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CRACK CONTROL IN RC TANK WALLS ACCORDING TO PN-EN 1992-3

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Abstract

The utility and appropriateness of PN-EN 1992-3 [8] codes of practice concerning the control of cracking occurring in RC tank walls with the use of a simplified method is verified. A special focus is given to the control of cracking induced by thermal effects. Moreover, the paper presents vital issues not mentioned in PN-EN 1992-3[8], i.e. the influence of concrete cover thickness, concreting conditions and non-uniform self-equilibrating stresses. It proposes relationships and corrective values, with the use of which values φ_s and s_z, appropriate for cases other than those defined in PN-EN 1992-3[8], can be established. It is shown that code charts 7.103N and 7.104N concerning φ_s^* and s_z^*, due to code assumptions, have a very limited range of practical use.

Keywords: codes of practice, maximum bar diameter, imposed loads

Streszczenie

W artykule zweryfikowano przydatność i poprawność wytycznych PN-EN 1992-3 [8] w zakresie kontroli zarysowania ścian zbiorników żelbetowych metodą uproszczoną. Szczególną uwagę poświęcono kontroli zarysowania od obciążeń termicznych. Ponadto w artykule wskazano na rzeczy istotne, lecz nieuwzględnione w PN-EN 1992-3 [8], tj. wpływ grubości otulenia, warunków betonowania oraz naprężeń własnych. Zaproponowano zależności oraz wartości korekcyjne, dzięki którym można wyznaczyć wielkości φ_s oraz s_z właściwe dla przypadków innych niż zdefiniowano w PN-EN 1992-3 [8]. Wykazano, że wykresy normowe 7.103N oraz 7.104N dotyczące odpowiednio φ_s^* oraz s_z^* ze względu na przyjęte założenia normowe mają bardzo ograniczony zakres praktycznego stosowania.

Słowa kluczowe: zastosowanie norm, maksymalna średnica pręt, obciążenie wymuszone

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1. Introduction

RC tanks are designed for the period of a hundred years. Tanks must fulfil all the requirements concerning their resistance, stability and construction durability, so as to avoid extra costs of their maintenance during the whole exploitation period. One of the major factors causing the decrease of durability of RC tanks is excessive wall cracking. Due to high diversity of RC tank construction methods and their various functions, which was shown inter alia by Halicka and Franczak [10], a way of fulfilling the watertightness condition, if other solutions (e.g. prestressing or liners) are not implemented, is to limit crack width. As postulated by Lewinski [18], the cracking criterion shall be verified for various calculation situations determined by the characteristics of a given RC tank. The code PN-EN 1992-3 [8] defines an acceptable crack width in RC liquid retaining tank walls, depending on a required watertightness class. On the basis of the codes of practice (PN-EN 1992-3 [8]), Halicka and Jabłoński [11] presented the influence of the watertightness class on the amount of required reinforcement in rectangular RC tank walls subject to various external loads.

PN-EN 1992-3 [8] additionally states: "Special care should be taken where members are subject to tensile stresses due to the restraint of shrinkage or thermal movements". In the case of semi-massive structures, where semi-massive RC tank walls belong, there is a phenomenon of strain restraint through formerly erected structural members, i.e. foundation slabs and wall segments. The influence of external restraints on the degree of cracking was presented by Petterson et al. [20]. It was shown that cracks occur at the place of temperature profile change from linear into constant. The code regulation also concerns cracks occurring at the stage of construction, which is then subject to significant temperature changes from:

- development of hydration heat (e.g. Flaga et al. [9]) – shows the ways of defining stresses and cracks caused by thermal effects in the period of concrete maturation),
- solar radiation (Buczkowski et al. [6]) – analyses cracks in the tank cover, showing a significant influence of solar radiation),
- daily ambient temperature changes generating, at the time of tank building (or exploitation), additional tensile forces combined with bending. In extreme cases, together with external loads, it may lead to shell damage, which was shown by Prusiel [21].

An extreme case of thermal load is a fire, taken into account only in the case of combustible material storage. Such structures are usually equipped with anti-fire installations. Błaszczyński et al. [5] present and analyse the effects of a fire, unpredicted at the design stage, spreading to a part of the cylindrical RC tank wall. Special emphasis was put on more intense cracking of cooler surfaces and a very important role of vertical reinforcement in the areas where usually there is only a membrane state.

It should be emphasised that, in a general case, focus should be placed both on strains developed at the early stage of concrete maturing, at its low mechanical parameters, and on those developed at the exploitation period with 28-day parameters, as further temperature decrease later on and growing drying shrinkage might cause crack widening and the formation of new cracks. Due to long term loading, it is also necessary to take into account concrete creep through the reduction of the modulus of elasticity and the resulting reinforcement stress changes, which was shown in the research monitoring structure's behaviour by Bednarski et al. [2, 3]. Due to a large number of factors affecting the imposed strains from: the conditions of structure erection, the type of concrete mix, structural solutions and rheological phenomena, there are no accurate guidelines concerning the way semi-massive RC tanks should be designed and erected (PN-EN 1992-3 [8]). The advanced numerical model, taking into account the majority of the factors playing an important role in the analysis of cracks caused by imposed strains, was presented by, for example, Knoppik-Wróbel and Klemczak [17]. On the other hand, on the basis of the realisation of massive structures, Kmita et al. [14] suggest that monitoring the realisation of massive or semi-massive concrete structures should also include the stage of concrete mix design and examination of concrete samples, for concrete mechanical and physical properties, which are a basis for specifying thermal strains, as shown by Nannan et al. [19] and Tayade et al. [27]. Generally, the issue of cracking in semi-massive structures in the aspects of concrete mix design, reinforcement design and the manner of object realisation was presented by Kiernożycki [13]. So far, there have been relatively few researches into this field, conducted on real objects. An example is the research presented by Seruga and Zych [23], [24], which includes detailed research results concerning cracks (and their development in time) in semi-massive RC tank walls, occurring at the stage of concrete maturing. Thermal influence leading to cracks was the result of both hydration heat development, daily ambient temperature changes, solar radiation and strain restraint. It was shown that cracking begins at the height of about 1 m, where, due to the temperature profile and the way the member is joined with the adjacent one, the restrained strain is the biggest.

