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DESIGN OF STEEL-CONCRETE (SC) COMPOSITE STRUCTURES FOR THE CONSTRUCTION STAGE

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ABSTRACT

SC structures are a form of construction comprising two steel plates with a concrete infill between them such that the steel and concrete act compositely. As well as the design checks performed at the composite stage, the steel plates of SC structures must also be assessed for the pre-composite construction stage (prior to and during concreting). During construction, the steel plates may be subjected to considerable forces due to transportation, lifting, concreting operations, as well as forces due to construction activity and environmental conditions. In some cases, these actions may govern the sizing of the steel elements.

This paper presents guidance for the design of SC panels at the construction stage. The relevant Eurocodes for the design of steel structures such as EN 1993-1-1 (2005), EN 1993-1-5 (2006) and EN 1993-1-7 (2007) are adopted and, where necessary, their requirements are adapted to SC structures or new analytical expressions are developed. The analytical models were validated against finite element analyses and, in some cases, against alternative methods available in the literature.

INTRODUCTION

SC is the name for a generic steel-concrete composite construction system using planar components comprising two steel plates connected by a grid of tie bars with structural concrete between the plates. The plates act as load bearing formwork during the placement of the concrete (core) and, in the completed condition, they provide the reinforcement to the concrete. Execution operations, including concrete filling result in a range of different forces on the steel elements of SC panels. These should be checked against limiting values to avoid failure or permanent (plastic) deformation of the panels.

A European funded project investigating the behaviour of SC structures was recently completed (SCIENCE, 2017). As part of this project, a methodology was developed to enable designers to assess the structural behaviour of the unfilled panels during construction. Guidance is provided for calculating the stiffness of the panels and their resistance to axial and out-of-plane loads. Relationships are provided for calculating the hydrostatic pressure due to concreting and the plate resistance to concrete pressure.

DESIGN OF UNFILLED PANELS FOR OUT-OF-PLANE LOADS

The unfilled panels require special treatment due to their shear flexibility as the plates are connected only by discrete bars. This means that, under transverse loading, the contribution of shear deflection to overall deflection is significant and cannot be neglected. The total panel deflection is therefore given by:

$$\delta_{\max} = \delta_{v,\max} + \delta_{b,\max} \tag{1}$$

where

$\delta_{v,\max}$ is the deflection due to shear;
$\delta_{b,\max}$ is the deflection due to flexure.

Deflections due to flexure

Flexural deflections can be assessed using simple bending theory (ignoring any shear effects) by assuming that only the plates are effective in the calculation of the second moment of area I_α:

$$I_\alpha = 2bt_p\left(\frac{h_c + t_p}{2}\right)^2 \tag{2}$$

where

b is the width of the plate;
t_p is the plate thickness;
h_c is the spacing between the plates.

Hence, $\delta_{b,\max}$, the deflection due to flexure is given by:

$$\delta_{b,\max} = \left(\frac{5wL^2}{384EI_a}\right) \tag{3}$$

where

w is the applied load per unit length of panel;
L is the span.

Deflections due to shear

The shear stiffness of an unfilled SC panel is calculated using the stiffness method for the bar/plate system shown in Figure 1. The out-of-plane shear deflection, Δ_H, due to the applied shear force, V, is given by:

$$\Delta_H = \left(\frac{s_1^3}{24EI_p} + \frac{s_1^2 h_c}{12EI_b}\right)V \tag{4}$$

where

E is the Young's modulus;
I_p is the second moment of area of one plate $= \left(b_{eff}t_p^3/12\right)$;
b_{eff} is an effective width of the plate acting with the bar $= 20t_p$;
I_b is the second moment of area of the bar $= \left(\pi\varphi^4/64\right)$;
φ is the bar diameter;
s_1 is equal to the distance between the bars along the panel length.

The shear stiffness per unit width of panel, k_v, is given by:

$$k_v = \frac{Vs_1}{\Delta_H s_2} \tag{5}$$

where s_2 is the bar spacing in the transverse direction.

