STRAIN STATE OF EARLY AGE REINFORCED CONCRETE WALL FIXED AT THE BOTTOM EDGE

Andrzej SERUGA
Professor PhD. C. Eng.
Cracow University
of Technology, Poland

Rafał SZYDŁOWSKI
MSc. C. Eng.
Cracow University
of Technology, Poland
rszydlowski@imikb.wil.pk.edu.pl

Mariusz ZYCH
MSc. C. Eng.
Cracow University
of Technology, Poland

Summary

Five large capacity rectangular reservoirs realized last year in Cracow are the subject of this paper. Due to the construction and technology solution which has been applied it was necessary to realize construction joints in vertical and horizontal directions as well as two expansion joints. In spite of such solution, many vertical cracks on some wall segments under construction has been found. Based on the preliminary analysis it was stated the reasons of concrete cracking in vertical direction. The minimum reinforcement area necessary to keep the width of the cracks bellow 0.10 mm has been also calculated.

Keywords: concrete; concrete reservoir; concrete tanks; minimum reinforcement; cracks

1. Introduction

Nowadays in Cracow there is realized the greatest sewage-treatment plant in Poland. Ten cylindrical reinforced concrete tanks and five rectangular reinforced concrete reservoirs were constructed in 2004 year. The authors of this paper have possibility to observe the methods of execution these tanks as well as to check on the development of mechanical concrete properties at early age.

Because of the time-limit predicted to set in motion the sewage-treatment plant, it was necessitate to concrete these structures also in winter 2004 year. Since a properly placed, highly impermeable concrete mix was essential to the fluid containment and long-term durability of the structure, the concrete class C25 has been used. On January, February and in the first part of March the metallurgical cement CEM III/A-42.5N has been applied. In the latter end the Portland rapid-hardening cement with slag addition CEM II/B-S-32.5R type and the metallurgical cement CEM III/A-32.5 type have been applied as equivalent.

2. Structure geometry, construction and mechanical concrete properties

Overall, the rectangular tank measures are 99.85×39.5×8.1 m high. The nominal volume of each reservoir is equal to 30000 cu.m. The whole construction of monolithic reinforced concrete reservoir rests on monolithic reinforced concrete floor slab the thickness of 0.75 m. Because of the total length of reservoir, two expansion joints has been applied. The maximal distances between the interior walls and their location are shown in Fig. 1. The thickness of the exterior walls is equal to 0.75 m only at the height to 1.8 m measured from the floor slab. The wall thickness of the upper part is equal to 0.50 m. Thickness of the interior walls are constant along the height and equal to 0.35 m.

