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Post-cracking steel fiber reinforced concrete slabs with subsoil interaction

Daniel Pieszka[1,a *], Karel Kubečka[1,b] and Ivan Kološ[2,c]

[1]VSB-Technical University of Ostrava, Faculty of Civil Engineering, Department of Building Constructions, L. Podeste 1875, 708 33 Ostrava Poruba, Czech Republic

[2]VSB-Technical University of Ostrava, Faculty of Civil Engineering, Department of Structural Mechanics, L. Podeste 1875, 708 33 Ostrava Poruba, Czech Republic

[a]daniel.pieszka@vsb.cz, [b]karel.kubecka@vsb.cz, [c]ivan.kolos@vsb.cz

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Abstract. The aim of the paper is to investigate the flexural behavior and property changes of concrete structures reinforced by steel fibers (SFRC) and to use the results for carrying capacity assessment of SFRC post-cracked slab on ground structure with subsoil interaction effect. Because the national codes cover neither design nor assessment of SFRC structures, the investigation is generally based on the nonlinear fracture mechanics models to establish the stress-crack opening and load-crack mouth opening displacement relationship. Then the flexural tensile strength and residual flexural tensile strength of the post-cracked SFRC structure is determined with respect to subsoil interaction.

Introduction

Generally, all the constructions must resist all possible loads and the other effects, which can occur during building of construction and subsequently during construction usage within the construction lifetime. Because the national codes cover neither design nor assessment of SFRC structures, the durability estimation of the SFRC structures belongs to hot topics at present time. The methods for lifetime assessment of construction shall take into account the presence of cracks in structures with respect to the crack effect on the flexural behavior of the structure and the subsoil interaction on the cracked structure. Property changes of the cracked structure must be set down at first and the three-point bending test of notched beams is considered as a relevant experimental method [1,2].

Slab on the subsoil

Bending theory. The bending properties of a plate (slab) depend on its thickness as compared with its other dimensions. This paper deals with industrial flooring slabs, so the Kirchhoff's thin plates theory with small deflection is used. Used approach is not applicable in case of inhomogeneous subsoil or very thick slabs. The simplest assumption is the intensity of subsoil reaction is proportional to deflection of the slab. The partial differential equation (Eq. 1) for the deflection with respect of the subsoil reaction $p = kw$ becomes

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D} - \frac{kw}{D} \qquad \Delta\Delta w = \frac{k}{D}\left(w_0 - w\right), \text{ where } \; w_0 = \frac{q}{k} \tag{1}$$

$$D = \frac{Eh^3}{12\left(1-\nu^2\right)} \tag{2}$$

where w is deflection of the slab, q is the intensity of lateral load and k is the modulus of the subsoil reaction which depends on the properties of the soil. In this idealization, the subsoil is physically represented as a system of closely spaced spring elements where each of spring is deformed by the stress applied directly to them and the neighboring spring elements remain unaffected. The shear interaction between closely spaced spring elements is not taken in respect. Due to this approach, the displacement has discontinuous behavior (see Fig. 1).
Quantity D taking the place of quantity EI in the case of beams is called flexural rigidity of the plate and as it is shown below (Eq. 2), the flexural rigidity depends on the modulus of elasticity in tension and compression E and on the thickness of the slab h. (the ν is the Poisson's ratio).

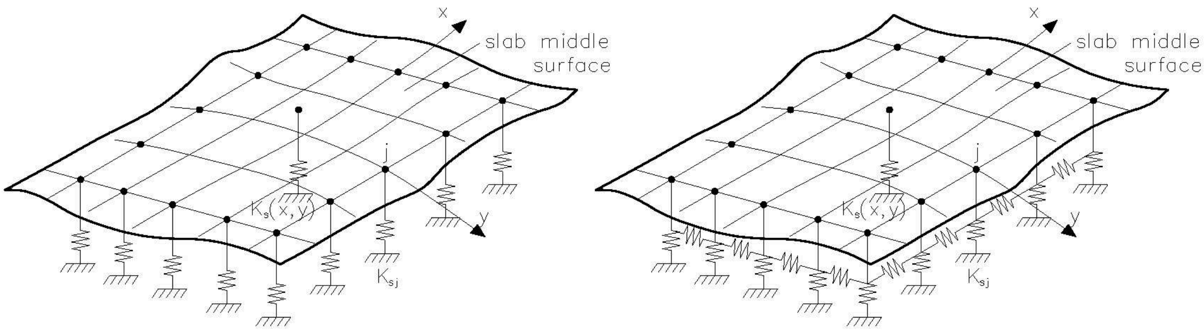

Fig. 1: Subsoil behavior without and with shear interaction between spring elements

