PROBLEM OF THERMAL AND SHRINKAGE CRACKING IN TANKS VERTICAL WALLS AND RETAINING WALLS NEAR THEIR CONTACT WITH SOLID FOUNDATION SLABS

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Abstract

The problem of vertical cracks that often appear in reinforced concrete walls above their contact zone with massive reinforced concrete foundations has been analysed. Thermal or shrinkage origin of the cracks resulting from non-uniform thermal and shrinkage deformations of both elements, usually concreted at large time intervals, has been proved. The deformation differences are basically caused by concrete self-heating due to cement hydration heat. A way of calculating unfavourable thermal and shrinkage stresses and the necessary amount of horizontal reinforcement, preventing excessive cracking, in the context of allowable crack width w_{lim}, has been given.

Streszczenie

W pracy poddano analizie problem pionowych zarysowań jakie pojawiają się często w ścianach żelbetowych nad ich stykiem z masywnymi fundamentami żelbetowymi. Wykazano termiczne lub skurczowe pochodzenie zarysowań, wynikające z niezrównoważonych odkształceń termicznych i skurczowych obu tych elementów, betonowanych zazwyczaj w dużym odstępie czasu. Różnice odkształceń termicznych są spowodowane przede wszystkim samoociepleniem się betonu pod wpływem ciepła hydratacji cementu. Podano sposób obliczenia niekorzystnych naprężeń termicznych i skurczowych oraz koniecznego poziomego zbrojenia konstrukcyjnego, przeciwdziałającego nadmiernym zarysowaniom, w kontekście dopuszczalnej szerokości rozwarcia rys w_{lim}.

Keywords: **Induced thermal stresses; Induced shrinkage stresses; Concrete creep; Crack limit width w_{lim}; Minimal amount of structural reinforcement $A_{s,min}$; Crack actual width w_k^{RC}**

1. INTRODUCTION

A frequent case in construction practice are vertical cracks in vertical walls of reinforced concrete tanks, sludge tanks or in the retaining walls of bridge abutments as well as in the walls of frame structures near their contact zone with solid foundation slabs [4], [6], [7], [11].

The cracks run vertically, reach – depending on the wall length and height – up to ⅓, ½ or even ⅔ of the wall height. They are spaced at about 1.5÷3.0 m, and crack maximum width wk,max is – at low horizontal reinforcement – 0.3÷0.5 mm and appears at ⅓ of crack height. The cracks start close above the contact zone between the wall and foundation, they next widen, to reduce the width on reaching the value of wk,max, until they disappear.

Another property of these cracks is that their largest height appears in the middle of wall length, while this height is reduced towards its beginning and end (or towards the expansion joints), as shown approximately in Fig. 1.

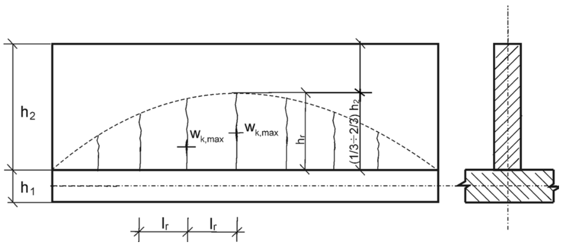

Figure 1.
Typical morphology of vertical cracks in walls near their contact zone with massive foundation

The cracks are troublesome, since in the lower part they usually run through and make the walls permeable for liquids and other media. This is important not only in tanks that are required to be impermeable, but also in other structural elements, mentioned above, in which there appears leakage on the side of ground backfill, producing an disadvantageous appearance of these walls from the outside, full of smears and damp patches.

A closer analysis of the problem shows that such walls are most frequently poorly reinforced horizontally, i.e. perpendicularly to the direction of their basic static and strength behaviour. Designers usually apply – following the code PN-B-03264:2002 [9] – structural reinforcement calculated as for beams of height larger than 1.0 m, which proves to be definitely inadequate. The reinforcement inefficiency increases with the wall thickness, and, to be more precise, with its massiveness "m". The requirements specified in bridge code PN-91/S-10042 [10] are much stricter, although they are not clearly and explicitly defined. The requirements in both quoted codes refer only to reinforcement in near-surface zone, disregarding the case of through-cracking of walls.