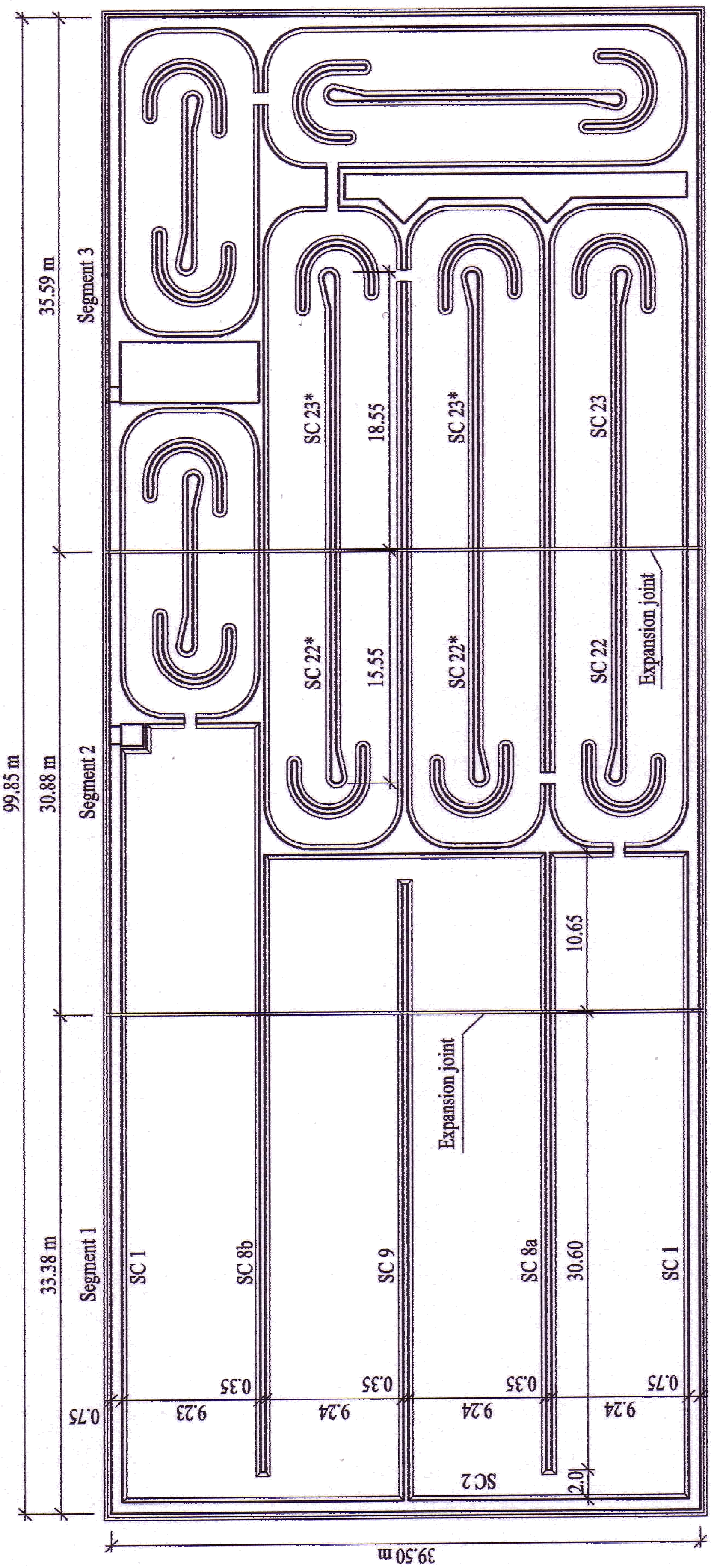

Fig. 1 Rectangular reinforced concrete reservoir geometry and expansion joints layout.

In the walls numbered SC1, SC2, SC8a, SC8b and SC9 it has been done vertical construction joints. Moreover horizontal construction joints has been done at the height of 0, 0.4, 1.8, 4.4, and 7.1m in exterior wall and at the height of 0, 0.4, 4.4 and 7.1 m in the interior wall. All construction joints were fully waterstopped using conventional PVC material.

The project team selected a wall formwork system that used through-wall removable taper ties. Breakoff – type permanent form ties were not selected due to the concern for insufficient cover and possible corrosion at the ties. As a result, some thousands holes nominally 30 mm in diameter 0.35 to 0.75 m long had to be grouted.

Reinforcing chairs were used on all walls and slabs to insure that the specified 50 mm cover was maintained consistently. The hydraulic curing method such as wet burlap and sprinkling has been applied for crack control and curing.

The wall under investigation type SC8a has been constructed in two segments height of 4.4 m. One vertical construction joint is located at the distance of 14.25 m measured from the expansion joint. The segment 1 of the wall SC8a was concreted on March 12 and the metallurgical cement CEM II/A-42.5N was used. The segment 2 was concreted on March 30 and in this case the Portland cement CEM II/B-S 32.5R was used.

Compressive strength was monitored closely during construction since higher than required strengths would lead to excessive heat generation during hydratation, and thus to shrinkage cracking. An excellent coefficient of variation in compressive strength was observed during the majority of concrete production. Air entrainment was also closely monitored because of its importance to concrete permeability and long-term corrosion resistance. The development of modulus of elasticity, shrinkage, tensile and compressive strength in day-time for Portland cement concrete are presented in figures 2, 3, 4 and 5 respectively.