The shear stiffness can be converted to an equivalent shear modulus, G_{eff} :

$$G_{eff} = \frac{Vs_1}{\Delta_H s_2 h_c} = \frac{k_v}{h_c} \tag{6}$$

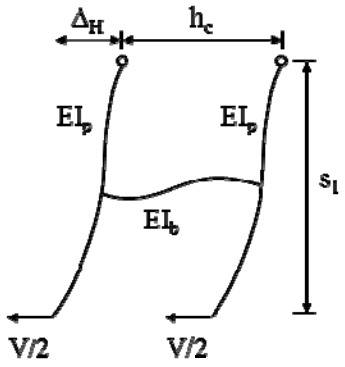

Figure 1. Single bar and plate assembly used for effective stiffness calculation.

The calculation method and the expression for the effective width, b_{eff} , were validated using a finite element model with the geometry, loading and boundary conditions shown in Figure 1. The bar and the plates were modelled using solid elements. Plate thickness, bar spacing and distance between plates were varied. The plate width in the FE model was taken to be equal to s_1, assuming equal bar spacing in the orthogonal directions. A typical bar diameter of 20 mm was assumed. The results obtained from the analyses suggest that an effective width of plate equal to 20 times the thickness is appropriate for the calculation of the shear stiffness. The deflections calculated using Equation 4 are compared with the FE results in Figure 2 and show very good agreement.

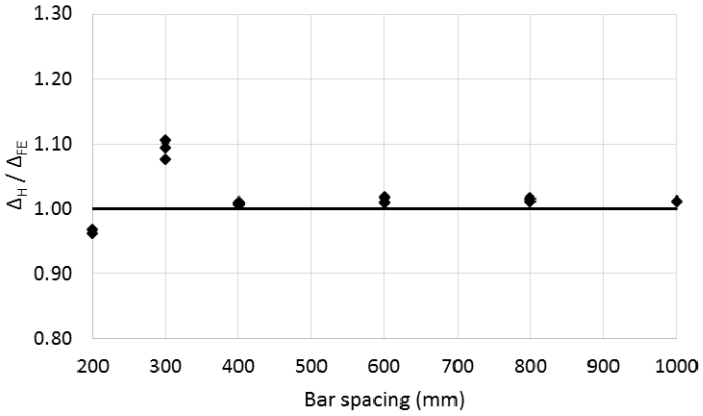

Figure 2. Calculated deflection Δ_H of geometry of Figure 1 against FE results.

Equations 4 to 6 are used for a panel of unit width acted on by a unit shear force ($V = 1$ kN). It follows that for the general case of a simply supported panel of width b, under uniform transverse loading, the maximum mid-span shear deflection is given by:

$$\delta_{v,\max} = \frac{wL^2}{8G_{eff}A_v} \tag{7}$$

where A_v is the shear area $= bh_c$.

Calculated values were compared with FE results. The plates and the bars were modelled using solid elements. The results are presented in Table 1.

Table 1: Comparisons of panel deflection (due to shear effects) obtained from the proposed method (Equation 7) against FE results.

Bar spacing s (mm)	Bar diameter φ (mm)	Plate thickness t_p (mm)	Spacing between plates h_c (mm)	Span L (mm)	$\delta_{v,\max} / \delta_{FE}$
400	16	8	300	4000	1.01
400	16	10	400	4000	1.09
400	16	12	400	4000	1.20
400	20	12	400	4000	1.02
400	20	12	600	4000	1.09
600	20	8	600	6000	1.15
600	20	10	800	6000	1.08
600	20	10	800	6000	1.11
600	20	12	800	6000	1.18
800	20	8	400	8000	1.28
800	20	10	600	8000	1.14
800	20	15	800	8000	1.28
1000	20	10	600	10000	1.19
1000	20	12	800	10000	1.23