According to PN-EN 1992-3 [8], in order to fulfil the condition of watertightness related to crack width control, appropriate reinforcement must be calculated. Both methods are acceptable: the accurate method consisting in calculating crack width, and the simplified one without direct calculations. The limitations of the simplified method are mentioned inter alia by Beeby and Narayanan [4] and a modification of this method accounting for these limitations was presented by Zych [29]. An alternative solution is the use of interior lining at the level of which watertightness is maintained, while the external crack width is calculated depending on the exposure class. An equally effective, but definitely more costly solution, is the use of internal cooling, which prevents wall temperature growth at the stage of its construction (Azenha et al., [1]). Thus, maturing concrete is not subject to intensive cooling and tensile strain. Obviously, the most effective method is the elimination of cracking effect or prevention of its formation through prestressing in stages, which can be performed in the case of both cylindrical and rectangular tanks (Seruga and Szydłowski [25]).

2. Maximum diameter of a reinforcement bar according to PN-EN 1992-3

When it is necessary to limit crack width for various values of $w_{\lim}$, it is also necessary to limit the maximum diameter of reinforcement bar φ_s to various degrees. Due to the increase in the area of the joint between reinforcement steel and concrete, there is growing concrete strain restraint along the bars.

In the case of both external load and imposed strains, and regardless of the type of restraint, the maximum diameter of the reinforcement bar φ_s^* is defined in Figure 7.103N (PN-EN 1992-3 [8]) as a function of reinforcement steel stresses and the acceptable crack width. In the case of imposed strains, there are two types of wall restraint, i.e. at its opposite ends and along its bottom edge. Figure 7.103N (PN-EN 1992-3 [8]) was drawn on the basis of the following formula:

$$\varphi_s^* = 8.529 \cdot f\left(w_k, \sigma_s\right) \cdot 2 \tag{1}$$

$$f\left(w_k, \sigma_s\right) = \frac{1}{\sigma_s}\left(\frac{E_s w_k}{W} - 3.4 \cdot 30 \text{ mm}\right) \tag{2}$$

where

- φ_s^* – the adjusted maximum bar diameter,
- W – simplified value $0.6\sigma_s$,
- σ_s – the reinforcement stress, which according to PN-EN 1992-3 [8] in order to define appropriate reinforcement layout can be calculated with the formula $\sigma_s = k_c k f_{ct,\text{eff}}/\rho$,
- E_s – 200 GPa,
- w_k – the maximum crack widths.

Eq. 1 was derived from Eqs. 3–5 below formulated in PN-EN 1992-1-1 [7] and used to define crack width with the accurate method. The derivation takes into account axial tension resulting from external loads and not the imposed ones, taking $k_c = k_2 = 1{,}0$, $h_{cr} = h$ and $a < 0{,}2h$. Derivation of Eq. 3 was presented, inter alia, by Beeby and Narayanan [4] and Knauff [15]:

$$w_k = s_{r,\max} \cdot \left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{3}$$

where:

- $s_{r,\max}$ – the maximum crack spacing,
- $(\varepsilon_{sm} - \varepsilon_{cm})$ – the difference between average strain in steel and concrete.

$$s_{r,\max} = 3.4 \cdot c_{\text{nom}} + 0.425 \cdot k_1 \cdot k_2 \cdot \varphi / \rho_{p,\text{eff}} \tag{4}$$

where:

- c_{nom} – the concrete cover to the longitudinal reinforcement,
- k_1 – a coefficient which takes account of the bond properties of the reinforcement = 0.8 for high bond bars,
- k_2 – a coefficient which takes account of the distribution of strain = 0.5 for bending and 1.0 for pure tension,
- φ – the bar diameter,
- $\rho_{p,\text{eff}}$ – the effective degree of reinforcement = $A_s/A_{c,\text{eff}}$, in which for pure tension $A_{c,\text{eff}} = \min(h/2; 2.5(h\text{-}d))$.

$$\varepsilon_{sm} - \varepsilon_{cm} = \left[\max\left[\sigma_s - k_t \cdot \frac{f_{ct,\mathrm{eff}}}{\rho_{p,\mathrm{eff}}} \cdot \left(1 + \alpha_e \cdot \rho_{p,\mathrm{eff}}\right) \right] \cdot E_s^{-1}; 0.6\sigma_s \cdot E_s^{-1} \right] \tag{5}$$

where:

k_t – a factor dependent on the duration of the load = 0.4 for long term loading,

$f_{ct,\mathrm{eff}}$ – the effective tensile strength of the concrete at the time when the cracks may first be expected to occur,

α_e – the ratio E_s/E_{cm}.

Crack spacing defined with Eq. 4 is the function of reinforcement and the type of external load. However, it does not take into account the type of wall strain restraint. The models concerning wall cracking due to imposed strain, presented by Stoffers [26], Rostásy and Henning [22] and Iványi [12], arbitrarily assume significantly bigger crack spacing, equalling (1,0-1,5)H, where H – wall height.

In determining the relationships 1 and 2, a simplifying assumption was made concerning the difference between average stresses in concrete and steel. This value is not a function of concrete strength and the degree of its reinforcement, but it is a value dependent only on steel stresses $W = 0.6\sigma_s$. What is more, the graph was plotted assuming that c_{nom} = 30 mm, unlike in PN-EN 1992-1-1 [7], where this value was 25 mm. This change is significant as together with increasing the concrete cover thickness c_{nom} (which is characteristic of tanks) the condition of maximum steel stress (see Fig. 1) is made stricter.