The neighbouring tank was hydrostatically tested by slowly filling with subsoil water. The tank under discussion will be tested this year.

3. Thermal cracking

S. J. Lokhorst and K. van Breugel [1] have analyzed the problem of thermal cracking in a wall and in a wall-slab structure. The self-equilibrating stresses in the wall increased with: decreasing ambient temperatures, increasing wall thickness, early removal of formwork, reactivity of the cement, increasing wind velocity. Additional stresses will develop if a part of the average load-independent axial deformations of the wall is restrained.

For a degree of restraint of 0% the structure is free to bend and to deform in axial direction. At 100% restraint the both bending and axial deformations are prevented completely. Walls thicker than 0.5m were likely to crack in most cases. In thin walls wind may have a positive effect since it reduces the peak temperatures and the temperature differentials between wall and slab. The probability of cracking increased for higher initial temperatures of the concrete mix.

Localization and layout of vertical cracks along the height of the wall segments 1 and 2 are presented in Fig. 6. As it is shown the cracks in the distances 4.70 (0.15 mm), 7.92 (0.15 mm), 10.63 (0.35 mm), 14.25 (construction joint), 16.86 (0.30 mm), 18.43 (0.30 mm) and 21.16 m (0.30 mm) measured from the expansion joint are through. Length of three cracks are equal to the height of the wall segment. The cracks width ranged from 0.05 to 0.35 mm. The concrete wall under consideration is reinforced in horizontal direction with steel bar 20 mm dia. uniform distributed along the height at the distance of 150 mm. The average distance between the expansion cracks equal to 2.09 m is smaller than 4.4 m.

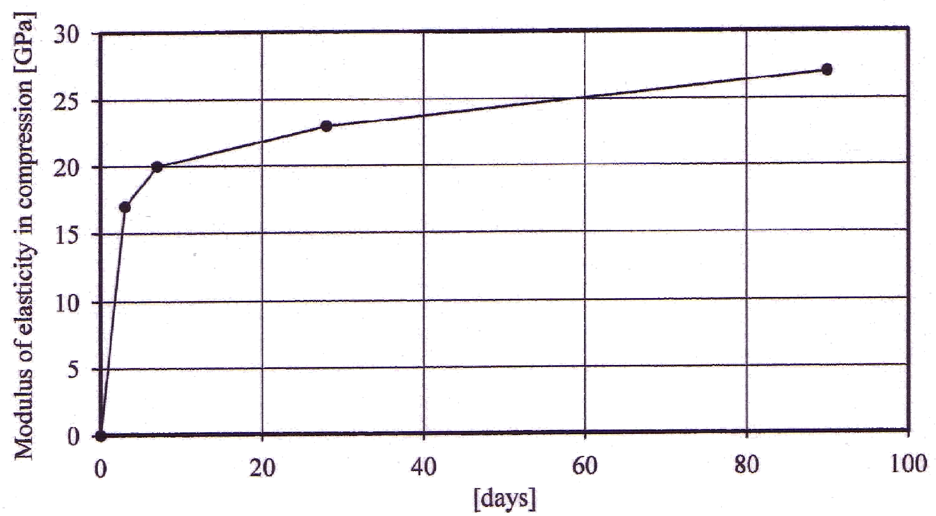

Fig. 2 Development of concrete modulus of elasticity in day-time.