Stresses due to shear

The maximum bar stress due to shear occurs at the ends of the panel (bar nearest to a support) and is given by:

$$\sigma_{bar} = \frac{32M}{\pi\varphi^3} \tag{8}$$

The moment M applied at the junction between the plate and the bar is obtained from:

$$M = (k_v \delta_v) s_1 / 2 \tag{9}$$

where

δ_v is the shear deflection midway between the first and second bars from the panel end, given by:

$$\delta_v = \frac{w s_1 (L - s_1)}{2 G_{eff} A_v} \tag{10}$$

The stress in the plate at the point of connection with a bar is given by:

$$\sigma_{v,pl} = \frac{M}{0.5 s t_p^2} K_{ps} \tag{11}$$

Equation (11) is derived by Young and Budynas (2002) for a circular plate with a simply supported edge loaded at its centre by a load acting over an area of a finite diameter. K_{ps} is a factor which depends on the ratio of the diameter of the loaded area to the diameter of the plate. In the present case, the loaded area is represented by the bar diameter, φ, and the plate diameter by the bar spacing, s, assuming bar spacing is equal in the two orthogonal directions.

Timoshenko and Woinowsky-Krieger (1987) presented expressions for calculating the moment, M_r, at any point in a circular plate in terms of the deflection, w:

$$M_r = -D\left(\frac{\partial^2 w}{\partial x^2} + v\frac{\partial^2 w}{\partial y^2}\right)_{\theta=0} = -D\left[\frac{\partial^2 w}{\partial r^2} + v\left(\frac{1}{r}\frac{\partial w}{\partial r} + \frac{1}{r^2}\frac{\partial^2 w}{\partial \theta^2}\right)\right] \tag{12}$$

where r and θ are polar coordinates, v is the Poisson's ratio of steel and D is the plate stiffness ($= EI_p$).

The deflection w is given by the relationship:

$$\begin{aligned} w = \frac{M(s/2)}{8\pi D[(3+v)+(1-v)\beta^4]} \{ & -[(1+v)+(1-v)\beta^2]\rho^3 + (1+v)(1-\beta^2)\rho + \\ & + 2[(3+v)+(1-v)\beta^4]\rho \ln\rho - \beta^2[(1+v)\beta^2 - (3+v)]\rho^{-1}\} \cos\theta \end{aligned} \tag{13}$$

where $\beta = \varphi / s$ and $\rho = 2r / s$.

Differentiating Equation 13 and substituting in Equation 12 gives M_r as a function of M. The stress magnification factor, K_{ps}, is then given by:

$$K_{ps} = \frac{6 M_r}{M} 0.5 s \tag{14}$$

K_{ps} is given in Table 2 for a range of values of the ratio of the bar diameter φ to the bar spacing s. For intermediate values of the ratio φ / s not included in Table 2, the value corresponding to the immediately lower φ / s ratio can be conservatively used.

Table 2: Plate stress magnification factor at the connection with the bars.

φ / s	Stress magnification factor K_{ps}
0.02	47.7
0.03	31.9
0.04	23.8
0.05	19.1
0.06	15.9
0.07	13.6
0.08	11.9
0.09	10.5
0.1	9.5

The bar and plate stresses calculated using the above expressions are for the support conditions shown in Figure 3. The preferred support position would be aligned with the bars, though this may not always be practical to achieve onsite. However, provided that both plates are supported and the distance of the first bar from the point of support is not greater than half the bar spacing, the expressions are valid. Furthermore, the stress in the bars calculated from Equation 8 was compared with that determined from the method presented in the Bi-Steel Design Guide (Corus, 2003) for a range of geometries and was found to be within 3%.

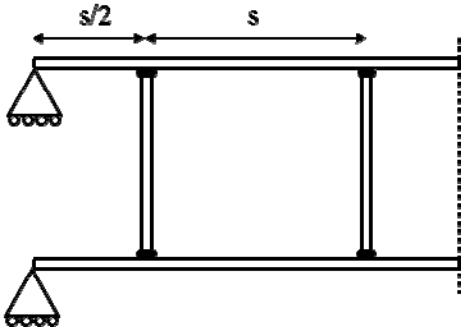

Figure 3. Assumed support conditions at panel ends.