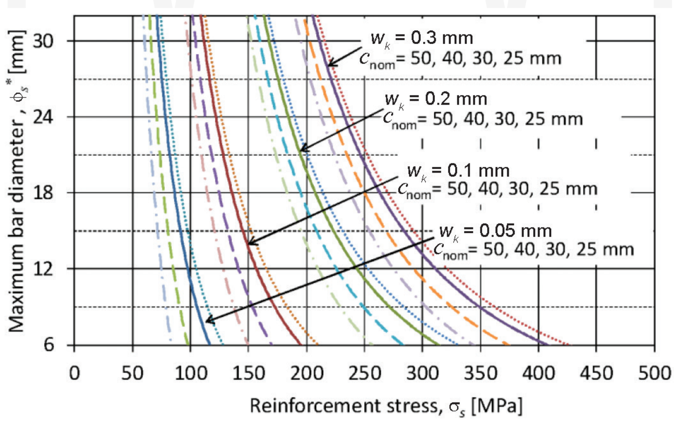

Fig. 1 Maximum bar diameter for cracks control in members subjected to axial tension as a function of: stresses, maximum crack width and the concrete cover thickness, according to Eqs. 1 and 2

Moreover, PN-EN 1992-3 [8] reads as follows: “For cracking caused dominantly by restraint, the bar sizes given in Fig. 7.103N **should not be exceeded** where the steel stress is the value obtained immediately after cracking”. Furthermore, “For cracking caused **dominantly by loading**, either the maximum bar sizes from Fig. 7.103N or the maximum bar spacing from Fig. 7.104N may be complied with”. According to the quoted code, it should be concluded that Figure 1 defines φ_s^* for RC wall sections with a minimum degree of reinforcement, which are subject to **external forces** or **imposed strain**. Thus, when the simplified method of crack control is employed, in the two completely different load cases, the relationships defining φ_s^* in PN-EN 1992-3 [8] are identical.

3. Members restrained at opposite ends

Taking into account the guideline in the annex *M* of the code in question, saying that in case of members restrained at opposite ends, the equation for $(\varepsilon_{sm}-\varepsilon_{cm})$ shall take the form of Eq. 6, it should be assumed that the relationship for φ_s^* (Eq. 1) should be defined taking into account:

$$\varepsilon_{sm}-\varepsilon_{cm}=\left(0.5\cdot\alpha_e\cdot k_c\cdot k\cdot f_{ct,\mathrm{eff}}\left(t\right)/E_s\right)\cdot\left(1+1/\alpha_e\cdot\rho\right) \tag{6}$$

where

$k_t - k_c = 1.0$ for pure tension,
k – the coefficient which allows for the effect of non-uniform self-equilibrating stresses,
ρ – the degree of reinforcement.

Thus, in the case of imposed strain, the simplifying assumption $W = 0.6\sigma_s$ in Eq. 2 should be substituted with:

$$W=0.5\cdot\alpha_e\cdot k_c\cdot k\cdot f_{ct,\mathrm{eff}}\cdot\left(1+1/\alpha_e\cdot\rho\right) \tag{7}$$

Following Knauff [16], for a random case of axial tension, where $a < 0.2h$, $\rho_{\mathrm{eff}} = 0.5A_s/2.5ab = 0.2\rho h/a$ and assuming, as in PN-EN 1992-1-1 [7] the simplification that $a = (h-d) \approx 0.1h$ is obtained $\rho_{\mathrm{eff}} = 2\rho$. Next, taking the formula according to Knauff [16]:

$$\rho_{\mathrm{eff}}=r\frac{k_c\cdot k\cdot f_{ct,\mathrm{eff}}\cdot h_{cr}}{h\cdot\sigma_s}=0.2\rho\cdot h/a\text{, for } a<0{,}2\ h \tag{8}$$

where:

$r = 0{,}2k_c h_{cr}/a$, after the simplification (for members with axial tension, from Eq. 8), $\rho = f_{ct,\mathrm{eff}}/\sigma_s$, or $\rho_{eff} = 2f_{ct,\mathrm{eff}}/\sigma_s$ is obtained.

Figure 2 presents the thickness of walls complying with the condition $a < 0.2h$, thus also in compliance with Eq. 8, in the function c_{nom} for $\varphi = 12$, 16 and 20 mm. For example, this condition is fulfilled for tank walls whose thickness equals $h > 25$ cm, reinforced with bars of $\varphi = 20$ mm in one layer and the concrete cover of $c_{\mathrm{nom}} = 40$ mm. Thus, this condition is fulfilled in the majority of cases of RC tank walls.

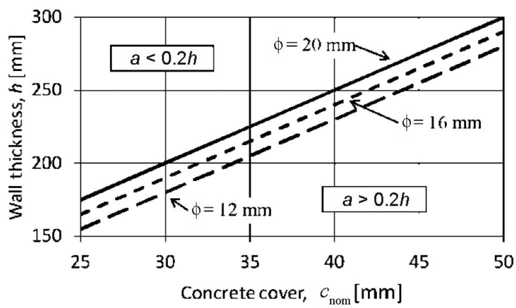

Fig. 2. The wall thickness *h* fulfilled the condition $a < 0.2\ h$

Therefore, substituting $\rho = f_{ct,\mathrm{eff}}/\sigma_s$ in Equation 7:

$$W = 0.5 \cdot \alpha_e \cdot k_c \cdot k \cdot f_{ct,\mathrm{eff}} \cdot \left(1 + \frac{\sigma_s}{\alpha_e \cdot f_{ct,\mathrm{eff}}}\right) \tag{9}$$

is obtained.

In the case of pure tension $k_c = 1.0$, Equation 9 is simplified in the following way:

$$W = 0.5 \cdot k \cdot \left(\sigma_s + \alpha_e \cdot f_{ct,\mathrm{eff}}\right) \tag{10}$$

On this basis, φ_s^* in the function of σ_s for particular crack widths w_k and c_{nom} was presented in Figure 3. From Figures 1 and 3, it can be concluded that the values of φ_s^* in both cases are similar. However, the values of φ_s^* read from the graph plotted for external loads are only slightly smaller in comparison with the case of imposed strain with $k = 1.0$. The application of coefficient $1.0 > k \geq 0.65$ as for the members of the section thickness of $300\ \mathrm{mm} < h \leq 800\ \mathrm{mm}$ contributes to the growth of φ_s^*. That is why, using the graph 7.103N in this case should be regarded as conservative. Nevertheless, the guideline in PN-EN 1992-3 [8] related to the possibility of reading a bar diameter from graph 7.103N for both the expected member cracking due to external load and imposed strain is justified. Therefore, using the simplified method contributes to defining a smaller reinforcement bar diameter, and, consequently, smaller crack width than would be obtained from the accurate method, having fulfilled Eq. 11. Otherwise, due to the increase of formula *W*, the simplified method may underestimate φ_s^*.