If the deflection of the slab is produced by concentrated force F the equation (Eq. 1) has to be modified. The maximum deflection can be set as

$$w_{\max} = \frac{F\sqrt{k}}{2\pi k\sqrt{2D}}\int_0^{\infty}\frac{1}{\sqrt{2}}\frac{\mathrm{d}u}{1+u^2} = \frac{F}{8\sqrt{kD}}, \text{ where } u \text{ depends on slab geometry and } D,\ k\ [10] \quad (3)$$

and the maximum pressure on the foundation with this value of deflection is

$$p_{\max} = kw_{\max} = \frac{1}{8}F\sqrt{\frac{k}{D}} \quad (4)$$

It needs to be noted, that in the slab-subsoil interaction system, areas with contact loosing can be found (see Fig. 2). In the example of Fig. 2, the *zone 2* is without the contact and the stress is carried by *zone 1* only. Value of *zone 1* area depends on deflection of the slab produced by concentrated force F and on the properties of the subsoil. In the estimation, the contact stress needs to be set to zero in *zone 2* and all the calculations have to be made as the iteration process.

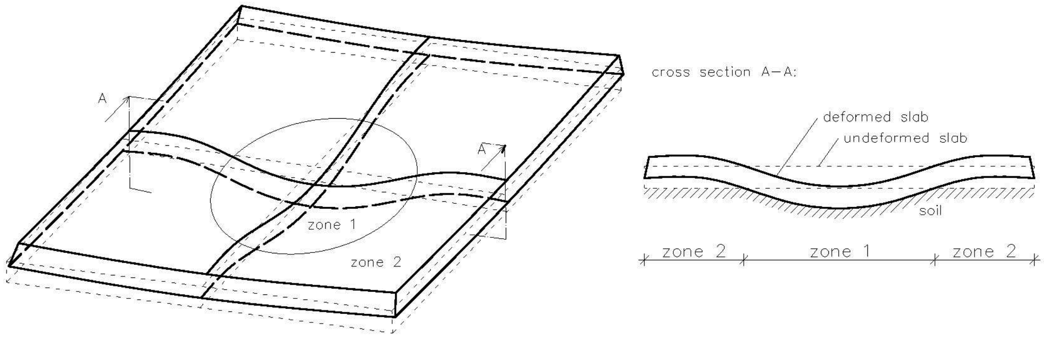

Fig. 2: Contact between the slab and the subsoil

The maximum tensile stress is at the bottom of the slab under the point of application of concentrated force F. In this case, the infinite value of bending moment is obtained in this point. The above mentioned Westergaard theory gives a equation for the maximum tensile stress at the bottom of the slab, which comes from the thick plate theory

$$\sigma_{u,\max} = 0{,}275(1+\nu)\frac{F}{h^2}\log\frac{Eh^3}{kb^4} \tag{5}$$

$$b = \sqrt{1{,}6c^2 + h^2} - 0{,}675h \text{ for } c \le 1{,}724h \text{, in other cases } b = c \tag{6}$$

where c is the radius of the circular area over which the load F is assumed to be uniformly distributed. In case a square loading area of dimension a is applied, the c has to be replaced by expression of $0{,}57a$.

Material properties. As is shown above, the flexural behavior of the slab mainly depends on the modulus of elasticity E and on the thickness of the slab h and these properties change when the crack in the slab occurs. Equivalent elastic crack models belong to the simplest non-linear models capturing the fracture behaviour of concrete. The approach is adopted in order to determine the equivalent elastic crack length. According to this approach the nonlinear fracture of quasi-brittle materials is simulated by replacing the real body containing a crack of a certain length and a fracture process zone (FPZ) ahead of it with a brittle body with an effective crack (longer than the initial one) and then forcing both bodies to exhibit the same stiffness parameters of the sample. Fracture tests are conducted in the configuration of single edge notched beams subjected to a three point bending (see Fig. 3). Specimens are prepared from standard beams of nominal dimensions $W \times B \times L$ (i.e. width × breadth × length) equal to $150 \times 150 \times 700$ mm. Specimens prepared from standard beams are provided with notches of relative length α_0 approximately equal 25 mm, using a diamond saw and are loaded under displacement control (see Fig. 3). The loading force and the mid-span deflection are recorded.