It also turns out that the phenomenon in question appears much more frequently for sequentially concreted walls, with an insufficient number of expansion joints and with no work breaks during concreting. For contemporary building industry, with its high construction work rate, these are "normal" conditions and this is why it is necessary to work out preventive measures to eliminate the unfavourable phenomenon of cracking.

2. FORMULATION OF THE PROBLEM

The character of the analyzed cracks shows that they are caused by tensile forces in walls, occurring at the contact zone of the walls and the solid foundation. These are T_2 forces resulting from the tendency to relative displacements of walls and foundation in the contact zone. These forces counterbalance each other, exerting eccentric tension in the walls and eccentric compression in the foundation slabs (Fig. 2). Consequently, there is unfavourable state of stress which exerts tension of the walls mainly in the layers adjacent to the foundation, the upper parts of the walls being under compression.

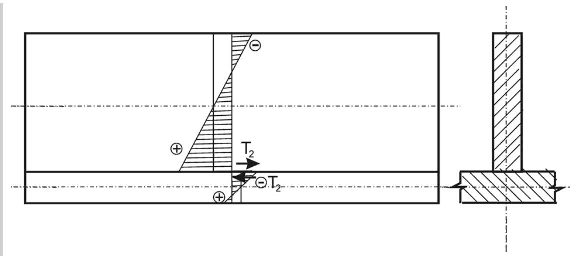

Figure 2.
Approximate state of stresses in the wall and foundation caused by force T_2

The main causes of T_2 force in the contact zone include:

1° non-uniform thermal deformations in the walls and foundation (force $T_{2,t}$),

2° non-uniform shrinkage deformations in the walls and foundation (force $T_{2,cs}$).

Both deformations occur particularly in case when the foundation-wall system is concreted in two stages, i.e. first the solid foundation is concreted, next – after several days– the wall.

The solid foundation first expands under the effect of cement hydration heat, next, when cooling, it contracts, most frequently to the original dimensions. When the concreting of the wall starts, the foundation has already been cooled. In this state, the wall concreted at the second stage, first expands under cement hydration heat (over two to three days) withstanding the resistance of the foundation, still weakly bonded with it. This induces force T1 in the contact zone, which compresses the lower part of the wall and tensile the upper fibres of the foundation. On reaching the self-heating maximum temperature the wall cools over a few days (Fig. 3), withstanding the strong resistance of the cooled foundation, inducing force $T_{2,t}$ mentioned above.

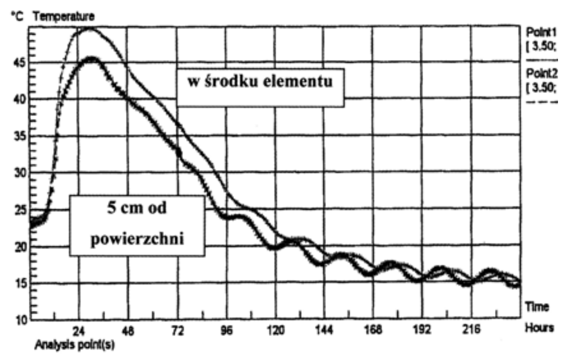

Figure 3.
The course of temperature in the wall due to self-heating under cement hydration heat (tests by CEMEX Poland for reinforced concrete elements with cross-section equal to 0.60 × 3.00 m) [3]

The value of force $T_{2,t}$ depends mainly on the difference between temperature $t_{2,śr}$ of the warmed up wall and $t_{1,śr}$ – cooled foundation ($\Delta tśr = t_{2,śr} - t_{1,śr}$) as well as massiveness „m" of both component elements of the structural system.

Concrete shrinkage acts similarly, but over a longer period of time. Force $T_{2,cs}$ in this case depends on the difference between massiveness „m" of the wall and foundation, as well as the difference between shrinkage deformations after time τ for the wall ($\varepsilon_{cs2,śr}$) and for the foundation ($\varepsilon_{cs1,śr}$) – Fig. 4.