$$0.6\sigma_s \geq \sigma_s - k_t \cdot \frac{f_{ct,\mathrm{eff}}}{\rho_{p,\mathrm{eff}}} \cdot \left(1 + \alpha_e \cdot \rho_{p,\mathrm{eff}}\right) \tag{11}$$

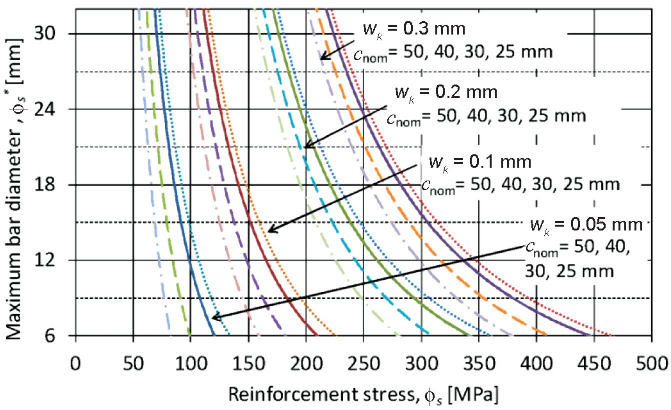

Fig. 3. Maximum bar diameter for cracks control in members restrained at the ends taking into account Eq. 10 and $k = 1.0$

4. Members restrained along the bottom edge

For a member restrained along the bottom edge, the formula for ($\varepsilon_{sm} - \varepsilon_{cm}$) was defined in the annex *M* for PN-EN 1992-3 [8] according to the formula:

$$\varepsilon_{sm} - \varepsilon_{cm} = R_{ax} \cdot \varepsilon_{\text{free}} \tag{12}$$

where:

- R_{ax} – the coefficient of external restraint,
- $\varepsilon_{\text{free}}$ – the strain which would occur if the element was completely unrestrained.

According to PN-EN 1992-3 [8] "the formation of a crack in this case only influences the distribution of stresses locally and the crack width is a function of the restrained strain rather than the tensile strain capacity of the concrete". Furthermore, taking $R_{ax} = 0.5$ and $\varepsilon_{\text{free}} = \Delta T \cdot \alpha_T$, (where: ΔT – average temperature change in the wall section, α_T – coefficient of concrete thermal expansion), formula *W* takes the form:

$$W = \left(R_{ax} \cdot \Delta T \cdot \alpha_T\right) E_s \tag{13}$$

Figure 4 presents φ_s^* as a function of σ_s and assumed ε_{free}, for example for $\Delta T = 40°C$. The graph φ_s^* obtained even for such a significant strain is not similar to the graph presented in PN-EN 1992-3 [8]. The results obtained should lead to the conclusion that the increased values of φ_s^* should be adopted in relation to the graph 7.103N. However, such a possibility is not recommended in PN-EN 1992-3 [8]. It also follows from the situation in which further cracking should occur later, accompanied by external load, when it is necessary to define σ_s and φ_s^* once again. Nevertheless, Eq. 4 defining the maximum crack spacing, which, according to PN-EN 1992-3 [8], is identical for both types of restraint, should be doubted. In case of the member restrained along the bottom edge, bigger crack spacing should be expected, which was proven by Zych [28]. That is why the obtained graph cannot be used, and according to the author, it only confirms the erroneous assumptions in the accurate method.

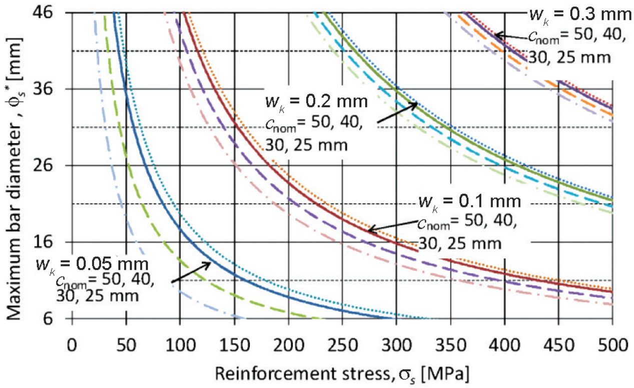

Fig 4. Maximum bar diameter for cracks control in members restrained along bottom edge taking into account Eq. 13 and ΔT = 40°C

5. Concreting conditions

Another factor significantly influencing the maximum diameter of a reinforcement bar are the bond conditions of concrete and steel, which result from bar location during concreting. Tank walls are usually divided into several-meter high segments. According to the guidelines presented by Kiernożycki [13] "Due to fresh concrete subsidence, the height of layers shall not exceed 2.5÷3.0 m". Engineering practice makes it possible to erect much higher concrete walls in one concreting cycle (Zych [28]). The definitions of favourable and unfavourable concreting conditions provided in PN-EN 1992-1-1 [7] do not include walls. However, the defined case of poor concreting conditions for reinforcement bars in the upper part of the member of the height of $h > 250$ mm can be a good justification for assuming poor concreting conditions also in walls. It concerns the case in which vertical cracks induced by imposed strain are analysed, which are at their maximum width at the height of about 1 meter from the wall foundation (Seruga and Zych [23, 24]). Thus, following PN-EN 1992-1-1 [7] and substituting coefficient $\eta_1 = 0,7$ in Eq. 4 the following is obtained:

$$s_{r,\max} = 3.4 \cdot c_{\text{nom}} + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / \rho_{p,\text{eff}} \cdot \eta_1 \tag{14}$$

where:

η_1 – a coefficient related to the quality of the bond condition and the position of the bar during concreting, η_1 = 1.0 for good conditions = 0.7, for all other cases.

On this basis, Eq. 1 takes the form:

$$\varphi_s^* = 8.529 \cdot \eta_1 \cdot f\left(w_k, \sigma_s\right) \cdot 2 \tag{15}$$

The relationship defined in this way results in Figure 5. This way, the influence of poor concreting conditions is reflected in the need to reduce the maximum diameter of the reinforcement bar by 30%, which is not mentioned in PN-EN 1992-3 [8]. It also explains the cases of leakage in the walls complying with the condition of minimal reinforcement degree (Zych [28]).