DESIGN OF UNFILLED PANELS FOR AXIAL LOADS

Global panel buckling

The buckling resistance of the unfilled panel is assessed using EN 1993-1-1 for built-up compression members. An additionally applied moment due to initial imperfections in such members is considered. This is given by:

$$\Delta M = \frac{N_{Ed} e_0 + M_{Ed}^I}{\left(1 - \frac{N_{Ed}}{N_{cr}}\right)} \tag{15}$$

and the associated shear force by:

$$\Delta V = \frac{\pi}{L} \Delta M \tag{16}$$

where

N_{Ed} is the design axial load;
e_0 is an imperfection taken as $L/500$;
M_{Ed}^I is the design value of the maximum moment in the middle of the panel without considering second order effects;
L is the length of the panel;
N_{cr} is the elastic critical buckling load.

The elastic critical buckling load N_{cr} can be calculated from:

$$N_{cr} = \frac{\pi^2 E I_{eff}}{(aL)^2} \tag{17}$$

where a is a parameter accounting for end conditions and the effective second moment of area for the panel is obtained from:

$$I_{eff} = \frac{I_\alpha}{\left(1 + \frac{\pi^2 E}{G_{eff}} \frac{(h_c + t_p)^2 t_p}{2 h_c L^2}\right)} \tag{18}$$

The analytical results obtained using Equation 17 are compared with FE elastic buckling analysis results. Pinned end conditions are assumed, i.e. $a = 1$. The comparison is presented in Table 3.

The deflection of the panel and the stresses in the bar and the plate are calculated using Equation 1 and Equations 8 and 11, respectively. Figure 4 presents results in terms of a reduction factor to the axial compressive resistance of the panel. Since the proposed method is stress based, the buckling resistance is governed by the yield strength of either the bar or the plate material. In Figure 4, a yield strength of 350 N/mm^2 and 355 N/mm^2 was assumed for the bars and the plates, respectively. The results from the method presented in the Bi-Steel Design Guide are also plotted, as well as the buckling reduction factor as obtained from EN 1993-1-1 (using buckling curves a-d). The proposed method agrees well with the Bi-Steel method.

Table 3: Elastic critical buckling load calculated using the proposed method compared against values obtained from elastic (eigenvalue) buckling analyses.

Bar spacing s (mm)	Plate thickness t_p (mm)	Spacing between plates h_c (mm)	Length L (mm)	N_{cr} (kN)	
				Analytical	FE
400	8	400	3200	160.0	163.3
400	8	300	3200	191.1	192.1
400	10	500	3200	169.7	171.9
600	10	600	4800	90.4	97.4
600	8	500	4800	79.3	90.6
600	10	800	4800	70.9	78.5
800	10	800	6400	50.7	57.5
800	10	600	6400	63.8	71.0
800	12	1000	6400	45.9	55.2

