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TECHNICAL CONDITION ASSESSMENT AND CONSTRUCTION SOLUTION OF RC DIGESTION CHAMBER STRENGTHENING

OCENA STANU TECHNICZNEGO ORAZ SPOSOBY WZMOCNIENIA ŻELBETOWEJ POWŁOKI STOŻKOWEJ KOMORY FERMENTACYJNEJ

A b s t r a c t

This paper presents the technical condition assessment of a RC digestion dome chamber after fourteen years of exploitation. The authors conducted experimental investigations, i.e. tests of concrete compressive strength using a scleroscope, concrete chemical analysis in the laboratory and detections of reinforcement. On the basis of the obtained results and cracks morphology, static-strength calculations were conducted. The results of the FEM numerical analysis indicated deficiencies in the bearing capacity of the dome. The widths of the calculated cracks exceeded allowable values. The authors proposed two alternative methods of chamber strengthening: pouring additional RC dome cover or post-tensioning the dome with external unbonded tendons.

Keywords: *digestion chamber, tank dome, tank strengthening, unbonded tendons*

S t r e s z c z e n i e

W artykule przedstawiono analizę stanu technicznego powłoki stożkowej przekrywającej zamkniętą komorę fermentacyjną po 15 latach eksploatacji. Podstawą do określenia nośności i obliczeń numerycznych były badania sklerometryczne, analiza chemiczna betonu, inwentaryzacja zbrojenia oraz powstałych rys. Wyniki analizy wykazały niedobór nośności w przekrojach charakterystycznych kopuły i przekroczenie dopuszczalnych wartości rozwarcia rys. Na tej podstawie zaproponowano dwa alternatywne sposoby naprawy uszkodzeń oraz wzmocnienia kopuły: wersję żelbetową z dodatkowym zewnętrznym płaczem oraz wersję sprężoną cięgnami zewnętrznymi bez przyczepności.

Słowa kluczowe: *cięgna bez przyczepności, powłoka stożkowa, wydzielona komora fermentacyjna, wzmocnienie zbiornika*

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1. Introduction

The basic requirement for liquid storage tanks is their tightness [1, 2]. The serviceability limit state of tanks is believed to be critical in terms of the elements dimensions (thicknesses), the quality of materials (including the concrete class and special properties e.g. water-tightness) and the amount of required reinforcement [13]. Tank designers should take into account the possibility of water-tightness loss during all stages of erection (e.g. early age cracking [8]) and operation of the structure (tank overflow, thermal loads in summer/winter season). Thus, the design process seems to be more complicated due to the additional requirements for construction materials and the technology of construction.

In most cases, the loss of water-tightness of any of the components in the reservoir means disruption in the operation of the tank. This is aimed at the protection of water purity in the water supply tanks and the prevention of polluting the environment with sewage or industrial waste water. On the other hand, any pause in plant operation leads to additional costs. The intensive development of new methods and materials for protection and repair, results in the need to formulate new standards for these kinds of works and products. Ten fundamental and sixty-five additional standards were proposed in the series of European standards EN 1504.

The article presents two alternative solutions for dome repair in the separated digester tank. The tank was out of use following eleven years of service due to numerous leaks in the conical shell. The first repair method considers the injection of cracks and pouring additional RC coat on the existing dome. In the alternative solution, the authors proposed the post-tensioning of the existing dome with external unbonded tendons. This method was implemented for the first time in Poland by the Chair of Prestressed Structures and the Laboratory of Building Materials and Structures at the Cracow University of Technology for strengthening the conical dome and the conical bed of the tanks. For each method, a simplified cost repair analysis has been prepared.

2. State of emergency

The analysed digestion chamber tank (Fig. 1) was erected in the time period between September 1997 and May 1999. The tank has been in use for more than eleven years. The flood in May 2010 resulted in long-term facility downtime (over two years), causing sludge sedimentation and clogged pipelines. In July 2012, the renovation work began which involved the mechanical removal of the old coatings by abrasive blasting (sandblasting) from the walls and the dome and the laying of a new two-component epoxy resin without any additional sealing. The contractors did not prepare the technical condition assessment of the dome surface before placing the resin coating. The repair works lasted until the end of August 2012. In October 2012, the engineers decided to fill the tank with the water at a rate of 10 m^3/h for a period of around eight hours per day. In November 2012, the first leaks on the dome surface were noticed and the administrator was forced to stop the process of tank filling. After a partial outcrop of the dome external surface, numerous cracks were discovered. The two widest cracks were oriented in a circumferential direction. Smaller cracks were located in a radial direction.