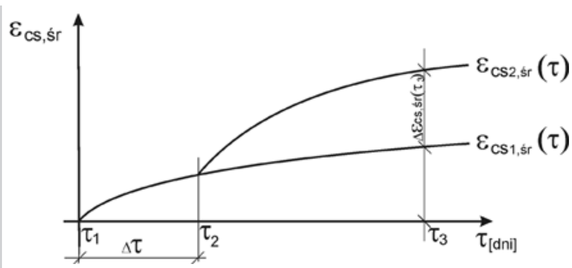

Figure 4.
Differences in shrinkage deformations for the wall and foundation $\Delta\varepsilon_{cs,śr}(\tau_3) = \varepsilon_{cs2,śr}(\tau_3) - \Delta_{\varepsilon cs1,śr}(\tau_3)$

3. INDUCED THERMAL STRESSES IN WALLS

The problem was presented in detail in [3]. The problem discussed in that paper refers to tensile stresses in the lower parts of the wall induced by external constraints, i.e. the resistance of the foundation plate for wall thermal displacements at the cooling stage. From among three practical examples quoted there we shall present one – dealing with the abutment of a bridge structure on a motorway A1 (Fig. 5). The structure of the abutment consists of a foundation plate 1.50 m in thickness (concreted at stage I) of surface module

$$m = \frac{F}{V} = 1.54\ m^{-1}$$

and the wall of body 1.00 m in thickness, developed into bridge seat in the upper part and back wall (concreted at stage II) of surface module m = 1.89 m^{-1}. In the above formula, F denotes the element exterior surface through which the heat (or humidity) is exchanged with the surroundings, V – is the element volume. It should be mentioned here that elements of module m ≤ 2.0 m^{-1} belong to massive elements, for which self-heating under cement hydration heat is $\Delta tśr = 20 \div 50°C$ (35°C on the average) [3].

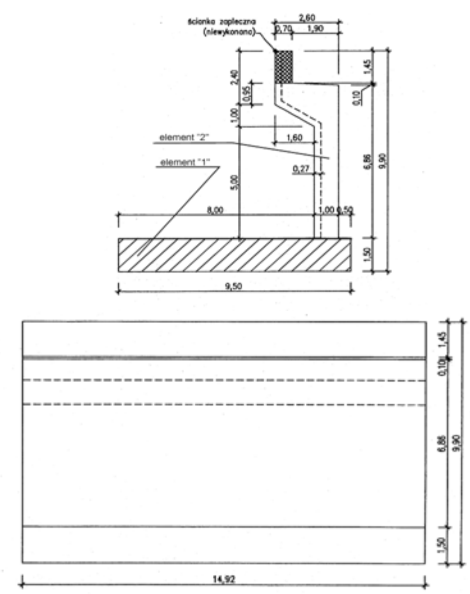

Figure 5.
Cross-section of abutment body in the bridge structure on motorway A1 [2]

The abutment with the total length of 59.55 m is divided into four – 13.50 m to 15.73 m in length – dilated segments. The body walls are monolithically connected with flank walls (wings) 1.00 m thick. The break between subsequent stages of concreting of segments (Δτ) was from 54 to 114 days.

After the removal of formworks off the abutment walls it was found that in their lower parts, on both sides, there were vertical cracks of $w_k = 0.1 \div 0.4$ mm width, at average spacing 2.03÷2.25 m, range in height

1.80÷5.50 m above the contact zone of the wall and the foundation plate. The cracks reached up to the half-height of the walls bodies, and in the wings (connected with them monolithically) as high as 0.75 of their height.

The cracks in the lower part ran through the whole thickness of the wall, which was demonstrated by traces of leakage. The abutment was made of concrete C 25/30, and water curing lasted seven days. After 28 days the concrete mean cube compressive strength in the foundation plate was 40.1 MPa (class B 30), and in the body walls – 47.3 MPa (class B 37).

The wall was reinforced horizontally with bars $2 \times \phi\ 20$ every 10 cm, to the height of up to 1.0 m ($A_{ss} = 62.80\ cm^2/m$), higher up with bars $2 \times \phi\ 20$ every 15 cm. It was quite strong reinforcement (albeit insufficient), which restricted the width of vertical cracks.