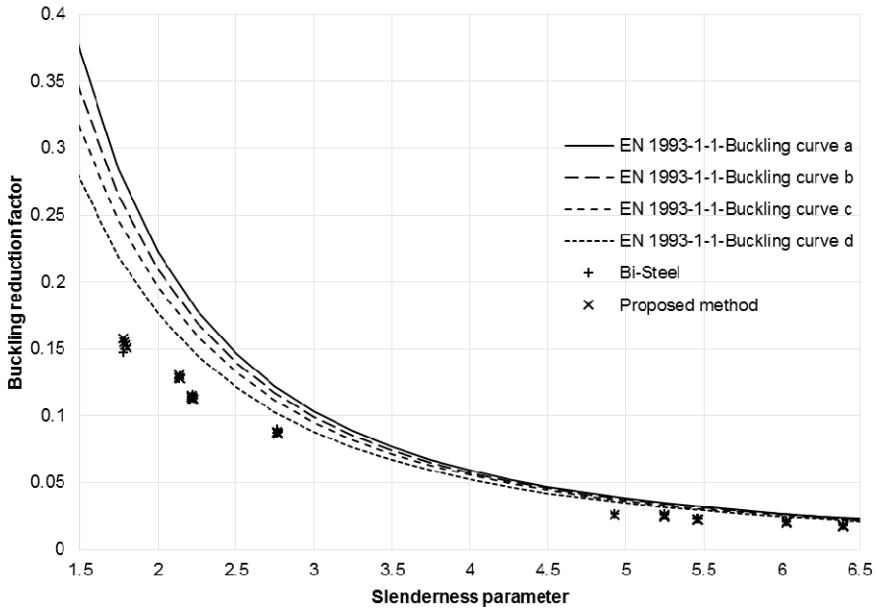

Figure 4. Reduction factor to panel's axial resistance accounting for global buckling.

Local plate buckling

The buckling reduction factor to the plate's axial resistance is obtained in accordance with EN 1993-1-5 using buckling curve (d) of EN 1993-1-1 (the plate is treated as an equivalent strut with pinned ends for the calculation of N_{cr}), based on the results of nonlinear FE analyses using a local model

of the plates attached to four bars. An imperfection due to manufacturing tolerances equal to 1/10 of the plate thickness was applied midway between the two rows of bars (single curvature bending shape). This imperfection is further increased by an additional imperfection equal to $\delta_{v,\lim}/8$ calculated using Equation 10 for a load, w, corresponding to the value of ΔM (Equation 15) which would cause yield in either the plate or the bar.

This additional imperfection accounts for the plate's sway as a Vierendeel truss under axial loading. The imperfection is taken at a distance of $s/4$ from the bar (point of maximum deviation from a straight line connecting the bar ends), hence $e_1 = \delta_{v,\lim}/8$ by linear interpolation (see Figure 5). This is conservative since it is assumed that the panel will be loaded up to the point that its global buckling resistance will be reached. The buckling reduction factor to the plate's axial resistance calculated from the FE results are presented in Figure 6, along with values from EN 1993-1-5 (for buckling curves a-d of EN 1993-1-1) and the Perry-Robertson formula which is adopted by the Bi-Steel Design Guide.

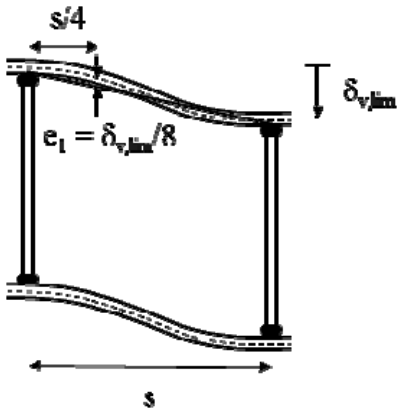

Figure 5. Imperfection arising from sway displacement.