Fig. 1. The view of the two digestion chamber tanks (analysed tank on the right)

3. Tank description

The analysed digestion chamber was made using traditional technology as a monolithic, reinforced concrete cylindrical tank with an inner radius of 7.5 m and a wall thickness of 0.60 m. The bottom of the tank is a conical shell with a thickness of 0.60 m. The connection of the conical bottom plate to the cylindrical tank wall is monolithic. The tank wall is monolithically connected with a conical dome (0.25 m thick). The dome is completed with an internal RC cylindrical shell with a thickness of 0.25 m. The inner diameter of the internal shell is 4.30 m. The covering slab has a ring-shaped design. The tank geometry is presented in Fig. 2.

The wall and the dome of the tank is made of water resistant and frost resistant concrete B30 F150 W8, which corresponds to today's class C25/30 F150 W8. The concrete mixture was made with crushed basalt aggregate and Portland cement with additives corresponding to the current grade CEM II 32.5N. The nominal cement content was 350 kg/m^3 and the water-cement ratio was 0.4.

The tank wall, with a total height of 11.00 m, was erected in five sections of concreting. Each section involved concreting a two-meter high wall segment. The last 1.00 m wall segment was concreted with the conical shell. During concreting, a set of cubic samples (0.15 m × 0.15 m × 0.15 m) was prepared. On the basis of site documentation, the average cubic concrete compressive strength was 46.0 MPa; 50.2 MPa; 51.5 MPa; 43.7 MPa and 46.2 MPa (in each concreting section). Thus, the elements met the requirements of the assumed concrete class.

The dome reinforcement was made of steel grade A-III (34GS). The dome was reinforced with #10 mm bars in circumferential and radial directions. The bar spacing was 0.07–0.1 m in a circumferential direction and 0.20 m in a radial direction (on the inner and outer surfaces of the dome). The nominal concrete cover was assumed to be 40 mm.

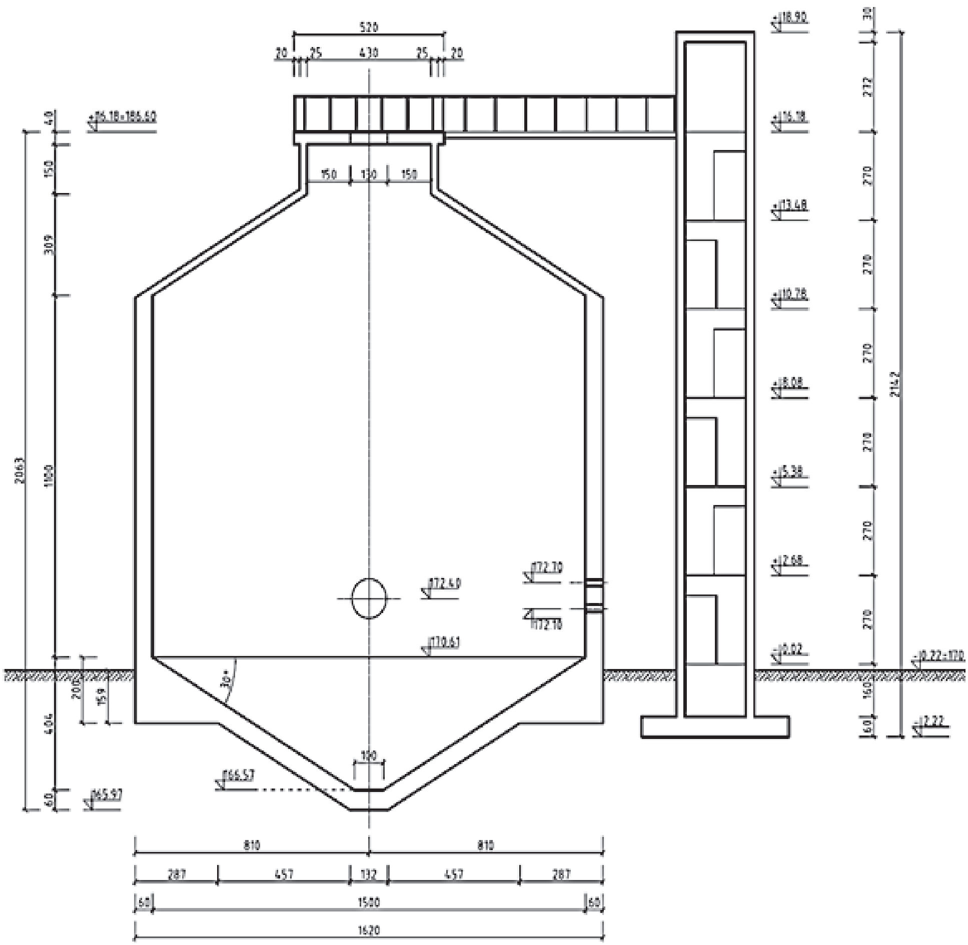

Fig. 2. The vertical cross-section of the tank

4. Experimental investigations

In order to determine the causes of the emergency state and propose the method for the restoration of the tank's chamber, an experimental test program was prepared [4, 7, 12]. The experimental studies consisted of a site investigation, sampling, and laboratory tests in the laboratory of building materials and structures at Cracow University of Technology. The experimental investigations included: an inventory of cracks; non-destructive testing methods to assess concrete strength; verification of reinforcement quantity, diameter and position; chemical analysis of the dome's concrete.