On the basis of the concrete mix composition self-heating values were assessed in the cross-section of surface module $m = 1.89\ m^{-1}$, which produced the following results:

– self-heating maximum temperature inside the wall – tw = 57.6 °C,

– self-heating temperature on wall surface – $t_p = 22.3$ °C,

– mean self-heating in wall – $\Delta t_{śr} = 30.9$°C.

The average position of wall gravity centre above wall-foundation plate contact line was

$$e = 4.13\ m.$$

The value of maximum tensile stress in the wall lower fibres was determined on the basis of thorough numerical analysis with the application of ROBOT Millennium 20.0 programme, performed for elastic state and the value of $\Delta t_{śr} = 35$°C by the present authors [13]. The isolines of these stresses have been shown in Fig. 6. The maximum stress obtained was $\sigma_{t,max} = 9.32$ MPa.

In reality these stresses are lower due to:

– the foundation plate being warmed partly from the wall self-heating,

– modulus of elasticity of concrete in the wall lower than assumed in calculations, in the period of so-called thermal shock,

– creep deformation of fresh concrete[5].

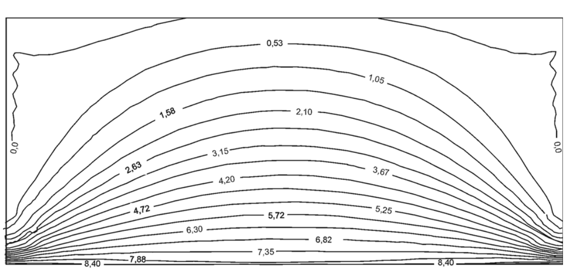

Figure 6.
Isolines of tensile stresses induced by thermal deformations ($t_{śr}$ = 35°C) in wall body of the presented abutment [13]

Adopting, following the authors' experiences:

$\Delta t'_{śr} = 0.9x0.9 = 27.8°C$, $E_{cm}(\tau_r) = 0.8\ E_{cm}$, $\varphi_p(\tau_r, \tau_o) = 0.6$, ageing coefficient $\chi = 0.8$ the maximum stress in question is reduced to the value

$$\sigma'_{t,\max} = \frac{27.8}{35.0}\,0.8\,\frac{1}{1+0.6\cdot 0.8}\,9.32 = \underline{4.00\ MPa} \quad f_{ctm}(\tau_r) \approx$$

$\approx 0.7\ f_{ctm} = 0.7 \cdot 2.60 = 1.82$ MPa.

τ_r denotes here the moment of crack formation, while f_{ctm} is the mean tensile strength of concrete class C 25/30 after 28 day curing.

The calculations above show that due to thermal stresses induced by self-heating there must appear cracks in the lower part of the wall. The height the cracks reach can be evaluated on the basis of Fig. 6 and value $f_{ctm}(\tau_r)$, as well as the value of correction coefficient $\xi = \frac{4.00}{9.32} = 0.429$ They may below isoline

$$\sigma'_t = \frac{f_{ctm}(\tau_r)}{\xi} = \frac{1.82}{0.429} = 4.24\ MPa,$$

i.e. in the wall mid-segment – below the height of ca. $h_{rs} = 0.366 \cdot 9.90 = \underline{3.63\ m}$.

The cracks reaching higher are cracks in the near-surface zone, induced by summing of through stresses and residual stresses in section, resulting from non-linear and non-stationary temperature distribution in wall cross-section. These stresses are not analysed in the present paper. An outline of such an analysis was given in [3].