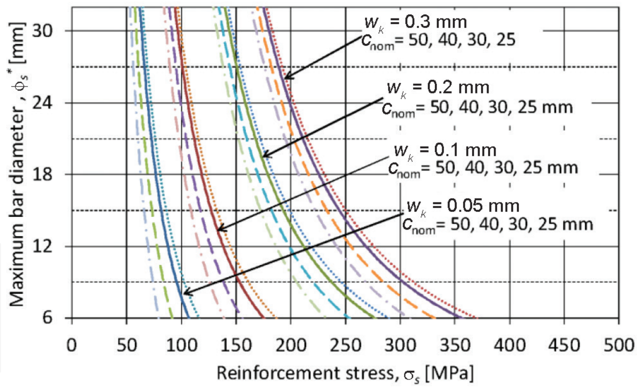

Fig. 5. Maximum bar diameter for cracks control in members subjected to axial tension as a function of: stresses, maximum crack width and the concrete cover thickness, taking into account poor bond conditions

6. Maximum diameter and reinforcement spacing

The maximum diameter of a reinforcement bar, due to $w_{\lim}$, can be written as follows (Knauff [15]):

$$\varphi_s\left(\sigma_s\right) = r\frac{k_c \cdot h_{cr}}{h} \cdot \frac{k \cdot f_{ct\,\mathrm{eff}}}{0.34 \cdot k_2 \cdot \sigma_s}\left(\frac{E_s \cdot w_{\lim}}{W\left(\sigma_s\right)} - 3.4 \cdot c_{\mathrm{nom}}\right) \tag{16}$$

taking $\varphi_s'^*$ as for 28-day concrete C30/37 in the form:

$$\varphi_s'^*\left(\sigma_s\right) = \frac{2.9\ \mathrm{MPa}}{0.34 \cdot k_2 \cdot \sigma_s}\left(\frac{E_s \cdot w_{\lim}}{W\left(\sigma_s\right)} - 3.4 \cdot c_{\mathrm{nom}}\right) \tag{17}$$

$\varphi_s^* = 2 \cdot \varphi_s'^*\left(\sigma_s\right)$ can be written, thus, for axial tension ($k_c = k_2 = 1$, $h_{cr} = h$) the following equation is obtained:

$$\varphi_s\left(\sigma_s\right) = r \cdot \varphi_s'^*\left(\sigma_s\right) \cdot \frac{k \cdot f_{ct\,\mathrm{eff}}}{2.9\ \mathrm{MPa}} \tag{18}$$

For the case of $a < 0.2\,h$ (Knauff [16]), where: $r = k_c \cdot h_{cr}/5a$ and after transformations, Eq. 19 is obtained:

$$\varphi_s(\sigma_s) = \frac{1}{5}\frac{h}{(h-d)}\cdot\varphi_s'^{*}(\sigma_s)\cdot\frac{k\cdot f_{ct\,\mathrm{eff}}}{2.9\ \mathrm{MPa}} = \frac{1}{10}\frac{h}{(h-d)}\cdot\varphi_s^{*}\cdot\frac{k\cdot f_{ct\,\mathrm{eff}}}{2.9\ \mathrm{MPa}} \tag{19}$$

In PN-EN 1992-3 [8], the formula for the correction of a maximum diameter of a reinforcement bar was written as:

$$\varphi_s(\sigma_s) = \frac{1}{10}\frac{h}{(h-d)}\cdot\varphi_s^{*}\cdot\frac{f_{ct\,\mathrm{eff}}}{2.9\ \mathrm{MPa}} \tag{20}$$

For the case of $a = 0{,}1h$ and $k{\cdot}f_{cteff} = 2.9$ MPa, $\varphi_s = \varphi_s^*$ is obtained. The coefficient k not included in PN-EN 1992-3 [8] influences the reduction of $A_{s,\min}$ and φ_s. It takes into account the effect of non-uniform self-equilibrating stresses occurring in the section before cracking on the reduction of the resultant force coming from the restrained strain by an external restraint.

For the case of $a \geq 0{,}2h$ where $r = 1$, Eq. 21 is obtained:

$$\varphi_s(\sigma_s) = \frac{1}{2}\cdot\varphi_s^{*}\cdot\frac{k\cdot f_{ct\,\mathrm{eff}}}{2.9\ \mathrm{MPa}} \tag{21}$$

PN-EN 1992-3 [8] specifies that "For cracking caused **dominantly by loading**, either the maximum bar sizes from Fig. 7.103N or the maximum bar spacing from Fig. 7.104N may be complied with". In case of imposed strain, the graph 7.103N can be used provided that "… the bar sizes given in Fig. 7.103N should not be exceeded where the steel stress is the value obtained immediately after cracking". This code does not contain any information about the possibility of using graph 7.104N (Fig. 6) in case of imposed strain, though graphs 7.103N and 7.104N are coupled. Figure 6 presents the maximum bar spacing obtained on the basis of the formula:

$$s_z^{*}(\sigma_s) = \frac{1\,\mathrm{m}\cdot 2\pi\left(\phi_s^{*}(\sigma_s)/2\right)^2}{A_{s,\min}(\sigma_s)} \tag{22}$$

where:

- $\varphi_s^*(\sigma_s)$ – for axial tension – according to the graph 7.103N (PN-EN 1992-3 [8]),
- $A_{s,\min}(\sigma_s)$ – is equal to $\rho_{p,\mathrm{eff}}(\sigma_s){\cdot}1\mathrm{m}{\cdot}2.5a{\cdot}2$, in which the effective depth of tensile layers of $2.5a$ for $a < 0{,}2\,h$,
- a – is equal to $c_{\mathrm{nom}} + \varphi_s^*(\sigma_s)/2$,
- $\rho_{p,\mathrm{eff}}(\sigma_s)$ – according to Eq. 23 obtained from substituting $s_{r,\max}$ and $(\varepsilon_{sm} - \varepsilon_{cm})$ in the formula for w_k and transformation due to $\rho_{p,\mathrm{eff}}$ and assuming the difference between average stresses in steel and concrete in the form of the formula $(\varepsilon_{sm} - \varepsilon_{cm}) = W(\sigma_s)/E_s$, where: $W(\sigma_s) = 0{,}6\,\sigma_s$.