4.1. Crack morphology

The morphology and the widths of the cracks on the outer surface of the dome are presented in Fig. 3. The on-site inspections revealed that there were two main circumferential and forty smaller radial cracks. The first circumferential crack was located at the connection of the cylindrical wall with the dome and the second was at a distance of about 2.40 m from the edge of the cylindrical wall. The width of the circumferential cracks varied from 0.3 to 0.6 mm (for one crack) and from 1.4 to 2.5 mm (for the other crack). The radial cracks widths ranged from 0.2 mm to 1.1 mm. During the on-site investigation, it was found that there were prior attempts to seal the cracks before installing the insulating layer.

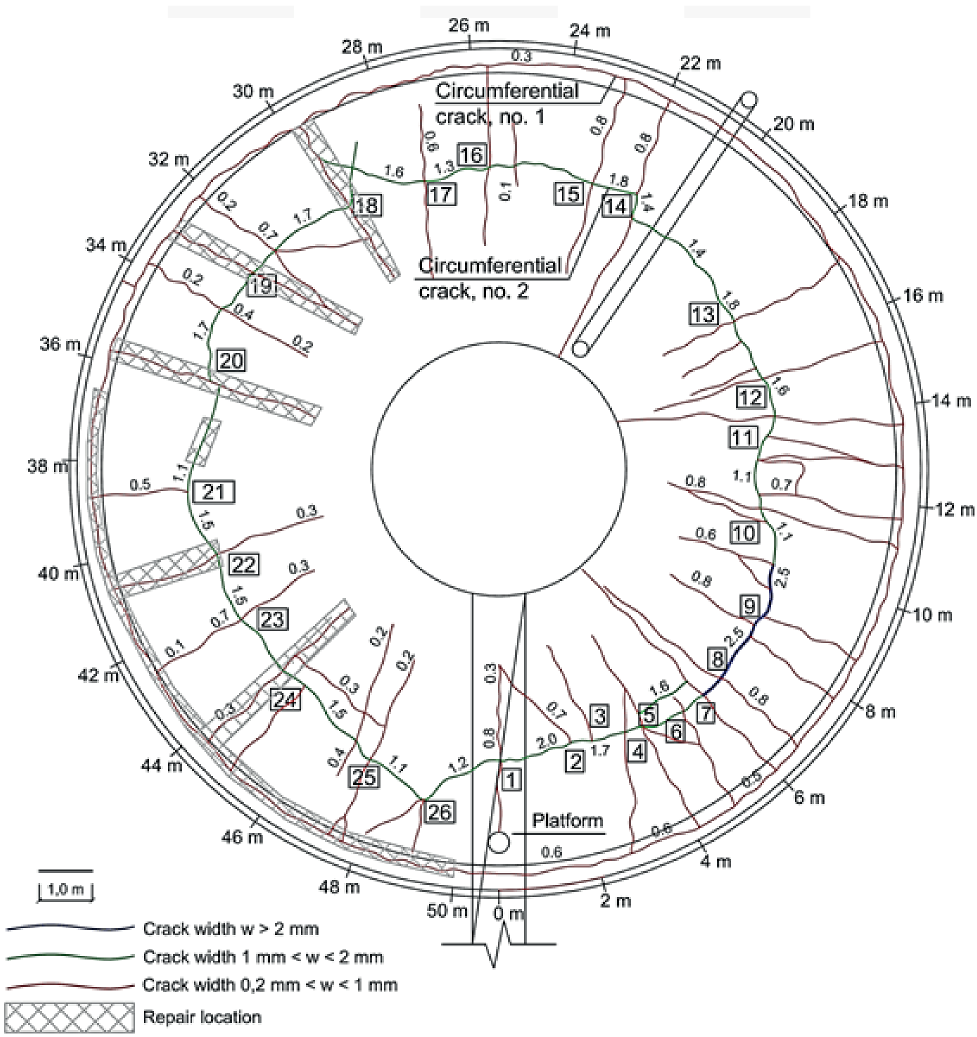

Fig. 3. The view of crack morphology on the outer surface of the tank's dome

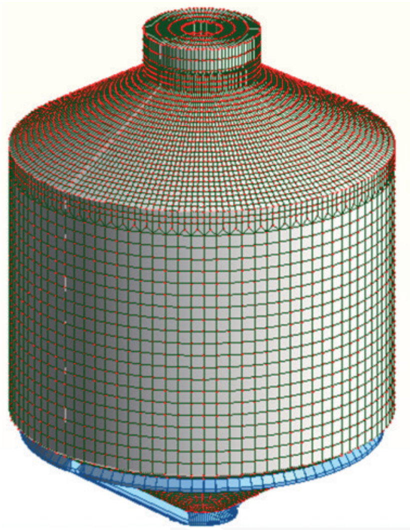

Fig. 4. The FEM model of the tank