4. INDUCED SHRINKAGE STRESSES IN WALLS

In case in question, the shrinkage stresses are generated by the difference in the values of concrete shrinkage that appears between the wall constructed later (element "2") and the foundation cast earlier (element "1"). An evaluation of this phenomenon was performed in [2] for a bridge abutment shown in Fig. 5. The fact that the foundation plate does not return humidity towards foundation footing (damp insulation) was taken into account. Also reduced humidity transfer by the lateral surface of the foundation due to covering it with ground to mid-height (after concreting element "2") was considered. This affected the values of determinant thickness of element $h_o = \frac{2}{m}$, which was:

– for foundation plate – element "1" (before concreting of element "2") – h_o = 2.28 m,
– for foundation plate – element "1" (after concreting element "2") – h_o = 2.85 m,
– for abutment body – element "2" (before insulating the walls from the outside) – h_o = 1.06 m.

Shrinkage deformations were calculated according to Eurocode 2 [8], adopting for the designed class of concrete C 25/30 and mean relative humidity of air surrounding the object RH = 70% the following values:

– drying shrinkage $\varepsilon_{cd,o}$ = 0.389‰,
– autogenic shrinkage $\varepsilon_{ca,\infty}$ = 0.04‰.

Detailed calculations have resulted in:

– shrinkage for foundation plate after 114 day curing – ε_{cs} (114) = 0.0414‰,
– shrinkage difference between element "1" and element "2":
  - after 90 day curing of element "2" – $\Delta\varepsilon_{cs}$ (90) = 0.0428‰,
  - after 180 day curing of element "2" – $\Delta\varepsilon_{cs}$ (180) = 0.0558‰,
  - after 360 day curing of element "2" – $\Delta\varepsilon_{cs}$ (360) = 0.0748‰,
  - after 720 day curing of element "2" – $\Delta\varepsilon_{cs}$ (720) = 0.1030‰,
  - after 1800 day curing of element "2" – $\Delta\varepsilon_{cs}$ (1800) = 0.1225‰.

As results from the calculations above, difference $\Delta\varepsilon_{cs}(\tau)$ increases continuously with time τ. There is no point in continuing calculations because after two years (τ = 720 days) the abutment will be completely covered with ground and the conditions of moisture exchange between the abutment and surroundings will alter. Difference $\Delta\varepsilon_{cs}(\tau)$ will begin to stabilise at the level of about 0.11‰ > $\varepsilon_{c,gr} \approx$ 0.1‰, so it is probable that cracks will appear in the wall due to concrete shrinkage alone.

The calculations for the wall of this abutment, performed on the basis of the isolines of stresses of elastic state, given in detail in [13], yielded the maximum value:

$$\sigma_{cs,max} = \underline{2.60\ \text{MPa}} = f_{ctm} = 2.60\ \text{MPa}.$$

Shrinkage is a long-term load, so the value of stress calculated above should be reduced due to concrete creep (for reinforced concrete in phase II) to the value of ca. [12]:

$$\sigma'_{cs,\max} = \left(\frac{1}{1+0.2\,\phi_p}\right)\sigma_{cs,\max} = \left(\frac{1}{1+0.2\cdot 2.0}\right)2.60 =$$

$$= 1.86\ \text{MPa} < f_{ctm} = 2.60\ \text{MPa},$$

$$> f_{ctk} = 1.80\ \text{MPa}.$$

For the sake of precision of calculations, also stresses in section of concrete induced by concrete shrinkage due to resistance of horizontal reinforcement should be taken into account. However, this effect is negligibly small. For example, for the abutment in question, reinforced at the bottom horizontally with bars 2 × ϕ 20 every 10 cm (A_{ss} = 62.80 cm^2/m), shrinkage deformation will be reduced to the value of [1]:

$$\Delta\varepsilon_{cs}^{Rc}(\tau) = 0.904\Delta\varepsilon_{cs(\tau)} = 0.904\cdot 0.11 = \underline{0.10\%}$$

At the same time, however, additional tensile stresses σ_{cs}^{II} = 0.125 MPa will be generated in cross-section.

In total then we shall obtain:

$$\sigma'_{cs,\max} = 0.904 \cdot 2.60 \cdot 0.71 + 0.125 = 1.68 + 0.125 = 1.80\ \text{MPa} < f_{ctm} = 2.60\ \text{MPa}.$$

The residual stresses from non-linear and non-stationary humidity fields in wall cross-section are disregarded in our analysis. The problem was presented in detail in [1].

