Accumulated material, workmanship and design errors in the unstable, prefabricated reinforced concrete cylindrical tank

M Maj

1 Wrocław University of Science and Technology, Wyb. Wyspiańskiego 27, 50-370 Wrocław, Poland

E-mail: marek.maj@pwr.edu.pl

Abstract: There are complex reasons for the failure of prefabricated reinforced concrete material. Those are errors related to putting the concrete structure up, insufficient supervision of material checkup during construction works, design errors, inappropriate usage of other building materials, and in particular mistakes made while linking material elements that do not allow for the complete connection of all individual parts into one monolithic as a whole block. In the case of the biological gas tanks presented in the submitted paper, the following methods applied for measuring the width of prefabricated concrete slabs joints did not prevent the contractor from committing major errors in material resulting in significant imperfections of the tank walls. As a result, the presented tank, which maintains stability for smaller diameters; for large diameters becomes a buckling-sensitive structure. Poorly made joints become places of the buckling wave initiation and drastically reduce critical force. Repeated failures of the same tank with constantly increasing discipline of the facility construction has confirmed low resistance of the tank to the second-order effects, which is strictly related to the usage of the bad material, characterized in detail in the research paper.

1. Introduction – the scope of the study

Reinforced concrete structures are exposed to many risks concerning both: the composite itself and the construction of steel armor [1, 2]. In the case of concrete material used in tanks, defects may also appear in the connections (joints) between concrete elements [3-4]. The risk nature of the defects of the connections may also have various proveniences: static and dynamic loading, thermal effects (loads) imperfections in geometry [5] and local stress concentrations (punching resistance) [6, 7]. Similar problems appear at the very early stage of construction (foundation piles [8]) till the last roof shells [9]. What is vital about elements made of concrete, is their durability [10] and changing material parameters in time. Cyclic loading and corrosion combined with creep effects put some demands for sophisticated analysis, considering variable safety factor in time domain. Many substance testing procedures are addressed for current control in course of construction of concrete structures and throughout its lifetime [11-15]. Structural Health Monitoring systems are being widely developed for that purpose.

The article presents a number of observations and reasons related to selection of appropriate building materials to the design, including material solutions for construction details and finally execution of a tank made of a prefabricated reinforced concrete for liquid prepared for a biogas plant.

In the article there are presented problems related to the material usage in the construction of prefabricated tanks, prestressed by external ties without adhesion. For instance vertical joints between prefabricated elements are glued and then compressed in the circumferential direction employing...
external steel tendons without adhesion, according to one of the European tank systems [16]. Some tanks constructed by using these materials have failed and in Poland, there were observed some failures during the construction phase of tanks utilizing the before mentioned combination of materials in this system. A grave dispute arose over the responsibility of the subsequent tank failure. It was revealed that the causes of subsequent failures during the construction phase of the same tank were seen in the wrong methodology of material usage thus the way of connecting prefabricated elements with glue. It was suspected that the combination of materials used was therefore inappropriate. In order to resolve this dispute and find out the reasons for the failure, it was necessary to carry out a detailed static and strength analysis of the materials used, and in consequence thorough mentioned in the article the assessment of the quality of prefabricated elements from reinforced concrete. Additionally a separate methodology of erecting the tank was conducted. In particular, the analysis of prefabricated concrete elements was carried out taking into account four-step approach:

- in terms of the quality of used material in our case concrete;
- deviations in the dimensions of the elements;
- construction methods in the different construction phase;
- quality control during the construction of the cylindrical shell of the tank.

The subject of analysis took into consideration also a number of errors during the preparation of serial production of the concrete elements hence used for large diameter reinforced concrete reinforced tanks.

Evidence for the reason of failure allowed to change basic constructional assumption for production and process of proper selection for such prefabricated tanks. In the article there are presented the following composite materials: non reinforced concrete joints at a very heavy load place, glue connecting two concrete slabs, reinforced concrete and prestressing steel without adhesion. These materials create a complex structure in which all elements are subjected to work together as regular as clockwork. However, it was not possible, mainly due to the ignorance and lack of knowledge about material behavior in the real construction of a large tank.

2. General technical and material parameters of the tank

The following strength calculations for a concrete, glued joint are intended to indicate that the use of concrete for very demanding structures, such as tanks with large diameters without knowledge of the physical characteristics of concrete and the glue applied, can lead to failure.