In order to investigate the behaviour of the digestion chamber dome, a FEM model of the tank was prepared (Fig. 4). In the calculations, all possible, relevant loadings during erection and operation were taken into consideration. The digestion chamber dome was analysed: at the different stages of loading (empty, full tank, temporary overflow in the emergency situation); for different thermal loads (winter and summer season) and for different thermal isolation thicknesses.

The distribution of radial stresses on the dome's inner and outer surfaces under operation and emergency (tank overflow) loading are shown in Fig. 5. The distribution of circumferential stresses on the dome's inner and outer surfaces under operation and accidental loading are shown in Fig. 6.

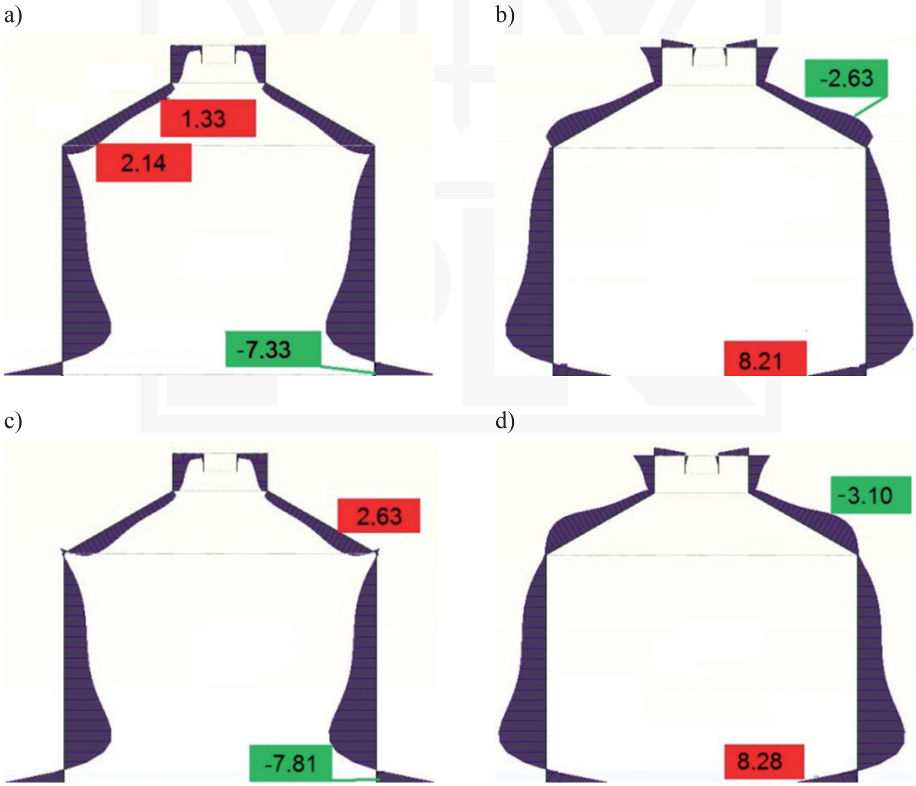

Fig. 5. The distribution of radial stresses: a), b) on the inner and outer surface during operation; c), d) on the inner and outer surface in the accidental (tank overflow) situation