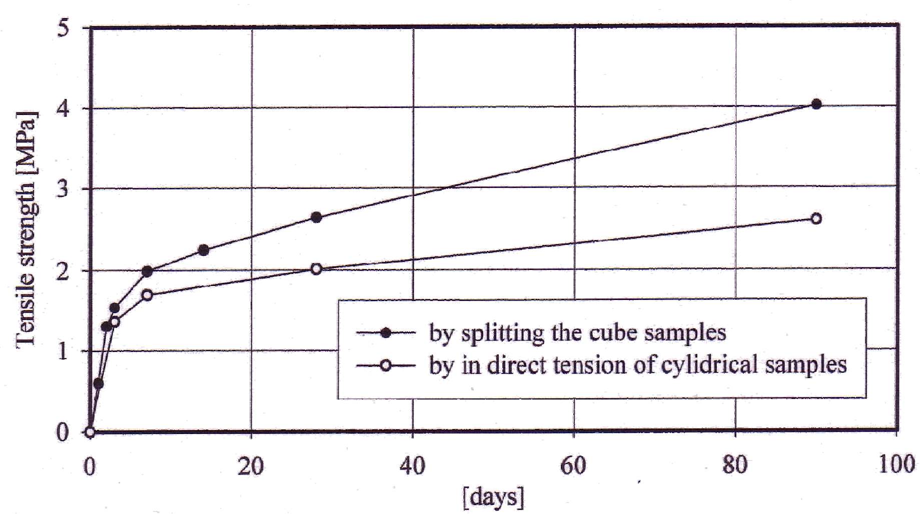

Fig. 4 Development of concrete tensile strength in day-time.

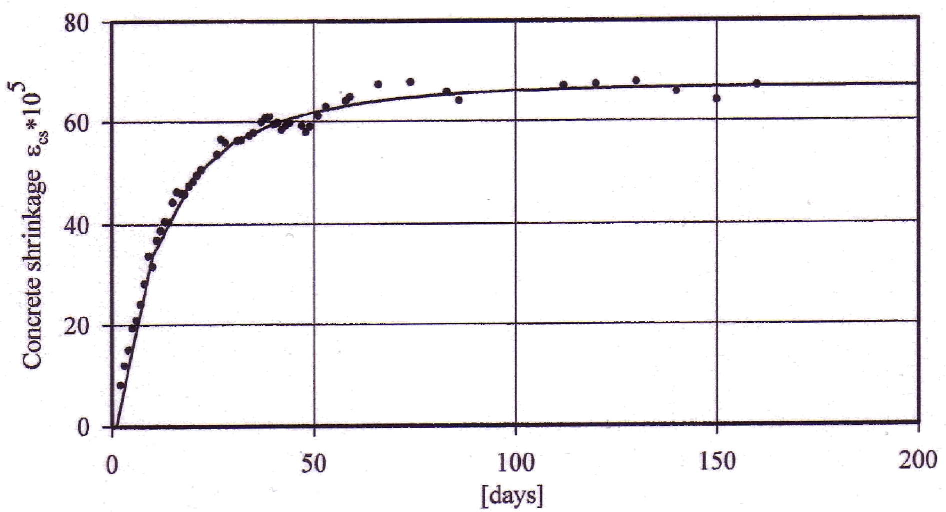

Fig. 3 Development of concrete shrinkage in day-time.

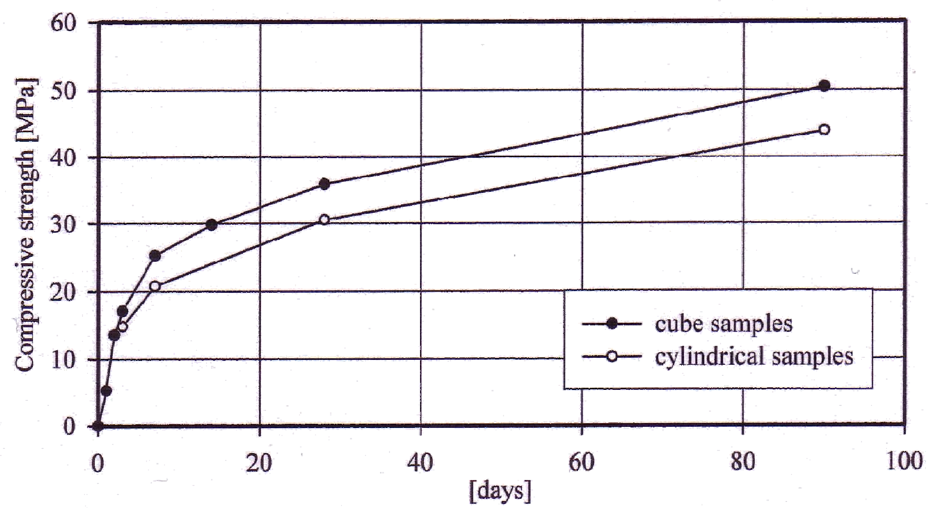

Fig. 5 Development of concrete compressive strength in day-time.