The digester (Figure 1) with a diameter of 29.82 m, with an installation area of 698.4 m², is founded on 0.15 m thick lean concrete of the C12/15 class. The height of the tank above the site according to the design is 5.0 m and the absolute height $H=6.0$ m, internal diameter $R_w=29.313$ m. The bottom of the tank is 0.15 m thick and made of C25/30 concrete reinforced with $\phi 8$ mm crossbars spaced 0.15×0.15 m, placed at a distance of 0.05 m from the top surface of the bottom of the tank. The slip layer between the bottom of the tank and the concrete base is made of one layer of hard PVC foil. The bottom slab is topped with a ring foundation measuring 0.44×0.50 m, also placed on a C12/15 concrete slab. The reinforcement of the ring consists of 4 bars $\phi 16$ mm at the bottom and 2 bars $\phi 12$ mm at the top. Rebars $\phi 6$ mm are placed every 0.30 m. The upper surfaces of the foundation and bottom slab are in one plane. Rebars $\phi 6$ mm in the shape of an inverted letter “U” protruding 80 mm above the upper surface of the bottom were released from the ring foundation, from the inside of the prefabricated wall: 2 pieces for each prefabricated wall element. One reinforcing bar $\phi 6$ mm was carried out under the rebars. It is designed to implement an internal concrete beam 0.20 m high and 0.25 m wide made of C25/30 waterproof concrete on such reinforcement after assembling the prefabricated elements (before the target compression). Also, a ring beam connected with the internal beam by concrete penetrating under the prefabricated elements was planned on the external side of the tank wall.

The tank wall is designed with 61 reinforced concrete elements made of C35/45 concrete; prefabricated width $b_w=1.513$ m, length $l=6$ m and variable thickness $g=110÷135$ mm. Reinforcement of the slab is made of BSt500 steel: two 125 mm meshes made of $\phi 8$ mm bars; vertical bars are
located on the external surface of the slab. There are 7 vertical bars $\phi 12$ mm between the meshes. Both reinforcing meshes with vertical bars make a dense spatial mesh 44 mm wide.

**Figure 1.** Tank failure and cut joint.

Reinforcement does not reach the vertical edges of the element. The vertical bars of the meshes are located at a distance of 80-85 mm from the vertical edges along the joint. The horizontal bars are terminated at a distance of 50 mm. The rib from the outer surface of the slab is reinforced with one $\phi 12$ mm bar connected to the vertical bar of the external reinforcing mesh $\phi 8$ mm with a two-cut rebar $\phi 6$ mm at the spacing of 0.40 m. The joint in the form of a tongue and a groove with a radius of $R_{km}=0.44$ m is unreinforced, the height of the groove being $h_z=0.093$. The glue used to connect the edges of the elements of slabs in the joints could be not sufficiently spread over the entire surface of the joint and in addition could not carry large shear stresses.

The mentioned above construction details have an impact on the load of the glued concrete joint, which was subjected to spatial influence of stresses but not axial, as assumed by the project designer, was destroyed. The slab is flat from the inside, while from the outside a vertical rib has been constructed along the left edge of the element. There are 33 standard holes located at the height of the slab where peripheral prestressing ties without adhesion are passed. (figure 2). Two pilasters were designed, i.e. prefabricated concrete elements thickened in the central part, through which internal ducts are run, allowing the strands to lead out of the element. The tension force is stabilized by jaw-type anchors embedded inside the seats in both faces of prefabricated elements. The tank wall is ultimately prestressed by 30 steel ties.

**Figure 2.** Cross-section through the prefabricated slab.
The tank’s geometrical data are presented in the following points. Parameters connected with geometry and static analysis of prefabricated element are juxtaposed in Table 1. Material properties of reinforced concrete elements (for concrete and reinforcing steel respectively), are juxtaposed in Table 2.

### Table 1. Forces and dimensions.

| Parameter description                        | Value used in the analysis |
|----------------------------------------------|-----------------------------|
| maximum compressing force in wall            | $N_{\text{max}} = 1070.51 \text{ kN/m}$ [1] |
| mean compressive stresses                    | $\sigma_{cp} = \frac{N_{\text{max}}}{A}=11.51 \text{ MPa}$ |
| compressing force in cable                   | $N_{\text{spr3}} = 132 \text{ kN}$ |
| external radius of the tank                  | $R_e = \frac{D_z}{2} = 14.810 \text{ m}$ |
| internal radius of the tank                  | $R_i = \frac{D_w}{2} = 14.655 \text{ m}$ |
| medium radius of the tank                    | $R_m = \frac{D_m}{2} = 14.73 \text{ m}$ |
| width of joint groove                        | $R_g = 0.044 \text{ m}$ |
| shear force perpendicular the direction of compression | $V_{\text{SD}} = 14.14 \text{ kN}$ [1] |
| joint width                                  | $b_z = 1 \text{ m}$ |
| contact width between the slabs minimum      | $t_i = 0.02 \text{ m}$ |
| effective depth of the joint                 | $D \equiv h_z = 0.093 \text{ m}$ |
| slab width in the mean of length             | $b = 1560 \text{ mm}$ |
| center of gravity of the shear section in the joint | $x_0 = 0.0047 \text{ m}$ |
| shear area in joint                          | $A = h_z \cdot 1m = 0.093 \text{ m}^2$ |
| static moment                                | $S_{\text{c0}} = A \cdot x_0 = 0.433 \text{ cm}^3$ |
| moment of inertia of the section             | $I_{\text{se}} = h_z^2 \cdot b_z = 0.67 \text{ cm}^4$ |
| section modulus                              | $W_z = h_z^2 \cdot b_z = 0.0014 \text{ m}^3$ |
| width of prefabricated element               | $l_{\text{hub}} = 1.513 \text{ m}$ |
| thickness of element at the edge             | $g_{\text{kon}} = 0.110 \text{ m}$ |
| average thickness of element                 | $g_{\text{obl}} = 0.125 \text{ m}$ |
| thickness of precast element counted in the „key” of the joint | $g = 0.093 \text{ m}$ |