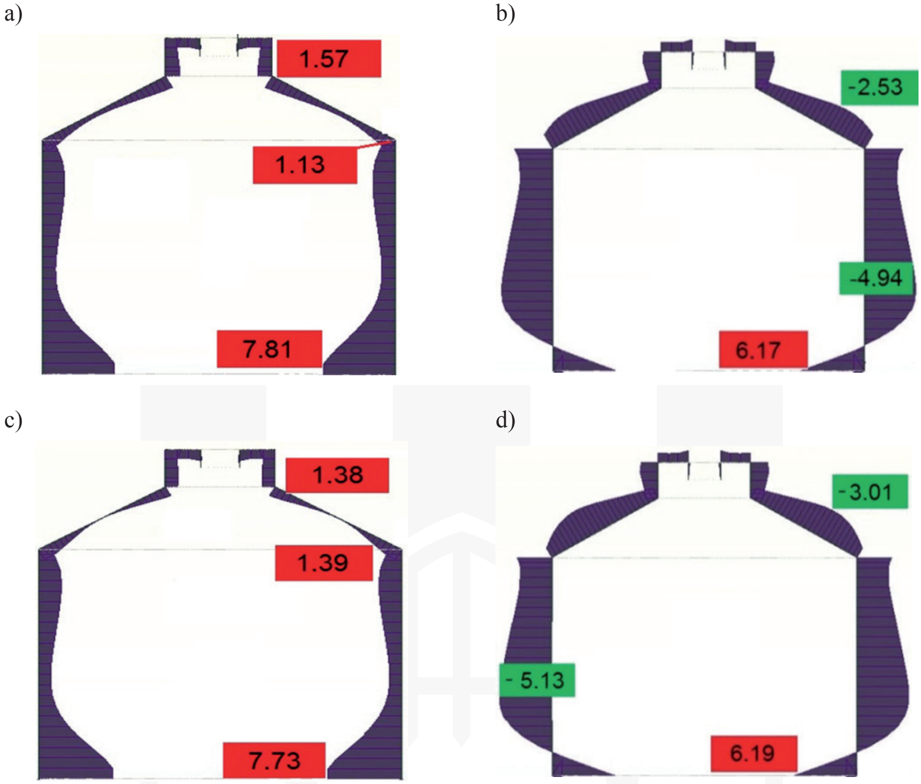

Fig. 6. The distribution of circumferential stresses: a), b) on the inner and outer surface during operation; c), d) on the inner and outer surface in the emergency (tank overflow) situation

The maximum tensile stresses in a radial direction on the outer surface of the dome were σ_r = 3.10MPa and σ_r = 2.63MPa during operation and emergency situation. In both situations, maximum tensile stresses occurred approximately 2.4 m from the outer edge of the tank wall and exceed the tensile strength of the concrete (f_{ctm} = 2.6 MPa for C25/30). In a circumferential direction, only loads acting in the emergency situation caused an excess of stresses: σ_ϕ = 3.01 MPa over the medium concrete tensile strength.

On the basis of the obtained results, it was found that the first circumferential crack (at the connection of the cylindrical wall with the conical dome) was caused by a rapid change in the shell stiffness. It is recommended to design a transitional element between the wall and the dome. Probably the second circumferential crack was developed in the winter during the first year of operation, in the line of the maximum stresses produced by hydrostatic pressure and temperature gradient. And then, the stresses at the second circumferential crack exceed the medium axial tensile concrete strength. Moreover, crack width calculations in accordance with Eurocode 2 [1, 2] resulted in a value of 0.235 mm. The crack width of 0.235 mm satisfies the serviceability limit state of normal reinforced concrete structures but does not meet crack width criteria for waterproof structures. The rapid propagation of circumferential cracks and the formation of radial cracks occurred as a result of hydrostatic pressure in the emergency state during the flood in May 2010 (clogged pipelines and tank overflow).

4.2. Concrete compressive tests using a scleroscope

Concrete compressive strength was estimated based on a series of tests conducted at twelve measuring points. The obtained results were corrected, taking into account the age of concrete, surface moisture conditions and the angle of the sclerometer during the tests. The average compressive strength determined on cube samples of 150 mm × 150 mm × 150 mm was $f_{cm.cube}$ = 31.98 MPa with a standard deviation of s = 3.56 MPa and a variation coefficient of ν = 6.68%.

4.3. Reinforcement detection

The authors used a ferrodetector to determine the position of the steel rebars, their diameter and the thickness of concrete cover. Tests were conducted at 17 measuring points and proved that the diameter of reinforcement and the spacing of rebars met the design assumptions. The thickness of the concrete cover varied from 51 mm to 77 mm.

The authors speculate that at the erection state, the contractors did not use stabilising rods for the upper and lower reinforcement mesh. The increased concrete cover thickness resulted in a decrease of the internal forces arm and a reduction in the load-bearing capacity.

4.4. Chemical analysis

The obtained results of chemical analysis showed a similar pH value in five out of six samples and varied from 11.30 to 12.00. The pH values of the water extract were higher than the minimum (pH = 10.80) which provides the protective properties of reinforcing. The pH of the specimen sampled directly from a circumferential crack area was below the limit. The results are shown in Table 1.

Table 1

The results of concrete chemical analysis

No.	Depth of concrete cover [outer layer, 0–10 mm]	pH value	Cl^- [% weight of cement]
1	K-K 1/A	11.75	0.17
2	K-K 2/A	11.30	0.23
3	K-K 3/A	11.55	0.22
4	K-K 4/A	12.00	0.27
5	K-K 5/A	11.70	0.26
6	K-K 6/A	9.95	0.33

Contamination of chloride ions in the dome cover does not exceed the limit value, which is 0.4% of the weight of the binder for reinforced concrete structures [3].