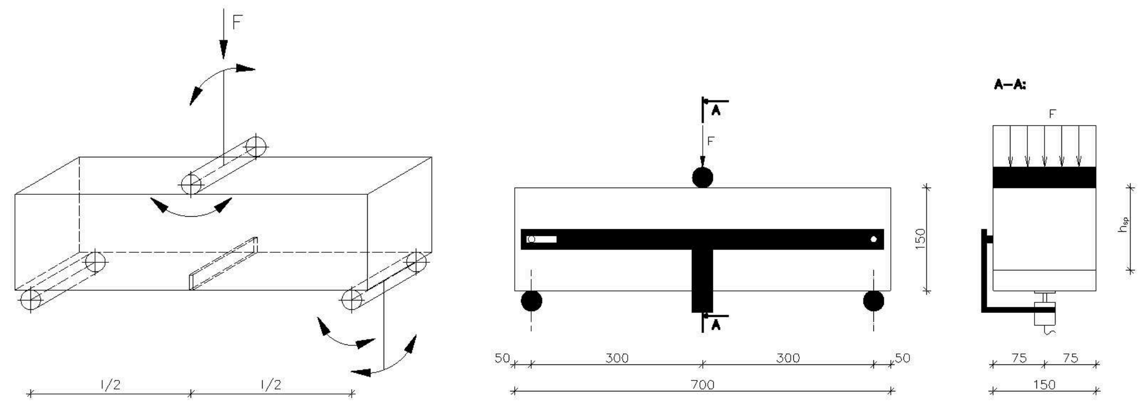

Fig. 3: Three point bending fracture test

After reaching the maximal force, unloading is performed (at value of approx. 80% of the peak load of the corresponding loading cycle). The unloading is performed automatically by switching the regime of the used electro-mechanical testing machine. Manual switching does not allow precise attaining of the desired unloading level. The dynamic event is indicated and the deflection is recalculated for a load-crack mouth opening displacement ($F-CMOD$) diagram (see Fig. 4). If the small number of points on the long section occurs, the approximation by polynominals can be used for further analysis.

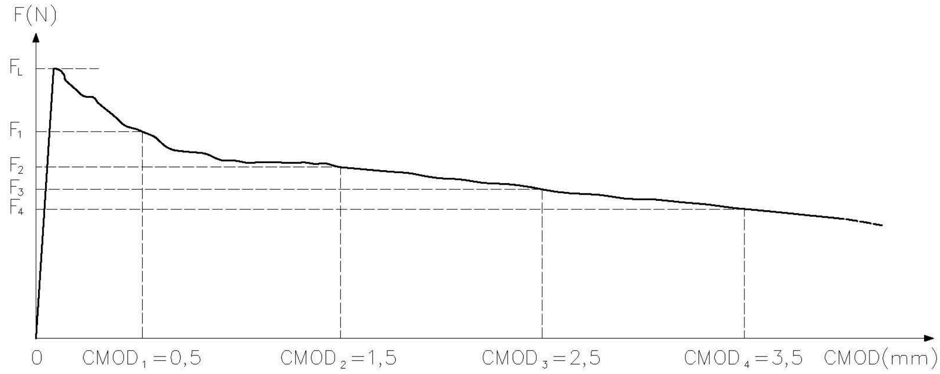

Fig. 4: Load-crack mouth opening displacement diagram (F−CMOD)

The equivalent elastic crack model [1,2,3] is used for determination of the effective crack length. It needs to be noted that adopted estimation of effective crack length is closest to the model according to Jenq-Shah. If the Nalathambi-Karihaloo modification based on the secant compliance is used, the effective crack lengths is larger than these determined from the loading compliance.

The flexural tensile strength and residual flexural tensile strength of the post-cracked SFRC structure, which are obtained from the three point bending fracture test, are used for estimation of the modulus of elasticity and the effective thickness of the slab.

Summary

To check the proposed analytical approach to the carrying capacity assessment of SFRC post-cracked slab on ground structure, more experiments need be carried out. The investigation based on the nonlinear fracture mechanics models, especially a smeared multi-fixed crack model, has to be used. Behavior of closely spaced spring elements has to be modelled as an elasto-plastic nonlinear system with respect to risk analysis assessment instruments [4-9].

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