sensitivity factor of the parameter under consideration, based on the simplified level II method as suggested by König and Hoser in CEB Bulletin 147: "Conceptional Preparation of Future Codes – Progress Report" (CEB, 1982).

β = 3.8 for structures of consequence class 2 according to EN 1990.

δ_R = coefficient of variation of the parameter under consideration: for example d_c = 0.15 is commonly used for normal quality concrete and d_s = 0.05 for reinforcing steel.

Based on these commonly used values and considering a normal distribution γ_c = 1.39 *and* γ_s = 1.08.

This finally results in:

$\gamma_c = \gamma_{Rd,c} \cdot \gamma_c = 1.10 \cdot 1.39 = 1.52 \cong 1.50$ *and*

$\gamma_s = \gamma_{Rd,s} \cdot \gamma_s = 1.08 \cdot 1.08 = 1.17 \cong 1.15$.

The commonly used partial safety factors mentioned before can be modified in operational codes, by justifying the values of the underlying assumptions.

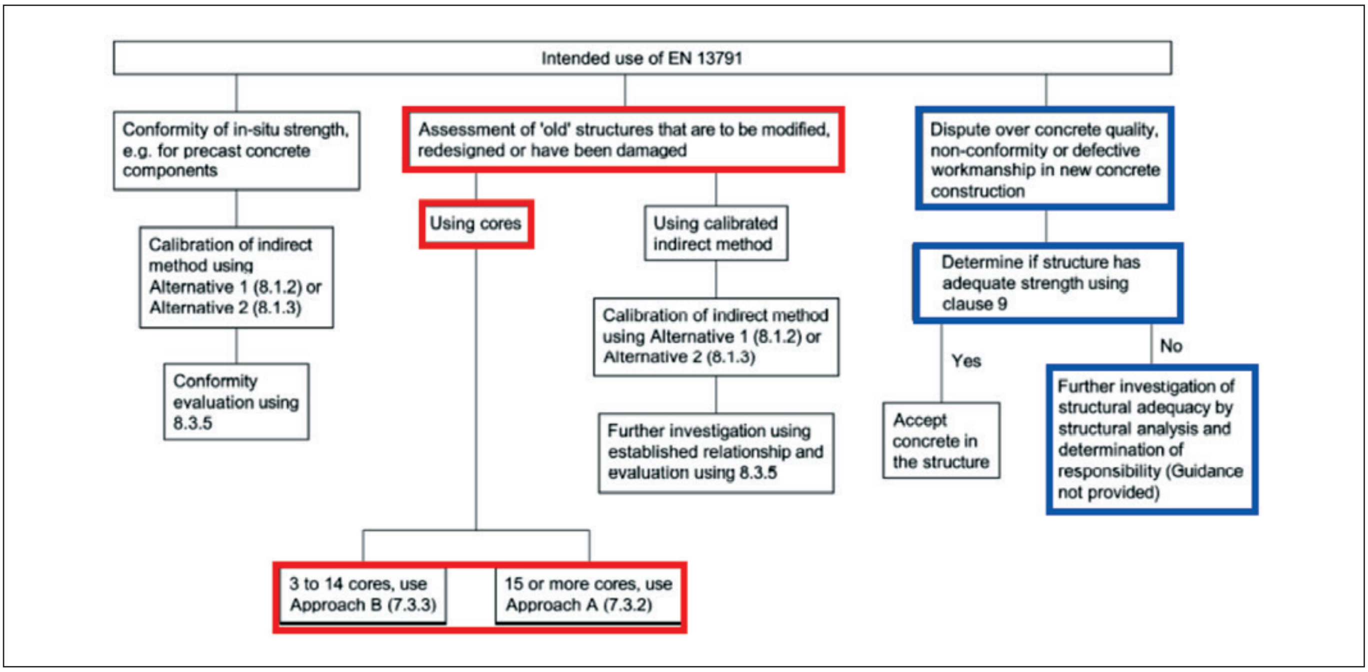
Following, the Colleagues argue that *"[...] The use of reduced partial safety factors as suggested (γ_c < 1,27 and γ_s = 1,05) cannot be utilized since it cannot be sustained, up to this moment, by structural reliability analysis"*.

Again, in the judgment of the article's authors, there must have been a misunderstanding of the Colleagues, for structural reliability analysis started to be used in very simple and special cases, and very after the adoption of partial safety factors, precisely because of the difficulty of a full probabilistic treatment required for a reliability analysis. The story is exactly the opposite, i.e., first the partial coefficients were calibrated to "get" and "set to" the old global safety factors of the first standards which used allowable stress design and, today, the β coeficientes of reliability are set to the values obtained with the partial safety factors. Unfortunately, the Colleagues wanted to reverse the course of the history of the safety introduction in the structural design (see Zagottis [4]).

The authors acknowledge the insightful and correct contribution of colleagues with respect to ACI 318-11 (equivalent in this regard to the current ACI 318-14) and ACI 214.4R-10. Both converge to equivalent values but are mutually exclusive, i.e., in new structures and under construction, it should be used only ACI 318-11 criteria. In existing structures it can be used the ACI 214.4R-10 criteria, combined with ACI 318-11 (Chapter 20) and ACI 562-13, according to each case.

Regarding the critics to Section 4.2.4 of the article, the Colleagues also argue that *"[…] the code EN 13791 [24] allows for the use of equations (16) and (17) only for old structures. In the event of non-compliance, a different set of procedures is used"*.

The authors explain that both procedures (structural analysis of "old" structures and cases of non-compliance of works under construction) are described in EN 13791:2007 (see transcript of an



excerpt from the introduction and *Flowchart 1* of the standard), and justify the use of the criteria adopted in Section 4.2.4 with a view that in cases of non-compliance in structures under construction, the procedure suggested by EN 13791:2007 item 9 (highlighted in blue in *Flowchart 1*) is much less conservative than the one suggested by item 7 (highlighted in red in the Flowchart 1, used by the authors and recommended for analysis of “old” structures), and is similar to the ACI 318-11 criteria.

EN 13791:2007 Introduction

[…] The following examples illustrate where this estimate of in-situ strength of concrete may be required:

- when an existing structure is to be modified or redesigned.
- to assess structural adequacy when doubt arises about the compressive strength in the structure due to defective workmanship, deterioration of concrete due to fire or other causes;
- when an assessment of the in-situ concrete strength is needed during construction;
- to assess structural adequacy in the case of non-conformity of the compressive strength obtained from standard tests specimens;
- assessment of conformity of the in-situ concrete compressive strength when specified in a specification or product standard.

[…]An outline of the procedures for these different uses of this standard is given in Flowchart 1[…]

Finally, the Colleagues express contrariness with respect to the example given by the authors in the article under discussion. The intention of the authors was to contribute to clarify the concepts so that they regret that this has not occurred with respect to these colleagues and commit to produce and publish more compelling and didactic examples.

[1] FÉDERATION INTERNATIONALE DU BÉTON. ***fib*** (CEB-FIP) Model Code for Concrete Structures 2010. Lausanne: Ernst & Sohn, 2013. p. 62.

[2] CREMONINI, R. A. Análise de Estruturas Acabadas: Contribuição para a Determinação da Relação entre as Resistências Potencial e Efetiva do Concreto. São Paulo, EPUSP, 1994 (tese de doutoramento).

[3] FUSCO, P. B. Controle da resistência do concreto. ABECE Informa, São Paulo, n. 89, p.12-19, Jan/Fev. 2012.

[4] ZAGOTTIS, D. L. de. Introdução da Segurança no Projeto Estrutural. São Paulo, EPUSP-PEF, 1974. 116 p.