The chemical analysis of the outer concrete layer of the dome (0÷10 mm) shows that despite the existence of the thermal insulation layer (50 mm thick expanded polystyrene), the process of carbonation in the area of the first circumferential crack proceeded in a similar way as for the exposed concrete surface (RH = 40%÷60%). Thus, the insulation layer has not been properly air sealed.

5. Strengthening of the dome

On the basis of numerical analysis, two alternative methods of tank dome repair were proposed.

5.1. Additional RC shell

The first solution of repairing the dome considers pouring an additional RC dome cover. This method proposes to seal existing cracks with epoxy injection resin – this would be applied to all cracks. The additional RC shell (Fig. 7–8) is designed of concrete class C30/37 W10. The extra cover is reinforced by a single reinforcement mesh (Fig. 7–8). The designed reinforcement meets the requirements of both the ultimate limit state and the minimum reinforcement ratio due to crack width control. The contact between the new concrete shell and the dome is provided by rebars anchored in the existing dome in a direction perpendicular

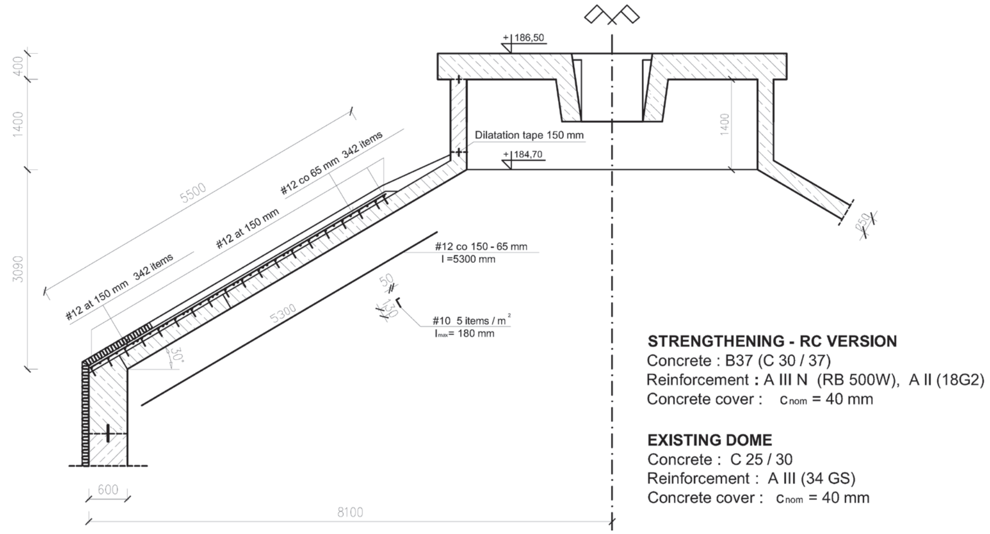

Fig. 7. Cross-section of composite shell

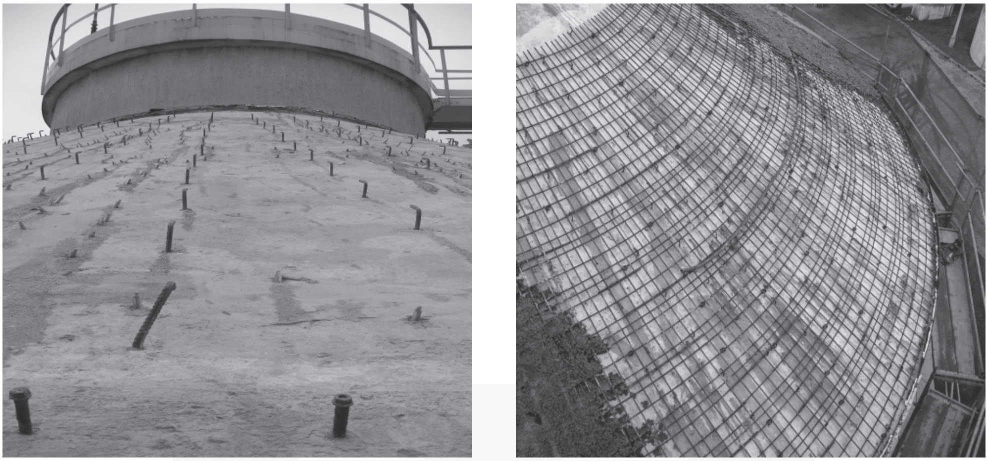

Fig. 8. Strengthening of the dome during concreting (RC version)