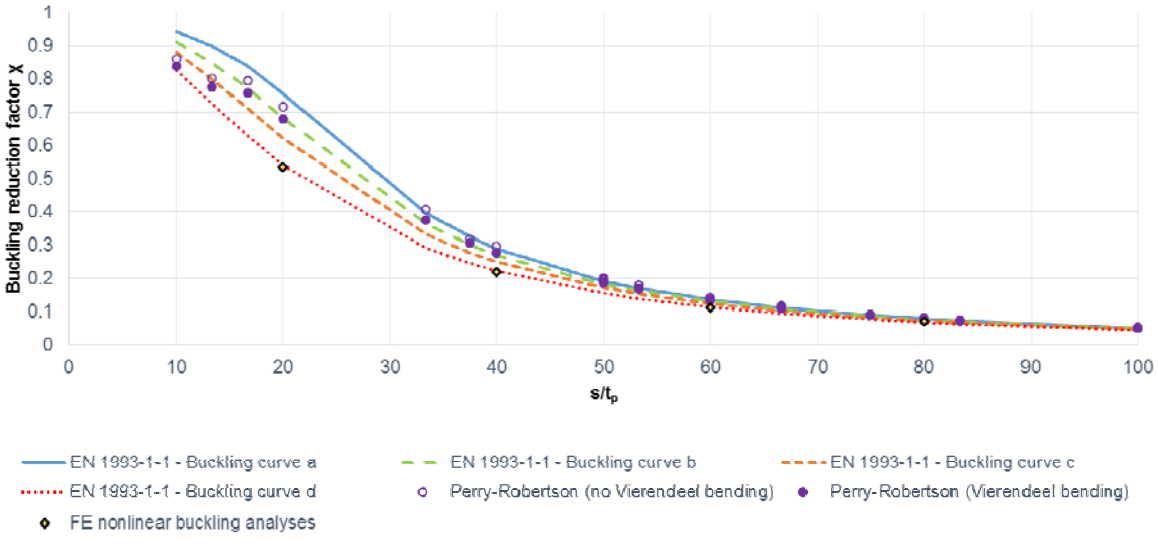

Figure 6. Reduction factor to plate's axial resistance accounting for local plate buckling.

DESIGN FOR CONCRETING

Calculation of fresh concrete pressures

When concrete is placed to a given height within the period required for its initial set (called rapid placement), the pressure acting on the plates is comparable to full liquid head (full fluid or hydrostatic pressure). With slower rate of placement, concrete at the bottom begins to harden and lateral pressure is reduced to less than full fluid pressure. The effective lateral pressure which is a modified hydrostatic pressure is influenced by the height, rate of placement, temperature of concrete mix (which affects setting time), use of retarders and other chemical and mineral admixtures, vibration (type, extent and depth) used to consolidate the fresh concrete. Other factors include the cement type, aggregate size, density of reinforcement (in SC, this is mainly the tie bars and stud connectors), concrete consistency, ambient temperature etc. Gardner (1985) additionally cites concrete density and method of placement (pouring from top will generally lead to lower pressures than pumping from the base). It should also be noted that flowable and self-compacting concrete (SCC) will normally exert higher pressures on the plates.

EN 13670 (2009) which covers the execution of concrete structures refers to EN 12812 (2004) performance requirements and general design guidance for falsework. EN 12812 specifies that lateral pressures from unset concrete shall be calculated using DIN 18218 (2010), CIRIA 108 (Clear and Harrison, 1985) or Manual de Technologie - Coffrage (CIB-FIB-CEB 27-98-83).

A comparison between the fresh concrete pressures calculated from CIRIA 108 and DIN 18218 for various concrete types is presented in Figure 7. CIRIA 108, although dated, has been successfully used for many years and was found to give conservative results, even for cases that are not explicitly covered, e.g. McCarthy et al. (2012) demonstrated this experimentally for SCC. However, the latter may not always be the case, i.e. for certain combinations of highly flowable concrete mixes with long setting times (such as SCC) and a high rate of placement, as demonstrated in Figure 7. Therefore, for the majority of cases, the designer can calculate the fresh concrete pressure using CIRIA 108. In some cases, e.g. use of advanced superplasticizers and other chemical products which, in combination with vibration techniques, increase the flowability and setting time of concrete considerably, the designers should use their discretion as to whether site tests are required to obtain relevant parameters that would allow the use of DIN 18218. It is also noted that both methods assume that concrete is poured from the top.