### Table 2. Materials’ properties used for further analysis.

| Parameter description                        | Value used in the analysis |
|----------------------------------------------|-----------------------------|
| Concrete C35/45                               | $\gamma_c = 25 \text{ kN/m}^3$ |
| modulus of elasticity of concrete            | $E_c = 34 \text{ GPa}$ |
| design compressive strength of concrete      | $f_{ctd} = 19.4 \text{ MPa}$ |
| characteristic compressive strength of concrete | $f_{ck} = 35 \text{ MPa}$ |
| design tensile strength of concrete          | $f_{td} = 1.47 \text{ MPa}$ |
| average tensile strength of concrete         | $f_{ct} = 3.2 \text{ MPa}$ |
| Poisson’s coefficient                        | $\nu = 0.2$ |
| BS500 – reinforcing steel                   | $f_{yk} = 500 \text{ MPa}$ |
| characteristic tensile strength of steel     | $f_{y} = 420 \text{ MPa}$ |
| design tensile strength of steel             | non-adhesive strands with 12.5 mm nominal diameter and nominal tensile strength are used in the compression |

3. Type of prestressed tendons used in the tank and tank wall compression

NEDRI SPANSTAAL BV strands without adhesion type Y1770S7-12.5 were used; $f_y = 1770 \text{ MPa}$. The strands are positioned at the height of the wall with an average distance between cables of 0.11-0.19 m (about 8 cables per 1 m of height in the lower part of the tank). The maximum compressive force in the wall during compression was $N_{\text{max}} = 1070.51 \text{ kN/m}$. It is obtained in stages; in the first stage, the force that stabilizes tank walls is applied with a tension value of $N_{\text{spr1}} = 40 \text{ kN}$; then the force in the
cables before concreting the rim - \( N_{sp2}=80 \text{kN} \), and finally full compression, with the prestressing force \( N_{sp3}=132 \text{kN} \). Adherence of the prestressed tie to the tank surface around the entire wall circumference was designed. The task of the used cables was to directly transfer the pressure from prestressing steel to concrete elements.

The prefabricated reinforced concrete digester failed before the water load test (leak test). Therefore, the extreme loads were dead weight and prestressing rods load. After carrying out all operations, including full compression of the tank, two days after the completion of works, about 1/3 of the slabs including one pilaster fell, and the compression rods broke off. The failure initiated with the disintegration of the foundation ring. The foundation ring broke into pieces from both the inside and outside, and the \( \phi 6 \text{ mm} \) plain steel wire which reinforced the foundation ring was torn off. At the same time, the vertical contact between two adjacent slabs was broken off. The concrete in the contact area between the slabs was crushed and chipped at the height of 4 prestressing ties. In total, 21 prefabricated elements were destroyed. The internal corner in the concave part of the joint was cut along the entire height of the slab. Then the outer part of the joint was destroyed. Among the debris, there were also elements of adhesive mortar several centimeters long and several millimeters thick detached from both fragments of slabs. Earlier, horizontal cracking in slabs appeared during the full tensioning of the tie-rods.

4. Tolerance values allowed in the system and variation coefficients of technical parameters

The sum of manufacturing deviations and normal permissible dimensional tolerances increases with the number of precast elements. For example, deviation tolerance in the prefabricated joint (glued) is 2 mm, which results in an average of dozens of millimeters of permanent deformations, so-called imperfections in the tank geometry (with 60 joints and systematic accumulation of deviations. It is very difficult to “lose” these deviations using the given construction technology, auxiliaries, and the deviation control system. For example, measuring the correct positioning of the slabs, i.e. the measurement of the horizontal angle \( \beta + \beta' \) between the slabs in the vertical joint is a linear measurement.