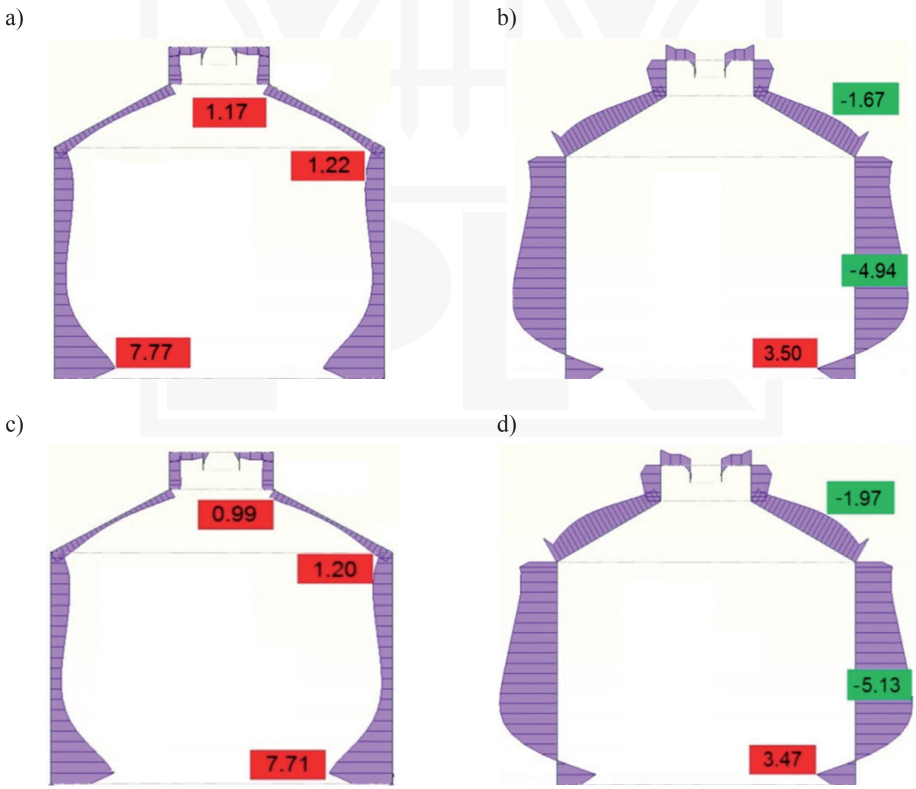

Fig. 9. The distribution of radial stresses in composite cross-section: a), b) on the inner and outer surfaces during operation; c), d) on the inner and outer surfaces in the emergency situation

to its surface. Due to the limited additional shell thickness (100 mm) and rough surface of the existing dome, it would be impossible to pour a normal concrete mixture into the mold and compact it. On the other hand, the use of self-compacting concrete requires a high bearing capacity and sealed formwork due to increased mixture pressure; therefore, it was decided to use a normal concrete mixture of consistency K2 to be applied without surface formwork. Only at the outer edge of the tank wall was a ring formwork used to prevent the concrete mixture from slipping down.

In order to investigate the behaviour of the composite cross-section consisting of a RC dome (250 mm thick) and additional RC shell (+100 mm thick) static calculations have been prepared. The distribution of the radial stresses in the composite cross-section of the dome's inner and outer surfaces under operation and emergency (tank overflow) loading are shown in Fig. 9. The distribution of the circumferential stresses in the composite cross-section on the inner and outer dome's surface under operation and emergency loading are shown in Fig. 10.