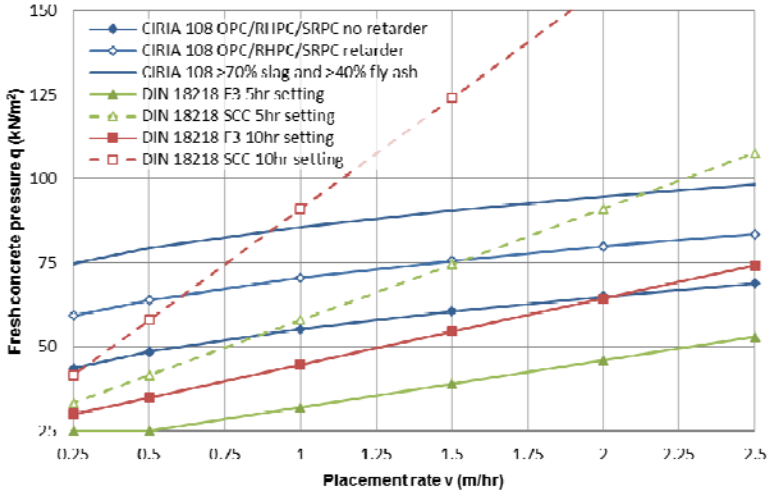

Figure 7. Typical comparisons between fresh concrete pressures calculated from CIRIA 108 and DIN 18218 for different concrete mixes and rates of placement.

Maximum fresh concrete pressure

Pressure exerted on the plates of vertical wall panels from fresh concrete causes double curvature bending of the plates, which are held together by tie bars at discrete locations. An effective width of plate is assumed as shown in Figure 8. This is different for positive (B_{eff}^{+}) and negative (B_{eff}^{-}) bending and may be taken as $s_1/4$ and $s_1/8$, respectively. The latter should not exceed a limiting value of $10t_p$.

The limiting (maximum) fresh concrete pressure that can be resisted is obtained from:

$$q_{\max} = \frac{1.6(B_{eff}^{+} + B_{eff}^{-})t_p^{\,2} f_y}{\gamma_f s_1^{\,2} s_2} \tag{19}$$

where γ_f is the partial factor for fresh concrete pressure taken as 1.5 in accordance with EN 12812 (variable action).

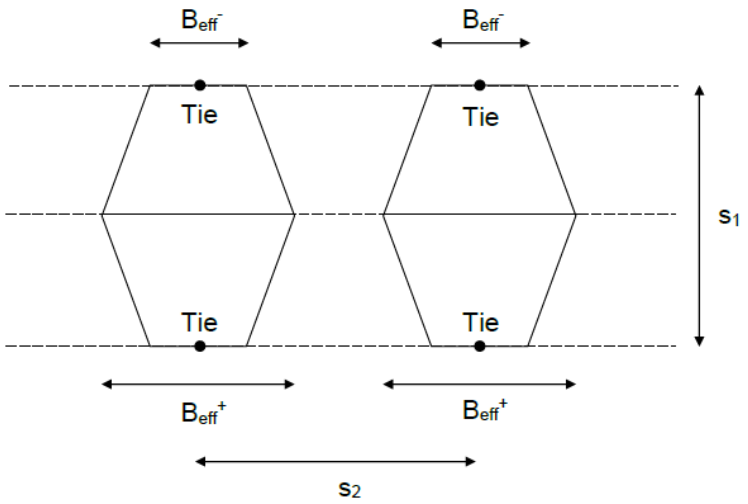

Figure 8. Effective width of plate in bending under pressure from fresh concrete.

The bending deflection of the plates can be obtained from:

$$\delta_b = \frac{q s_1^{\,4} s_2}{384 E I_p} \le \min\left(\frac{s_1}{200}, \frac{t_p}{3}\right) \tag{20}$$

The tensile stress in the tie bars is given by:

$$\sigma_{tpb} = \frac{4\gamma_f q s_1 s_2}{\pi \varphi^2} \tag{21}$$

CONCLUSIONS

A methodology for calculating the stresses and displacements in SC panels at the construction stage has been developed. Analytical expressions have been derived and validated using FE analyses. The methodology presented enables designers to account for the construction stage actions and to determine such aspects as the maximum length of members suitable for lifting, maximum height for stacking panels, maximum height of fresh concrete and placement rates for concreting operations.

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