![Figure 3. Method of control measurement of the angle between adjacent slabs.](image)

The coefficient of variation of the measured parameters was also calculated based on [1] and own calculations (Table 3). The tolerance for angle measurement is 1.5°, which corresponds to the change in the distance between the contact edge and the linear dimension measured perpendicularly to the slab within \( 0 \pm 6 \text{ mm} \). The angle measurement itself consists in determining the distance between the right edge of the left slab \( x \) expressed in millimeters and the distance between the left edge of the right slab \( x' \) (figure 3). The opening width between the slabs was not measured. Figure 3 shows the arrangement of two adjacent slabs and a linear gauge 1000 mm long applied in the axis of vertical contact. The sum of the angles \( \beta \) and \( \beta' \) is the angle between adjacent slabs determined based on distances \( x \) and \( x' \). The analysis shows that the used measuring system is not sensitive to the actual width of the vertical contact. This means that the contact width in the range of 20÷50 mm does not cause noticeable changes in the horizontal angle. The tolerance of radius determination in relation to the recommended value in the facility construction system is 10 mm. The change in horizontal angle between the connected elements ±1.5°, which corresponds to the change in the distance between the joint edge and the linear gauge measured perpendicularly to the slab within \( 0 \pm 6 \text{ mm} \). The height of the upper surface of the tank depends on the height of the washers under the prefabricated elements,
which should be 40 mm.

The generally applicable dimensional tolerances for prefabricated reinforced concrete elements are up to approx. 15 mm for length and width of elements, approx. 10 mm for surface deplanes, and approx. 10 mm for element thickness. The applied tolerances in conjunction with the methods used to control the tank geometry may result in the summation of the assembly error, which may occur in every joint and every element of the 61-element wall. The postponing error becomes apparent when local imperfections from the cylindrical shape appear. These imperfections cause additional shear forces and bending moments. Measurements of slab deviations from the vertical (and also from the assumed wheel) were carried out on several existing tanks with the same geometry. The results of the measurements are shown on Figure 4. One can only guess that similar and maybe larger imperfections took place in the case of a damaged tank.

Figure 4. Distribution of geometric deviations of the upper part of the existing tank

Additionally, due to the large radius of the tank, the geometric analysis shows that the cable does not fully adhere to the prefabricated wall but is applied to the notch in the middle of the panel width at the place of the largest width of 0.135 m, which is an additional concentrated load in the center of the panel. This means that an additional concentrated force of about 20 kN, unforeseen in the technical design, was applied to the slab. Based on [1] and own calculations, variation coefficients of the following technical parameters of the digester walls were calculated:

| Tested quantity                                           | Coefficient of variation $v_{10d}$ [%] |
|-----------------------------------------------------------|---------------------------------------|
| concrete volumetric weight                                | 2.5                                   |
| concrete strength                                         | 2.5                                   |
| strength of reinforcing steel $f_{yd} = 2.5\%$            | 2.5                                   |
| extension of ties                                         | 10.2                                  |
| joints width                                              | 10.2                                  |
| horizontal angle measured between prefabricated elements   | 4.6                                   |
| tank circle radius                                        | 0.1                                   |
| geometric deviations (imperfections)                      | 72                                    |

Elements of the construction show low variability in the range of measured parameters, which indicates a good level of workmanship. Only deviations of the upper edge of the tank are very heterogeneous. Low values of the coefficient of variation show that with large tank diameters and very good performance, the material which was used, concrete knots and adhesive glue, are inadequate.

5. Static strength calculations and results

Static strength calculations were carried out for geometrical data. The purpose of these calculations is to show the exceeding of the load capacity and instability of the tank walls as the cause of the failure. The results of the calculations of internal forces are shown in Figure 5 [1]
Static calculations indicate the magnitude of forces that the structure could transmit. However, there were circumstances not included in the system in which the tank was erected. Figure 5 shows the results of calculations of internal forces for an empty tank loaded only with the strands. The maximum horizontal compressive force is \( N_{\text{max}} = 1070.51 \text{kN/m} \); it can also be directly calculated from the distribution of prestressing forces spread across the tank. The longitudinal bending moment is 9.2 kNm/m; the shear force in the longitudinal direction ranges from -12.5 to 26.5 kN/m.

5.1. Appearance of diagonal cracks, shear resistance of the construction:
According to [17], the possibility of diagonal cracks indicating the necessity to reinforce the shear zone is expressed by the formula:

\[
|\sigma_{t,\text{max}}| < f_{\text{ctm}}
\]

where:
\( \sigma_{t,\text{max}} \) – principal tensile stress,
\( \tau_{xy} \) – tangential stress

\[
\sigma_{t,\text{max}} = \frac{(\sigma_x + \sigma_y)}{2} - \frac{[(\sigma_x - \sigma_y)^2 + \tau_{xy}^2]^{1/2}}{2}
\]

\[
\tau_{xy} = \frac{V_{SD} \cdot S_{c0}}{(I_{csz} \cdot b_z)}
\]

Stress from prestressing \( \sigma_x \); \( \sigma_y = 0.00 \text{ MPa} \)

For the parameters contained in this study it is obtained:
\( V_{SD} = 14.14 \text{ kN} \)
\( |\sigma_{t,\text{max}}| = 0.07 \text{ MPa} \)
\( f_{\text{ctm}} = 3.2 \text{ MPa} \)
\( |\sigma_{t,\text{max}}| << f_{\text{ctm}} \)