From the calculations performed in points 3 and 4 of the present article it results that the vertical cracks in the walls near their contact zone with the massive foundations were mainly caused by induced thermal stresses $\sigma'_{t,\max}$. In presented example they reached the value of $\sigma'_{t,\max}$ = 4.00 MPa » $f_{cm}(\tau_r)$ = 1.82 MPa. The shrinkage stresses in time τ → ∞ attained the value of $\sigma'_{cs,\max}$ = 1.80 MPa < f_{ctm} = 2.60 MPa.

Shrinkage stresses can, however, sum up with thermal stresses relaxed by creep. With time τ this can enhance an increment of tensile stresses, which will cause further expansion of cracking in the walls lower part, above the contact zone with the solid foundation (formation of new cracks or larger width of the existing ones).

Assuming after [12] that relaxation coefficient, due to creep, for reinforced concrete in phase II is approximately $k_3 = 1/(1+0.3\,\varphi_p)$, for our case we shall obtain, for τ → ∞:

$$\sigma''_{t,\max} + \sigma'_{cs,\max} = \frac{1}{1+0.3\,\phi_p} f_{ctm}(\tau_r) + 1.80 = \frac{1}{1+0.3\cdot 2.0}\ 82$$

$$+ 1.80 = 2.94\ \text{MPa} > f_{ctm} = 2.60\ \text{MPa}.$$

5. NECESSARY STRUCTURAL REINFORCEMENT IN WALLS

There are various preventive measures to reduce the possibility of thermal-shrinkage cracks formation, described in detail in [3]. These are connected with concrete technology and technology of concrete works. The most efficient measure, however, which takes into account climatic conditions difficult for a designer to predict, in which the structure will be concreted, is the application of adequate structural reinforcement. To calculate the amount of reinforcement the formula, given in Eurocode 2 [8], for the minimum reinforcement amount in view of cracking can be used:

$$A_{s,min} = k_c \; k \; f_{ct,eff} \; \frac{A_{ct}}{\sigma_{s,\lim}} \tag{1}$$

where: k_c – coefficient considering distribution of stresses in cross-section at the moment prior to cracking,

k – coefficient considering the effect of non-uniform self-balancing stresses in cross-section,

$f_{ct,eff}$ – concrete mean tensile strength at the moment of expected cracking,

A_{ct} – cross-sectional area of element tensile zone at the moment prior to cracking,

$\sigma_{s,lim}$ – maximum stress adopted in reinforcement in tension immediately following cracking, depending on cracks limit width and bars diameter.

For the case analysed in the present publication the following can be adopted [1]:

$k_c = 1.0$ – as for axial tension,

$k = 1.0$ – for wall thickness $h \le 300$ mm,

$k = 0.65$ – for wall thickness $h \ge 800$ mm,

$$f_{ct,eff} = \begin{cases} 0.7\, f_{ctm} & \text{– for thermal stresses,} \\ f_{ctm} & \text{– for shrinkage stresses and combined thermal-shrinkage stresses} \end{cases}$$

$A_{ct} = b \cdot h_{rs}$

h_{rs} – height of through-cracks (resulted from detailed calculations),

$\sigma_{s,lim}$ – depending on rebars diameter ϕ and crack limit width wlim.

Value $\sigma_{s,lim}$ can be calculated from formula [1], [12]:

$$\sigma_{s,\lim} = f_{yk} \sqrt{\frac{\varphi_s}{\varphi}} \tag{2}$$

where:

f_{yk} – characteristic yield point of reinforcing steel,

ϕ – diameter of reinforcement applied,

ϕ_s – reinforcement optimum diameter, ensuring that condition

$w_k \le w_{lim}$, calculated from formula [1], [12]:

$$\phi_s \le \frac{3\tau_1 \cdot w_{\lim} \cdot E_s}{f_{yk}^{\;2}} \tag{3}$$

will be satisfied

where:

τ_1 – bond strength between concrete and reinforcement; in case of bars cast horizontally $\tau_1 = 0.15\, f_{cm}$ can be adopted,

f_{cm} – concrete mean compressive strength $f_{cm} = f_{ck} + 8$ MPa.