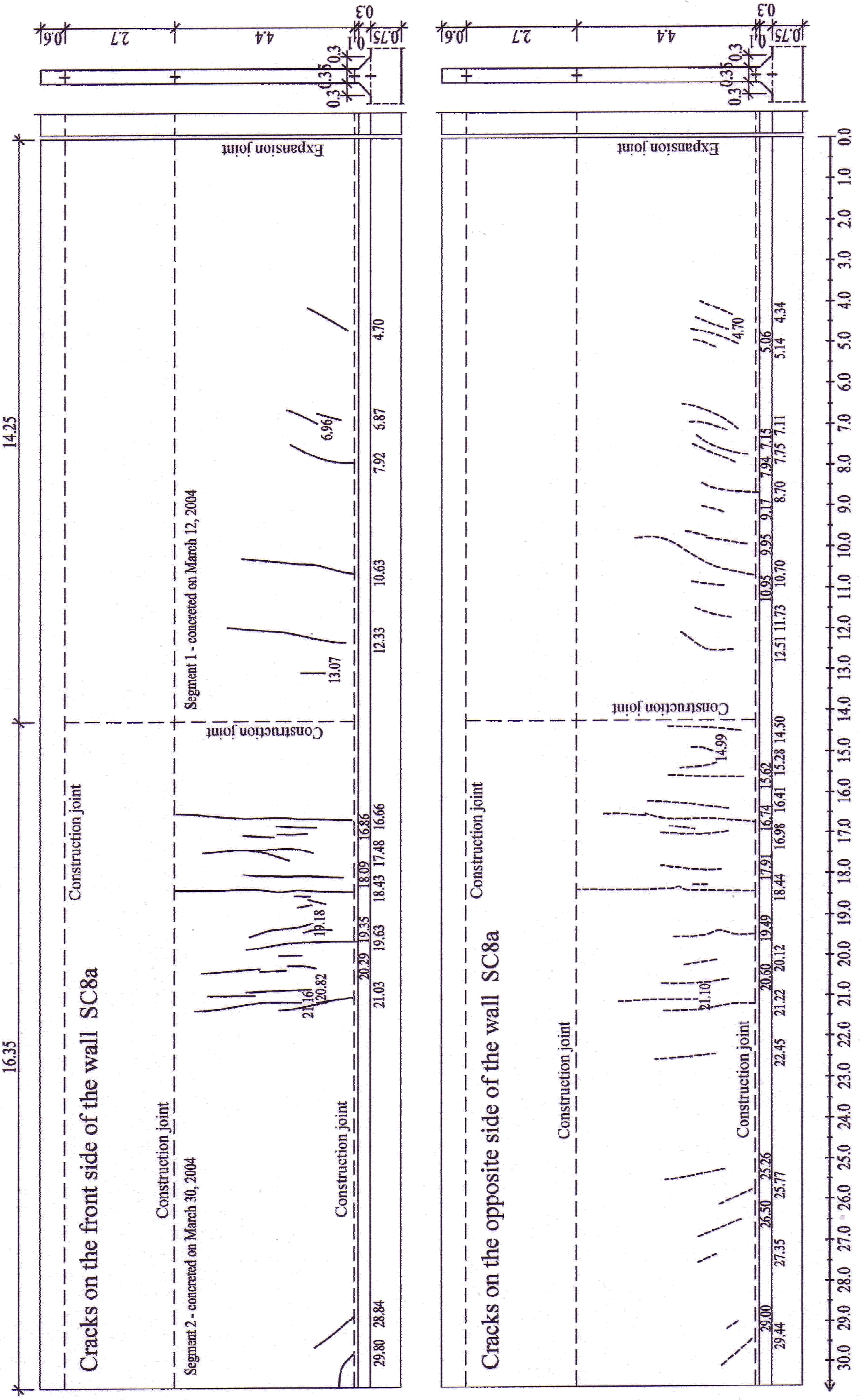

Fig. 6 Location of the construction and expansion joints as well al layout of vertical cracks.