5.2. Shear strength of tank walls on first-order sections according to [17]
Minimum strength of concrete vertical post according to

\[
V_{Rd,c} = \left[ C_{Rd,c} \cdot k \cdot \left( 100 \cdot \rho \cdot f_{ck} \right)^{1/2} + k_1 \cdot \sigma_{cp} \right] \cdot b_z \cdot d \cdot (v_{\text{min}} + k_1 \cdot \sigma_{cp}) \cdot b_z \cdot d
\]

\[
v_{\text{min}} = 0.035 \cdot (k_1^{3/2} \cdot f_{ck}^{1/2})
\]

where:
\( C_{Rd,c} = 0.18 / 1.4 = 0.13 \)
\( k_1 = 0.15 \)
\( k = \min \{ (200/d)^{1/2} ; 2 \} = 2 \)
\( v_{\text{min}} = 0.035 \cdot (k_1^{3/2} \cdot f_{ck}^{1/2}) = 0.035 \cdot (2^{3/2} \cdot 35)^{1/2} = 0.59 \text{ MPa} \)

Expected compressive calculated stress: \( \sigma_{cp} = 11.51 \text{ MPa} \); \( k_1 \cdot \sigma_{cp} = 1.73 \text{ MPa} \);
\( V_{Rd,c} = \max (160, 56; 200) \text{ kN} \). In the unreinforced joint \( \rho = 0 \). Shear resistance on first-order sections is determined by compressive stress from the tension of prestressing tie-bonds, not the concrete element.
itself non-reinforced in joints. But in the situation when shear section is I joint groove than \( V_{rd,c} = \max(76.946) \) kN. Than in the case of appearance of shear forces caused by imperfections joint shear forces can be bigger than shear strength.

5.3. Cracking moment \( M_{cr} \)
Crackes in the slab may appear when bending moments exceed the cracked moment value \( M_{cr} \)
\[
M_{cr} = W_{z} \cdot (\sigma_{cp} + f_{ct,e})
\]
where: \( f_{ct,e} = f_{ct} = 3.20 \) MPa
\[
M_{cr} = 0.0014 \cdot (8.56 + 3.20) = 16.46 \text{ kNm}
\]
Bending moments exceed the value of cracking moment, so we can expect cracked zones on slabs.

5.4. Buckling sensitivity according to [18]
Substitute compressive load on the shell (pressure normal to shell surface, usually critical pressure \( p \)) can be obtained by dividing the maximum compressive forces in the cables by the tank radius. About 8 cables work for 1 meter in height of wall: \( p_{obl} = 8 \cdot 132/14.81 = 72 \) kN which is compliant with [19]