Following Eurocode 2 and PN-B-03264:2002 for leak-proof structures $w_{lim} = 0.1$ mm should be adopted, for regular reinforced concrete structures in building industry and bridge engineering $w_{lim} = 0.2 \div 0.3$ mm depending on exposure class. In the opinion of the present publication, sufficient protection against vertical cracking can be obtained adopting $f_{ct,eff} = f_{ctm}$ oraz $\sigma_{s,lim}$ for $w_{lim} = 0.3$ mm.

Calculating $A_{s,min}$ after the code PN-B-03264:2002 [9] we would have $k = 0.8$ – for $h \le 300$ mm and $k = 0.5$ – for $h \ge 800$ mm, and the calculated reinforcement amount would be $A_{s,min}$ smaller by about 20%.

It should be taken into account now that formula (1) does not correspond to the task ideally. Tensile force N acting on the element in question is not constant, but decreases rapidly the moment thermal-shrinkage cracking appears, to reach the value N = 0 for a plain concrete element. Hence the crack resultant width, when the amount of reinforcement resulting from formula (1) has been applied, will be w_k^{RC} and will be much smaller than wlim. Value w_k^{RC} in the function of wlim can be estimated from formulae (2) and (3) assuming that for $w_{lim} \rightarrow w_k$ (w_k – crack width for a plain concrete wall) force N decreases in a non-linear manner down to zero. Complete force N and complete value $\sigma_{s,lim}$ appear only at the moment of cracking.

From formulae (2) i (3) it appears that:

$$w_{\lim} = \frac{\varphi \cdot \sigma_{s,\lim}}{3\tau_1 \cdot E_s} = \alpha\, \sigma_{s,\lim}^2 \qquad (4)$$

which has been shown in Fig. 7. With stress $\sigma_s < \sigma_{s,\lim}$ decreasing in the reinforcement after cracking, value will be:

$$w_k^{RC} = \frac{\alpha}{\sigma_{s,\lim}} \int_o^{\sigma_{s,\lim}} \sigma_s^2 ds = \frac{1}{3}\, w_{\lim} \qquad (5)$$

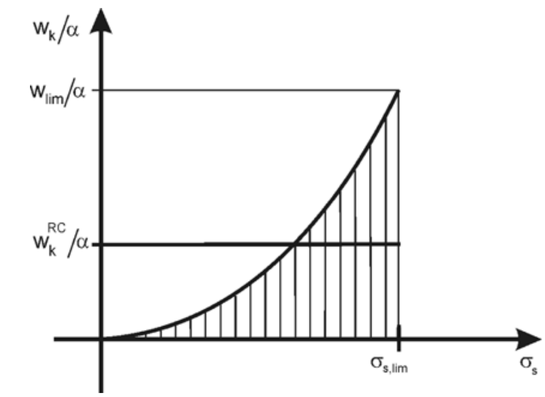

Figure 7.
Relationship between w_k/α and stress in reinforcement σ_s as a function of crack width

Hence it appears that adopting $w_{\lim} = 0{,}3$ mm, when the horizontal reinforcement has been used in the amount resulting from formula (1), the real crack width will be only $w_k^{RC} = 0.10$ mm, which means in practice that the condition of wall leak tightness has been satisfied. This principle proved true when further bridge objects on motorway A1, between Sośnica and Bełk junctions, were constructed.

Formula (5) also points to the fact that for less demanding structures and difficult thermal conditions connected with concreting $w_{\lim,2} = 0.5$ mm $= {}^5/_3\, w_{\lim,1}$, for example, could be adopted (which corresponds to many observed cases of cracking when there is no or definitely too small amount of horizontal reinforcement).