4. The temperature development in hardening concrete

The calculation of the mean temperature variation in hardening concrete it has been done based on [2]. The thermal material properties used in the temperature calculation are: λ_b - 2.59W/mK the thermal conductivity was calculated on the basis of concrete composition [2], however with respect to calculation which were made during the first week of hardening of concrete, the value of thermal conductivity was increased by 20% [6] up to value 3.11W/mK.
c_b – 0.772kJ/kgK the coefficient of heat capacity also was calculated based on the concrete composition [2].
α - 5.8W/m^{2}K the convection coefficient was assumed according to C.L. Townsend [5] under the assumption that the value of wind speed is equal to zero. In case of wall in the formwork it has been assumed the substitute convection coefficient α_w expressed as follows:

$$\alpha_w = \frac{\alpha}{R\alpha + 1} \qquad \text{and} \qquad R = \frac{d}{\lambda}$$

where: d = the thickness of the formwork, λ = the thermal conductivity of the formwork.

The solution of the heat transport equation (2) in case of one-way heat transport with the assumption that a function Q is describes by (3), is given by K. Hirschfeld [3][2]. Based on the assumption that the heat transfer on the both surfaces of concrete wall is the same, the mean temperature in a cross section of the wall can be written as follows:

$$\overline{T}(t) = \left[\frac{2\alpha(1-k) + dc_b\gamma_b K_{Tp}}{2\alpha k - dc_b\gamma_b K_{Tp}}\right] \cdot \frac{qC}{c_b\gamma_b}\exp\left(-K_{Tp}t\right) + \left[T_p - T_o - \frac{qCdK_{Tp}}{2\alpha_o^n - dK_{Tp}c_b\gamma_b}\right] \cdot$$

$$\cdot f(\varphi_1)\exp\left(-\frac{\alpha_o}{c_b\gamma_b d}\right)t + T_o \tag{1}$$

$$\frac{\partial T}{\partial \tau} = a_b \cdot \nabla^2 T + \frac{\partial Q}{\partial \tau} \cdot \frac{1}{c_b \cdot \gamma_b} \tag{2}$$

$$Q_{(\tau)} = C \cdot q \cdot \left[1 - \exp\left(-K_{Tp} \cdot t\right)\right] \tag{3}$$

where: φ_1 = is the solution of: $\varphi_1 tg\varphi_1 = \frac{\alpha d}{2\lambda} = B_i$,

$P = \frac{d^2 K_{Tp}}{4a_b}$ - Predwoditielew`s number,

$$f(\varphi_1) = \frac{2\sin^2\varphi_1}{\varphi_1^2\left(1 + \frac{\sin 2\varphi_1}{2\varphi_1}\right)}, \qquad \alpha_o^n = \frac{2\lambda\varphi_1^2}{d}.$$

$k = ctg\sqrt{P} \cdot \sqrt{P}$, T_o – temperature of atmospheric air, T_p – initial temperature of concrete.