Calculation of the critical compressive force of the cylindrical shell due to the possibility of buckling was performed according to the formulas given in [18].
\[
N_{c1} = 1070.51 \text{ kN} - \text{compressive circumferential from prestressing}
\]
\[
p = N_{c1}/R_{z} = 72.2 \text{ kN/m} - \text{radial pressure from prestressing}
\]
\[
\beta = (n \cdot l)/(\pi \cdot R_{z} \cdot m) = 2.579 \text{ m}^{-1}
\]
Buckling aspect ratio \( \beta \) can be derived from formula (8):
\[
\beta = (n \cdot l)/(\pi \cdot R_{z} \cdot m)
\]
For 61 elements on perimeter of tank assumed \( n = 20 \) zero points for the buckling wave form in circumferential direction and \( m = 1 \)-number of buckle half waves in the vertical direction. Wall flexural stiffness per unit width \( D \):
\[
D = E_{c} \cdot h_{z}^3/(12(1 - \nu^2)) = 2373.97 \text{ kNm}
\]
Correlation factor to account for difference between classical theory and predicted instability loads \( \gamma = 0.5 \cdot 0.7 \cdot 1.0 \), \( N_{c1} = N_{c1} - \text{compression caused by pressure} \)
\[
N_{c1} = 1.04 \cdot (\gamma \cdot Z)^{1/2} = 5.263
\]
Curvature parameter \( Z \) for isotropic cylinder can be derived from formula (8):
\[
Z = \beta^2/(R_{z} \cdot h_{z}) \cdot (1 - \nu^2)^{1/2} = 25.61
\]
Equation of critical circumferential force with coefficient \( k_{y} \)
\[
N_{cr}^{(1)} = k_{y} \cdot \pi^2 \cdot D / l^2
\]
\[
p_{cr}^{(1)} = N_{cr}^{(1)}/R_{z}
\]
Buckling factor \( k_{y} \)
\[
k_{y} = 1/\beta^2 \cdot (1 + \beta^2)^2 + 12/\pi^4 \cdot \gamma^2 \cdot Z^2 \cdot (1 + \beta^2)^3
\]
where: \( k_{y} = 9.0 \)
\[
N_{cr}^{(1)} = 5863.896 \text{ kN}
\]
\[
p_{cr}^{(1)} = N_{cr}^{(1)}/R_{z} = 395.94 \text{ kN/m}^2 - \text{critical pressure of} \ N_{cr}^{(1)}
\]
For minimum value \( k_{y} \) compression critical pressure \( p_{cr}^{(2)} \):
\[
p_{cr}^{(2)} = (0.855/(1 - \nu^2)^{3/4}) \cdot (E \cdot (\gamma \cdot Z)^{1/2})/(R_{z} \cdot h_{z})^{5/2} \cdot (l/R_{z})
\]
where: \( p_{cr}^{(2)} = 231.18 \text{ kN/m}^2 \)
Critical circumferential force: \( N_{cr}^{(2)} = p_{cr}^{(2)} \cdot R_{z} = 3423.89 \text{ kN} \). The dependences of the critical forces \( N_{cr}^{(1)}, N_{cr}^{(2)} \) and accompany critical compressed pressure \( p_{cr}^{(1)}, p_{cr}^{(2)} \) on changes in wall thickness, changes in coating radius and changes of the number of buckling waves on the tank perimeter shown in Figure 6, 7 and 8. In axially compressed elements with the force \( N \) and the bending moment \( M_{zg} \).
the effect of slenderness is ignored if the increase in the compressive force eccentricity is not more than 10%. The expression \( 1/(1-N/N_{cr})<1.1 \) describes the coefficient that increases static eccentricity. The critical force \( N_{cr} \), for which any second-order influences can still be ignored, is \( N_{cr}=1.1\cdot N \). \( N_{cr}=11\,000 \) kN is the critical force beyond which the dimensioning related to the second-order theory should be corrected for stresses in the tank from prestressing tie-bonds \( N=1070.51 \) kN. Figures 6, 7, 8 show that the phenomenon of shell buckling has a big impact on the walls of the tank. The shell is made unstable by a large number of places with frequent waves (figure 8). The shell also becomes unstable if the radius is larger than \( R=12 \) m and the shell’s thickness is below \( g=11.5 \) cm. \( N_{cr}^{(2)} \) force has a constant value calculated from \( p_{cr}^{2} \) and indicates that once it is reached, the system becomes unstable.

Figure 6. Critical force in the tank wall depending on wall thickness

Figure 7. Critical force in the tank wall depending on tank radius

Figure 8. Critical force in the tank wall depending on the number of buckling waveforms

Due to the poorly designed joints, only concrete and glued ones, which are not resistant to the
appearance of imperfections, buckling can occur at much lower value of $p_{cr}$ stresses. The wall of the shell made of precast slab-like elements (with unreinforced edges glued together, vulnerable to imperfections and local loss of stiffness, with the presence of shear forces and bending moments) may buckle. The edges can initiate the so-called curve buckling line with nodes on them. It can be assumed that the number of waves is similar to the number of elements with weak joints to the adjacent elements buckling has a wave-like form and the joints in the locks are weak. In this way, the critical force (critical load) compressing the shell is drastically reduced.

5.5. Internal forces caused by the local pressure of prestressing

The pressure made by the cable with prestressing force $N_{spr3}=132$ kN induces local bending. The equation for the meridional bending moment $M_{zg}$ and shear force $Q$ was developed by Timoshenko [19], page 199.

$$M_{zg}=\frac{\phi F_{des}}{4\cdot b_{p}}$$

$$Q=\frac{\phi F_{des}}{2}$$

$$b_{p}=\left[12\cdot(1-\nu^{2})/(D_{w}h_{z}^{2})\right]^{1/4}$$

where: $F_{des}=N_{spr3}/R_{z}=8.91$ kN

$D_{w}=2\cdot R_{w}=29.31$ m

$\beta_{p}=0.9621/m$

parameters $\phi$, $\omega=1$;

According to calculations, the bending moment for a single tie-bond is $M_{zg}=2.31$ kNm/m, and for several adjacent tie-bonds, it is $M_{zg}=6.49$ kNm/m. The shear force reaches $Q=4.45$ kN/m for a single cable and $Q_{l}=14.48$ kN/m for adjacent cables. These are longitudinal forces. About 20% of them are transmitted in the latitudinal direction in line with Poisson's ratio $\nu=0.2$

5.6. Influence of imperfections and notch in the precast slab on the increase in shear forces and latitudinal bending moments