$$\rho_{p,\text{eff}}\left(\sigma_s\right) = \frac{0.34 \cdot \phi_s^*\left(\sigma_s\right)}{\dfrac{E_s \cdot w_k}{W\left(\sigma_s\right)} - 3.4 \cdot c_{\text{nom}}} \tag{23}$$

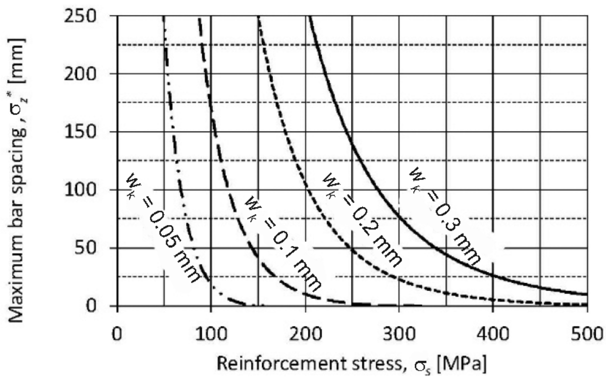

Fig. 6. Maximum bar spacing for concrete cover c_{nom} = 30 mm (PN-EN 1992-3 [8])

7. Correction of maximum reinforcement bar spacing s_z*

Maximum reinforcement bar spacing for particular crack width w_k defined according to Eq. 22 complies with the assumptions of the accurate method only when Eq. 24 is fulfilled. It is linked with the assumption of maximum crack spacing in the form of Eq. 4.

$$s_z^{\max}\left(\sigma_s\right) = 5 \cdot \left(c_{\text{nom}} + \frac{\varphi_s^*\left(\sigma_s\right)}{2}\right) \tag{24}$$

Figure 7 presents the maximum reinforcement bar spacing calculated for particular $c_{nom,}$ according to Eq. 22. In order to present clearly (further on in this paper) the deviation from the reinforcement bar spacing resulting from the change of c_{nom}, Eq. 24 was taken into account in separate calculations. In the case of tank walls with concrete cover thickness of c_{nom} = 40 and 50 mm, Eq. 22 is the decisive expression.

On the basis of Figure 7, the differences between maximum reinforcement bar spacing were determined for cases with various concrete cover thickness in comparison with the plots presented in Figure 6 (drawn for c_{nom} = 30 mm). The obtained results are illustrated in Figure 8. The values Δs_1 added to the values taken from Figure 6 are the maximum reinforcement bar spacing, including the thickness of concrete cover other than 30 mm, without the necessity of supplementary calculations. Although the simplified method renders only approximate results, the obtained values of Δs_1 significantly affect the need to reduce the maximum reinforcement bar spacing. For the majority of cases, the values Δs_1 are negative, which results from the concrete cover thickness greater than 30 mm.

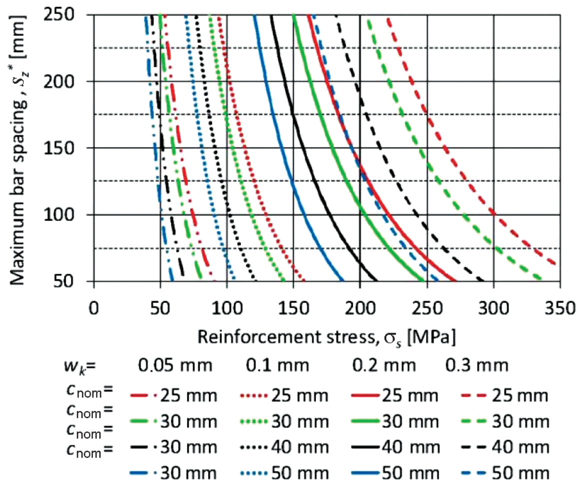

Fig 7. Maximum bar spacing for selected concrete cover c_{nom}

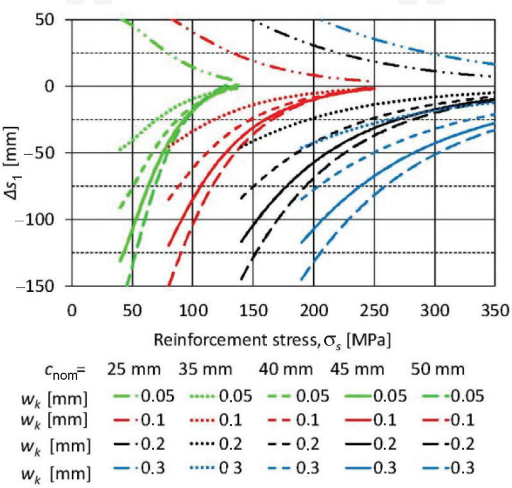

Fig. 8. Deviation Δs_1 for bar spacing for selected concrete cover c_{nom} in relation to c_{nom} = 30 mm (Fig. 6)

Figure 9, in turn, illustrates the influence of Eq. 24 on the reduction of maximum reinforcement bar spacing. The reduction of Δs_2 should be made for greater values of w_k in the case of concrete cover thickness of c_{nom} = 25 and 30 mm. In other cases, Eq. 24 is not vital.

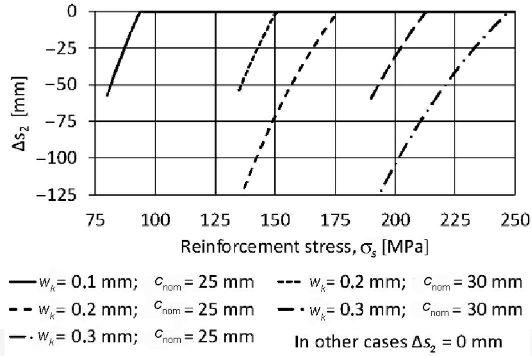

Fig. 9. Reduction of bar spacing taking into account Eq. 24 in relation to Fig. 6

8. Maximum reinforcement bar spacing s_z

The values of s_z^* taken from graph 7.104N of PN-EN 1992-3 [8] and corrected due to c_{nom} should be subject to further correction (analogically as performed for the case of φ_s^*) due to $f_{ct,\mathrm{eff}}$, d and k. PN-EN 1992-3 [8] does not provide any information about the necessity of this correction in case of s_z^*. The reinforcement bar spacing s_z can be directly defined from the value of φ_s according to Eq. 22. Figure 10 presents the influence of concrete class on s_z for the case of $a < 0.2\ h$ and $a/h = 0.1$. Moreover, the calculations assumed the concrete cover thickness $c_{\mathrm{nom}} = 40$ mm often used for RC tanks. The results obtained lead to the conclusion that there is a necessity of significant reduction in the maximum reinforcement bar spacing for all the cases considered. The values Δs_3 obtained indicate the biggest necessity of such reduction for lower class concrete (due to lower ultimate bond stress).