The influence of the ambient temperature on the mean temperature of the wall was taken into account by calculating changes in time of relative difference temperature value (mean on the whole cross section) interdependence on Fourier`s number [4].
Presented above solution of the temperature development in hardening concrete is only an approximation of the real temperature in the cross section, because it has been not taken

w

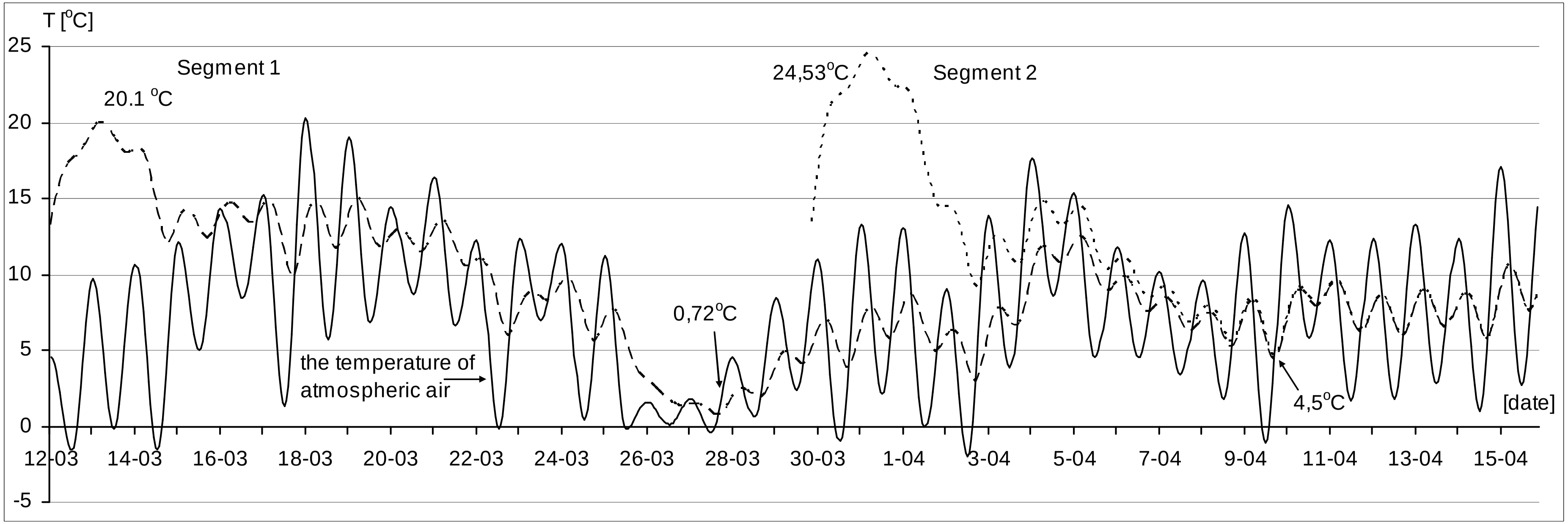

Fig. 7 Development of the mean temperature in cross sections of the wall and the temperature of atmospheric air.

The wall segment concreted 30.03.2004, E_c= 20 GPa, Δt = -19.9°C, (18 days) The wall segment concreted 12.03.2004, E_c= 20 GPa, Δt = -15.6°C, (28 days)

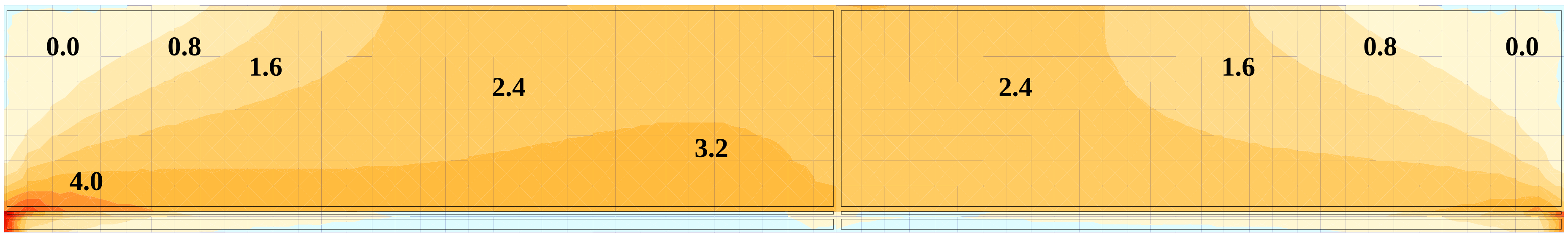

Fig. 8 The thermal stresses distribution in considered wall, 28 days after concreting the first segment.

into account the effect of solar radiation and the distribution of the temperature in the cross section. Moreover, in the calculation it has not taken into account the influence of the moisture on the temperature. In the immediate future the temperature will be calculate taking into account these phenomenons.