Figure 9 shows the additional local force $T$ in the point (imperfection) caused by the displacement angle $d\beta$ in the point where the prestressed cable is applied. Equation of equilibrium can be written in form:

$$2\cdot N\cdot \sin(d\alpha+2q\cdot R_{z}\cdot d\beta)=0$$

$$2\cdot q\cdot R_{z}\cdot d\alpha=2\cdot N\cdot \sin(d\beta)$$

$$T=2\cdot N\cdot \sin(d\beta)$$

$$q=2\cdot N\cdot \sin(d\beta)/(2\cdot R_{z}\cdot d\alpha)$$

where: $l_{wob}=1.513$ m

$R_{z}=14.91$ m

$d\beta=0.0533$ rad

$d\alpha=0.0203$ rad

Tension force of 8 tie-bonds per 1 meter
- $N=1070.51$ kN
- $q=189.90$ kN/m
- $T=114.01$ kN
- $M=1/8\cdot T\cdot l=21.54$ kN
- $T/2=57.00$ kN

The application of prestressing force to the notch may result in shear force with the value of $V_{des}=0.5\cdot 57.00=28.5$ kN/m and a bending moment $M=21.54$ kNm/m. The same applies to
imperfections. The outwardly protruding slab joints are point-loaded with the prestressing force, as is the central notch of the slabs. By calculating the total additional bending moments, it can be estimated that latitudinal moments $M_r > 40$ kNm/m may have occurred and the bending moment is greater than the cracking moment. The bending moment is meaningful, and yet it was not considered for calculations of reinforcement in the prefabricated slabs, especially in the joints.

Similarly, by adding up all additional cutting forces that may appear in the wall joint, a significant value of $T_{r} > 60$ kN/m is obtained. The longitudinal bending moment calculated from the statics has a value of 9.2 kNm/m, and the additional moment resulting from imperfection significantly increases it. The weakest point is the vertical joint of prefabricated slabs. The joint is unreinforced, only glued. It accumulates shear force and bending moments caused by local application of prestressing force to the central notch of the slab and the outwardly protruding joint due to imperfection.

6. Discussion and conclusions
The exhaustion of the wall load capacity within the joint, whose width significantly exceeded the designed width, was the direct cause of the collapse. The system of design and construction of prefabricated reinforced concrete tanks presented in the article did not take into consideration the direct current control of the angle of slab joints and the materials used to reinforce and their connection in between. This means that the contact width in the range of 20 to 50 mm does not cause noticeable changes in the horizontal angle between prefabricated slabs measured during the construction, which causes the appearance of wall imperfections, especially at the joints. It is difficult to comply with the requirement for 4.0 mm thickness of the adhesive layer (precisely selected glue) on the convex joint, and 1.5 mm once the elements are pressed together. The selection of glue joining two prefabricated elements in variable assembly time was the reason for two previous failures of tanks according to the ruling and the effected material test.

Simultaneous control of the precise positioning of elements along the tank perimeter, the exact level of the upper edge of the elements, the verticality of the wall with a pitch tolerance of 1-2 mm, execution of a working break under the whole tank with a height of 40 mm, control of tension by measuring the deformation of tie-bonds in the anchorages, appropriate clamping force of the slab in the joint, etc. is the cause of large difference from the designed parameters as low-precision control equipment is used. Materials considered for the design phase, had not been neither properly selected. No interaction and synergy pretesting between materials was executed. Besides, the construction of outer and inner rings with appropriate reinforcement and proper care is the basis for non-sliding tank supports.

As the tank radius increases, the wall susceptibility to uncontrolled second-order influences increases, especially in the joints of prefabricated elements. It means that the materials in the moment of increase shrink. Tank geometry errors (imperfections of over 30 mm), increased vulnerability to buckling result in increased lateral forces and bending moments in slab joints, especially when significant imperfections occur.

The failure could have been caused by the so-called "thick" error or systematic overlapping of many events unfavorable for the facility, such as construction errors, unforeseen deviations from dimensions, summing up of unforeseen additional static forces, the indefinite stiffness of vertical slab joints, etc. The probable cause of the failure was the rupture of the ring that fixed the wall to the foundation slab. The ring was constructed on a mature concrete base, which is the bottom slab of the tank, which again led to significant stresses that were not transferred by the structural reinforcement applied. The ring could have split into separate parts, which reduced its load capacity as a result of shrinkage stress. In addition, insufficient longitudinal reinforcement as well as insufficient anchorage of rebars in the bottom slab was designed, which did not guarantee non-sliding support for the ring capable of transferring high compressive force, shear force and initial bending moment from prestressing. The additional rotation of the wall in its lower part was not compensated by appropriate reinforcement in the ring, which caused cracking and detachment of the ring from the bottom slab. A structure in which these load capacities have been exhausted may be in a state of unstable equilibrium.
Any additional impact of loads, such as the rupture of the supporting beam under the tank wall (which occurred in the case under consideration), or the rupture of the prestressing tie-bond giving a short-term dynamic impulse, may be the reason for a sudden change in the equilibrium state of the structure (appearance of the so-called bifurcation points). The structure may thus rapidly lose its apparent balance and form without any visible warning.