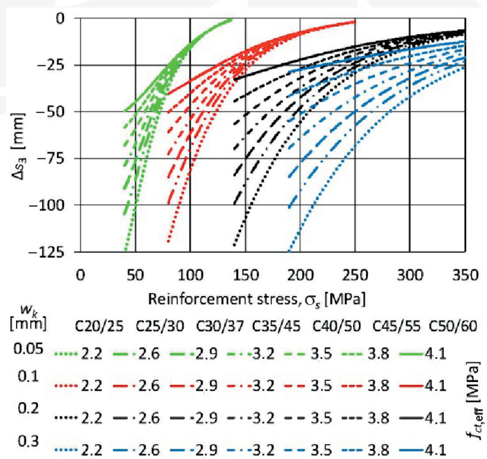

Fig. 10. Deviation Δs_3 of bar spacing (for: $t = 28$ day, concrete cover $c_{\mathrm{nom}} = 40$ mm and $a/h = 0.1$) in relation to Fig. 7.104N (PN-EN 1992-3 [8])

For walls thicker than 300 mm, according to PN-EN 1992-1-1 [7], the reduction of effective concrete tensile strength is recommended due to the occurrence of non-uniform self-equilibrating stresses leading to early structure cracking. Figure 11 presents the effect of coefficient k on the reinforcement bar spacing for particular crack widths w_k in the form of deviation Δs_4 defined in relation to Figure 7.104N.

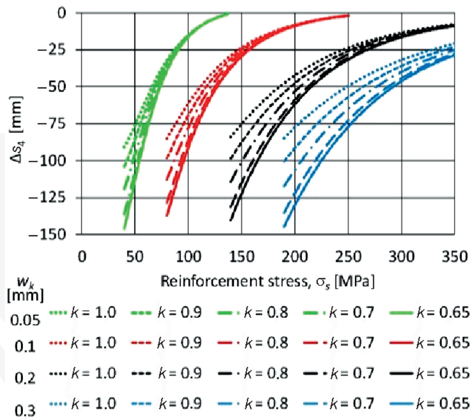

Fig. 11. Deviation Δs_4 of bar spacing for selected values k (for concrete cover c_{nom} = 40 mm and a/h = 0.10) in relation to Fig. 7.104N (PN-EN 1992-3 [8])

9. Conclusions

PN-EN 1992-3 [8] presents the simplified method of crack control for strain imposed cracks by defining the adequate minimum reinforcement area and the maximum diameter of reinforcement bars. The code does not mention the significant effect of concrete cover thickness on allowable stresses in steel. Neither does it give the value of c_{nom} for which graph 7.103N was plotted. In the case of tanks the bar concrete cover thickness used is definitely greater than in traditional structures.

Figure 7.103N in PN-EN 1992-3 [8] defines φ_s^* for RC wall sections loaded by external loads. However, the values of φ_s^* obtained from the graph for external loads are only slightly lower than in the case of imposed strain for k = 1.0. Therefore, the code recommendation about the possibility of reading the bar diameter from graph 7.103N for the case of expected member cracking, both from external load and imposed strain, is justified. It contributes to determine a smaller reinforcement bar diameter, and consequently a smaller crack width than it would be obtained from the accurate method, complying with Eq. 11.

In the case of a member restrained along one edge, the graph of φ_s^* should not be considered proper for application. In the author's opinion, the discrepancies might stem from the formula defining the maximum crack spacing, which despite a different character of cracking, in both cases of restraint, was adopted in the same form (PN-EN 1992-3 [8]).

Taking into account poor concreting conditions leads to the necessity of reducing the maximum reinforcement bar diameter by 30%.

The deviations Δs_1 of maximum reinforcement bar spacing (s_z^*) for cases different in concrete cover thickness in relation to the case shown in Figure 7.104N of PN-EN 1992-3 [8] point to the necessity of significant reduction of reinforcement bar spacing, together with increasing concrete cover thickness for all cases of w_{lim}.

The present paper proposes a graph of values decreasing the reinforcement bar spacing resulting from the assumption of bar spacing (Eqs. 4 and 24) in the accurate method after PN-EN 1992-1-1 [7]. These corrections are necessary only in the case of smaller concrete cover thickness (25 and 30 mm).

On the basis of the conducted analyses, it can be concluded that Figure 7.104N for maximum reinforcement bar spacing included in PN-EN 1992-3 [8] can be used when: the concrete class is not lower than C30/37, cracking occurs after the period of concrete maturation, wall thickness is not greater than 300 mm, the relation a/h is less or equal 0.1 and the concrete cover is not thicker than 30 mm. For some cases (c_{nom}, a/h, k and for concrete class), the author proposes, in a graphical form, the values of Δs_i reducing reinforcement bar spacing s_z defined in relation to graph 7.104N.

References

[1] Azenha M., Lameiras R., de Sousa Ch., Barros J., *Application of air cooled pipes for reduction of early age cracking risk in a massive RC wall*, "Engineering Structures", 62–63(14), 2014, 148–163.

[2] Bednarski Ł., Sieńko R., Howiacki T., *Estimation of the value and the variability of elastic modulus for concrete in existing structure on the basis of continous in situ measurements*, "Cement-Wapno-Beton", 81(6), 2014, 396–404.

[3] Bednarski Ł., Sieńko R., Howiacki T., *Analysis of rheological phenomena in reinforced concrete cross-section of Rędziński bridge pylon based on in situ measurements*, In Proceedings of 7th Scientific-Technical Conference Material Problems in Civil Engineering (MATBUD'2015), Procedia Engineering 108(6), 2015, 536–543.

[4] Beeby A.W., Narayanan R.S., *Designersą Guide to EN* 1992-1-1 *and EN* 1992-1-2 *Eurocode* 2*: Design of Concrete Structures. General rules and rules for buildings and structural fire design*, Thomas Telford, London 2005.