The development of the average concrete temperature in the cross section of the wall and the ambient temperature are presented in Fig. 7. Seasonal temperatures taken from meteorological office in Cracow ranged from -3.6°C to 20.3°C in the period from 12.03 to 15.04.2004. In fact the real temperatures values measured at building place were higher. The calculations of average concrete temperature in cross section has been done taking into consideration the initial temperatures of the concrete mixture measured in each day of concreting: 12.03 (11.0°C), and 30.03 (13.5°C). The maximal and minimal values of average concrete temperatures for each wall segment are shown in Fig. 7.

5. Results of statical calculations and final conclusion

Presented wall has been modeled with computer FEM system software Robot Millennium. The horizontal junction between the wall and floor slab has been modeled taking into consideration the existing longitudinal trapeze cross-section beam the height of 0.40 m (Fig. 6). It has been assumed the total length of the wall equal to 30.6 m. The vertical construction joint has been neglected. From the development of the average temperature in the cross section of the wall presented in fig. 7, can be seen that 10th of April the equalization of temperature value in both segment is realized. In the statical calculation it has been assumed the temperature decrease in each wall segment from the time of concreting to 10th of April. Moreover the adequate values of modulus of elasticity for different concrete and suitable age has been taken in calculation. The map of internal tensile stresses in main direction resulted from the statical calculation is presented in fig.8.

From comparative analysis the results presented in fig.6 and 8 can be seen that localization of vertical cracks in distances 14.25, 16.86 and 18.44 m correspond to the tensile stress in concrete equal to 3.2 MPa. Moreover in the bottom corner of wall segment 2 the diagonal cracks in distances 28.84 m (0.30 mm) and 29.80 m (0.20 mm) correspond to maximal tensile stress equal to 4.0 MPa.

Taking into consideration the existent reinforcement it has been calculated the width of crack and the steel stress according to [7] and [8] assumed $\Delta T = 20°C$. In the first case the resulted value of $w_{se} = 0.08$ mm and $\sigma_s = 118$ MPa whereas in the second case $w_s = 0.15$ mm and $\sigma_s = 315$ MPa for average crack distance equal to 2.09 m. In the purpose to secure $w_s \leq 0.1$ mm it has been necessary to reinforce concrete wall with steel bar 20 mm dia. at a distance of 110 mm ($\sigma_s = 250$ MPa).

References

[1] LOKHORST S.J., BREUGEL K., „Prediction of thermal cracking in hardening concrete structures: Effects of various degrees of restraint", *Concrete 95 Toward better concrete structures*, Lisbone, Australia 4-7 September 1995,

[2] KIERNOŻYCKI W., „Betonowe konstrukcje masywne", Kraków, 2003.

[3] HIRSCHFELD K., „Die Temperaturverteilung im Beton. Springer Verlag", Berlin, 1948.

[4] BOGOSŁOWSKI W. N., „Fizyka budowli", Warszawa, 1975.

[5] TOWNSEND C.L., 1959. "USBR practices for control of cracking in arch dams", *Journal of Power Division*, Vol. 85, 1959.

[6] ROSTASY F.S., „Fruhe Risse in massigen Betonbauteilen-Ingenieurmodelle fur die Planung von GegenmaBnahmen", Berlin, 2001.

[7] ROSTASY F.S., HENNING W., "Zwang in Stahlbetonwänder auf Fundamenten", *Beton-und Stahlbetonbau*, Vol. 8, 9, 1989, pp. 208-214, 232-237.

[8] IVANYI G., „Bemerkungen zu „Mindestbewehrung" in Wänden", *Beton-und Stahlbetonbau*, Vol. 90,11, 1995, pp. 283-289.