The tank is a structure for which the ratio of the effective thickness to diameter ratio is \( g/D_m = 1/318 \). On average, reinforced concrete tanks are marked by the parameters \( g/D_m = 1/50 \div 1/200 \). For smaller values, tanks are classified into the group of ‘thin-walled silo’ ([20]), which are sensitive to buckling. The joints were designed only for compression, so that the reduced strength values of the joints between elements combined with additional shear forces and bending moments may become points shell buckling is initiated in the horizontal direction, when the unreinforced and glued joints themselves become imperfect and change the cylindrical geometry of the tank. About 10.5 m is the boundary radius of the tank at which the designed evenly distributed loads reach their critical value.

In summary the research shows that not reinforced concrete joints connected by glue taking into consideration the construction are very sensitive on shear stress. The prefabricated tanks with large diameters made of reinforced prestressed concrete are susceptible on random imperfections, which create a complex stress state in the concrete material. For instance, these imperfections are classified as errors in joints, geometrical deviations and material defects. In conclusion the concrete material by its nature has a low tensile and shear strength, therefore it is considerable that the reinforced concrete must be used to construct the joint slabs rather than non-reinforced one.

References

[1] Maj M and Ubysz A 2018 Cracked reinforced concrete walls of chimneys, silos and cooling towers as result of using, formworks. MATEC Web Conf., 146, 02002
[2] Ubysz A and Tamrazyan A G 2020 Phenomenon of cracks in chimney walls. Journal of Physics: Conf. Ser., 1425(1), 012028
[3] Dogangun A, Karaca Z, Durmus A and Sezen H 2009 Cause of damage and failures in silo structures. Journal of Performance of Constructed Facilities, 23(2), 65-71
[4] Calderón P A, Adam J M and Payá-Zaforzeza I 2009 Failure analysis and remedial measures applied to a RC water tank. Engineering Failure Analysis, 16(5), 1674-1685
[5] Rybak J 2020 Non-destructive determining of foundation pile length variability for reliability analysis. Journal of Physics: Conf. Ser., 1425(1), 012205
[6] Zee M, Chikkam A K, Larkin E, Peyman T, Rezaie A and Campbell A 2019 Corrosion risk assessment, failure analysis and corrosion mitigation for aboveground storage tanks and case histories. NACE - International Corrosion Conference Series, 2019-March, 12826
[7] Stawiski B and Kania T 2019 Examining the Distribution of Strength across the Thickness of Reinforced Concrete Elements Subject to Sulphate Corrosion Using the Ultrasonic Method, Materials, 12(16), 2519
[8] Rybak J 2014 Stress wave velocity tests in early-stage of concrete piles. Concrete Solutions – Proc. of Concrete Solutions, 5th International Conference on Concrete Repair, 571-576
[9] Moncarz P D, Griffith M and Noakowski P 2007 Collapse of a reinforced concrete dome in a wastewater treatment plant digester tank. J. of Performance of Constructed Facilities, 21(1), 4-12
[10] Maj M, Grzymski F and Ubysz A 2020 The loss of durability in reinforced concrete structures. Journal of Physics: Conf. Ser., 1425(1), 012207
[11] Gorzelanačzyk T, Hoła J, Sadowski Ł and Schabowicz K 2013 Methodology of nondestructive identification of defective concrete zones in unilaterally accessible massive members. Journal of Civil Engineering and Management, 19(6), 775-786
[12] Hoła J, Bień J, Sadowski Ł and Schabowicz K 2015 Non-destructive and semi-destructive diagnostics of concrete structures in assessment of their durability. Bulletin of the Polish Academy of Sciences: Technical Sciences, 63(1), 87-96
[13] Gorzelanačzyk T, Hoła J, Sadowski Ł and Schabowicz K 2016 Non-destructive identification of
cracks in unilaterally accessible massive concrete walls in hydroelectric power plant. *Archives of Civil and Mechanical Engineering, 16*(3), 413-421

[14] Schabowicz K and Zawiślak L 2019 Effect of geometric imperfections in the shape of buckling form on the reduction of load capacity of cylindrical shell. *Archives of Civil Engineering, 65*(3), 153-166

[15] Schabowicz K 2019 Non-destructive testing of materials in civil engineering. *Materials, 12*(19), 3237

[16] Seruga A, Faustmann D, Szydłowski R and Zych M 2012 Zbiorniki o ścianie prefabrykowanej z klejonymi pionowymi stykami sprężone zewnętrznymi cięgnami bez przyczepności. *Przegląd Budowlany, 4*, 58-63 [In Polish]

[17] EN 1992-1-1 (2004): Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC

[18] Buckling of thin – walled circular cylinders, NAS Space Vehicle Design Criteria, NASA SP-8007

[19] Safarian S S and Harris E C 1985 Design and construction of silos and bunkers. VNR 1985

[20] EN1991-4: Eurocode 1, 2006 Actions on structures. Part 4: Actions on silos and tanks.