[5] Błaszczyński T., Babiak M., Wielentejczyk P., *Numerical methods advantage to the analysis of destructions caused by the fire of the silo for biomass (Wykorzystanie metod numerycznych do analizy zniszczeń wywołanych pożarem silosu na biomasę)* [in Polish], "Materiały Budowlane", 517(9), 2015, 42–43.

[6] Buczkowski W., Szymczyk-Graczyk A., Walczak Z., *Reasons of cracking of RC circular plates covering tanks for sewage (Przyczyny zarysowań żelbetowych płyt kołowych przekrywających zbiorniki na ścieki)* [in Polish], "Materiały Budowlane", 517(9), 2015, 63–64.

[7] CEN (European Committee for Standardization) PN-EN 1991-1:2004 Eurocode 2: Design of concrete structures–Part 1-1: General rules and rules for buildings, CEN, Brussels.

[8] CEN (European Committee for Standardization) PN-EN 1992–3:2006 Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containment structures, CEN, Brussels.

[9] Flaga K., Klemczak B., Knoppik-Wróbel A., *Early thermal-shrinkage cracks in the walls of bridge supports (Wczesne rysy termiczno-skurczowe w ścianach przyczółków mostowych)* [in Polish], "Inżynieria i Budownictwo", 2013(4), 2013, 197–200.

[10] Halicka A., Franczak D., *Design of RC tanks. Tanks for liquids (Projektowanie zbiorników żelbetowych. Zbiorniki na ciecze)* [in Polish], Vol. 2, Wydawnictwo Naukowe PWN, Warszawa 2013.

[11] Halicka A., Jabłoński Ł., *The influence of watertightness class on the amount of reinforcement area in the walls of prismatic liquid tanks* (*Wpływ klasy szczelności na zbrojenie ścian żelbetowych zbiorników prostopadłościennych)* [in Polish], "Materiały Budowlane", 517(9), 2015, 50–52.

[12] Iványi G., *Minimum reinforcement in the walls (Bemerkungen zu „Mindestbewehrung in Wänden)* [in German], "Beton- und Stahlbetonbau", 90(11): 1995, 283–289.

[13] Kiernożycki W., *Concrete massive structures (Betonowe konstrukcje masywne)* [in Polish], first ed., Polish Cement, Cracow 2003.

[14] Kmita A., Kostecki S., Logoń D., et al., *Executive problems concerning a construction of the dam in Niedów*, (Problemy wykonawcze związane z budową zapry betonowej w Niedowie [in Polish]). "*Materiały Budowlane"*, 517(9), 2015, 71–73.

[15] Knauff M., *Cracks, minimum reinforcement area. In Scientific commentary to PN-B-*03264*:*2002*, Concrete, RC and Prestressed Construction*, Vol. 2. Institute for Building Technology, Warszawa 2004, 47–76.

[16] Knauff M., *Serviceability limit states*. (*Stany graniczne użytkowalności*).In *Design of concrete and prestressed structures according to EC*2. (*Podstawy projektowani konstrukcji żelbetowych i sprężonych wg EC2)* [in Polish], Dolnośląskie Wydawnictwo Edukacyjne, Wrocław, 2006, 579–648.

[17] Knoppik-Wróbel A., Klemczak B., *Degree of restraint concept in analysis of early-age stresses in concrete walls*, "Engineering Structures", 102(11), 2015, 369–386.

[18] Lewiński P., *Principles for the design of RC tanks for liquids, taking into account the requirements of Eurocode* 2, (*Zasady projektowania zbiorników żelbetowych na ciecze z uwzględnieniem wymagań Eurokodu* 2*)* [in Polish], Institute for Building Technology, Warszawa, 2011.

[19] Nannan S., Jianshu O., Runxiao Z., Dahai H., *Experimental Study on Early-Age Crack of Mass Concrete under the Controlled Temperature History*, "Advances in Materials Science and Engineering", 2014, 1–10, Article ID 671795.

[20] Pettersson D., Alemo J., Thelandersson S., *Influence on crack development in concrete structures from imposed strains and varying boundary conditions*, *"Construction and Building Materials",* 16(4), 2002, 207–213.

[21] Prusiel J., *Analysis of thermal stresses in reinforcement concrete walls of the grain silo*. *(Analiza naprężeń termicznych w żelbetowych ścianach silosów na zboże)* [in Polish], "Materiały Budowlane", 517(9), 2015, 55–56.

[22] Rostásy F S., Henning W., *Imposed loads in walls on the foundation* (*Zwang in Stahlbetonwänden auf Fundamenten*) [in German], "Beton- und Stahlbetonbau", 84(8), 1989, 208–214.

[23] Seruga A., Zych M., *Thermal Cracking of the Cylindrical Tank under Construction. I*: *Case Study*, "ASCE Journal of Performance Construction Facilities", 29(4), 2015, 04014100–1–04014100–9.
[24] Seruga A., Zych M., *Research on Thermal Cracking of a Rectangular RC Tank Wall under Construction. I: Case Study*. "*ASCE Journal of Performance Construction Facilities"*, Online Publication Date: 9 Dec 2014.
[25] Seruga A., Szydłowski R., *Thermal cracking prevention with unbonded steel tendons in cylindrical concrete tank wall restrained at foundation slab*, The Third International fib Congress and Exhibition, Washington, May 29 – June 2, 2010.
[26] Stoffers H., *Cracking due to shrinkage and temperature variation in walls*, third ed., Delft University of Technology and I.B.B.C., Delft 1978.
[27] Tayade K. C., Deshpande N. V., Pofale A. D., *Experimental study of temperature rise of concrete and assessment of cracking due to internal restraint*, "International Journal of Civil and Structural Engineering", 4(3): 2014, 353–364.
[28] Zych M., *Analysis of RC tank's walls during early hardening period of concrete, in aspect of watertightness* (*Analiza pracy ścian zbiorników żelbetowych we wczesnym okresie dojrzewania betonu, w aspekcie ich wodoszczelności)* [in Polish], Ph.D. thesis, Faculty of Civil Engineering, Cracow University of Technology, Cracow 2011.
[29] Zych M., *Modification of the simplified method of crack control included in EN 1992-3*, „Structural Concrete", 2016.