In such case, we would obtain in reference to the example discussed above (for which wlim,1 = 0.3 mm was adopted and ϕ · was calculated from formula (3)):

$$\phi_{s,2} = \frac{5}{3}\, \phi_{s,1}$$

and

$$\sigma_{s,\lim} = f_{yk} \sqrt{\frac{\varphi_{s,2}}{\varphi}} = 1.29\, \sigma_{s1,\lim} \le f_{yk},$$

so, with the given diameter of horizontal reinforcement ϕ, its amount could be reduced in comparison with that calculated originally (for $w_{\lim,1} = 0{,}3$ mm) to the value 1/1.29 $A_{s,\min}$, i.e. by about 22 %. The thermal-shrinkage cracks would then have the width $w_k = \frac{1}{3} \cdot 0{,}5 = 0.17$ mm < 0.20 mm, which would correspond to normal reinforced concrete work conditions for which the requirements for leak tightness do not need to be satisfied.

To satisfy the condition of leak tightness in the abutment body wall analysed in the present article reinforcement on both sides should be applied in the amount:

$$\sigma_{s,\lim} = 222 \text{ MPa, dla } w_{\lim} = 0.3 \text{ mm i } \phi = 20 \text{ mm}$$

$$A_{s,\min} = 0.65 \cdot 2.60\ \frac{100x100}{222} = \underline{76.0 \text{ cm}^2/\text{m}}$$

up to the height $h_{rs} = 3.63$ m, following the principles given above.

In the 1.0 m high zone, 2 ϕ 20 every 10 cm with $A_{ss} = 62.80$ cm^2/m was given, which is 82,6% of the necessary reinforcement. So, the expected width of through cracks should be:

$$\sigma_s = \frac{76.0}{62.80} \cdot 222 = 268.7 \text{ MPa} < \text{fyk} = 355 \text{ MPa},$$

$$w_k^{RC} = \frac{1}{3} \cdot \frac{20 \cdot 268.7^2}{3 \cdot 0.15 \cdot 33 \cdot 200 \cdot 10^3} = 0.162 \text{ mm.}$$

On the height over 1.0 m, 2 ϕ 20 every 15 cm with $A_{ss} = 41.86$ cm^2/m was applied. This is, to the height of $h_{rs} = 3.63$ m, only 55.1% of the necessary reinforcement. Consequently, the expected width of through cracks should be:

$$\sigma_s = \frac{76.0}{41.86} \cdot 222 \qquad f_{yk} = 355 \text{ MPa},$$

the steel yields, there is uncontrolled cracking of concrete. For $\sigma_s = f_{yk} = 355$ MPa, it would be:

$$w_k^{RC} = \frac{1}{3} \cdot \frac{20 \cdot 355.0^2}{3 \cdot 0.15 \cdot 33 \cdot 200 \cdot 10^3} = \underline{0.283\ mm}$$

Should $\sigma_s = 403.1$ MPa $< f_{tk} = 480$ MPa were adopted, then

$$w_k^{RC} = \frac{1}{3} \cdot \frac{20 \cdot 355.0^2}{3 \cdot 0.15 \cdot 33 \cdot 200 \cdot 10^3} = \underline{0.283\ mm} = 0.365 \text{ mm},$$

i.e. a value close to that observed in the body wall structure.

Over the height h_{rs} = 3.63 m reinforcement in the near-surface zone is practically sufficient, the minimum amount of which defined in [2] is A_{1ss} = 16.15 cm^2/m on each side of the section, which is smaller than the existing reinforcement A_{1ss} = 20.93 cm^2/m.

6. CONCLUSIONS

The paper describes the original method of calculating the thermal and shrinkage induced stresses in RC walls of tanks with massive foundation slabs. These walls are usually constructed at large time intervals with the application of continuous concreting method. During the first stage of maturing, the cement hydration heat generates temperature fields in the walls that resulted in the significant tensile stresses in the lower parts of walls. In presented paper the values of these stresses are estimated for two practical cases. They are the reason for vertical cracks with the width 0.1-0.4 mm, leading to leakage and in consequences to walls gradual degradation. The method for determining the necessary amount of structural horizontal reinforcement is included, concluding that for the analyzed practical cases the amount of this reinforcement is insufficient. The important achievement of the work constitutes the equation (5) that couples the real crack width in the wall w_k^{RC} with the value w_{lim} for equations (1), (2) and (3) that are used for the calculation of the optimal horizontal reinforcement in walls.

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