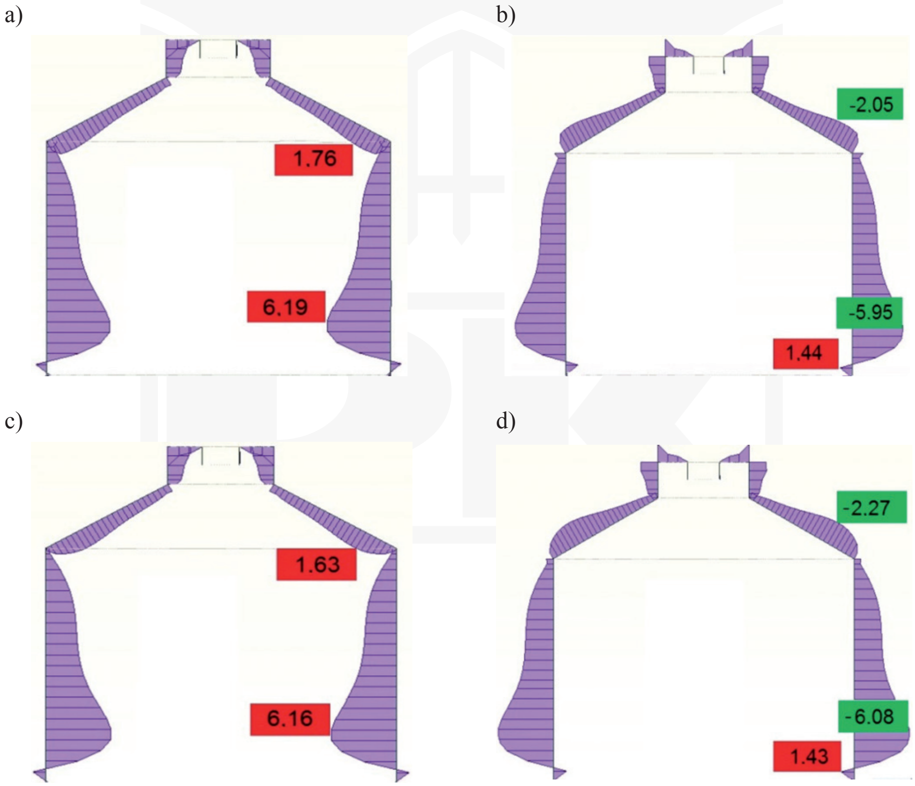

Fig. 10. The distribution of circumferential stresses in composite cross-section: a), b) on the inner and outer surfaces during operation; c), d) on the inner and outer surfaces in the emergency situation

The maximum tensile stresses on the outer surface of the composite cross-section in a radial direction were σ_r = 1.67 MPa (during operation) and σ_r = 1.97 MPa (in emergency situation). In a circumferential direction, the maximum stresses were σ_ϕ = 2.05 MPa and σ_ϕ = 2.27 MPa respectively during operation and the emergency situation. The stresses in the concrete composite shell do not exceed the mean value of concrete tensile strength (f_{ctm} = 2.9 MPa for the concrete class C30/37). Moreover, pouring an additional RC dome cover does not result in a significant increase of tensile forces in the circumferential ring around the tank's dome.

5.2. Post-tensioning of the dome

The second repair method considers the use of steel deviators and external unbonded tendons for post-tensioning of the tank's dome [9–12, 14]. This would provide compressive stresses in the tank's dome cross-sections. It is proposed to seal the forty smaller existing radial cracks with polypropylene injection resin and the two main circumferential cracks with epoxy resins. After injecting and filling the tank, the first tightness test should be carried out.

The static-strength analyses included: selection of the tendons and dimensioning additional elements-deviators, calculations of prestressing force and losses, verification of efficiency in terms of Serviceability Limit State and Ultimate Limit State.

Due to the necessity of limiting post-tensioning force losses, the authors decided to use unbonded steel tendons 7ϕ5. It was found that the RC tank's dome should be prestressed in a circumferential direction with a tendon length of up to 32 m. Tendons with a nominal diameter of 15.7 mm and cross-section area A_{p1} = 150 mm^2 (Y1860S7, f_{pk} = 1860 N/mm^2) were installed in plastic ducts. In order to ensure the durability of the tendons and minimize contact stresses, a four-layer protection system has been introduced. The system consists of: a petroleum grease; a leak-tight, corrosion-resistant plastic duct; mortar grout (between the duct and the plastic pipe); plastic pipe.

The required post-tensioning force was estimated on the basis of static-strength analyses conducted during the assessment of the technical condition. It was required to use circumferential tendons, each tendon post-tensioned with the force of 220kN. The prestressing force was introduced to the FEM model [5, 6] as an equivalent concentrated load consisting of horizontal and vertical components (slope of the dome α = 30°). The calculations also take into account the cam of prestressing force (average moment M_{Pmt} about 12.4 kNm).

The stress state analyses concerned:
- initial situation – empty tank in the winter season,
- operation situation – filled tank in the winter season,
- emergency situation – tank overflow.

The post-tensioning forces should provide compressive stresses in the dome during all operation stages (full and empty tank). Only in the emergency situation are tensile stresses allowed to occur, but their value should not exceed the medium tensile strength of concrete ($\sigma_t \leq f_{ctm}$). The calculated post-tensioning force after immediate losses was 197 kN and after long-term losses, it was 186 kN. Spacing between the tendons conform to tensile force distribution (Fig. 11) and vary along the conical dome.

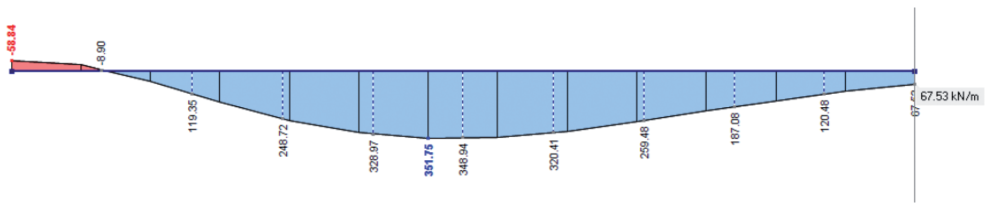

Fig. 11. Circumferential tensile forces due to hydrostatic pressure in the emergency situation

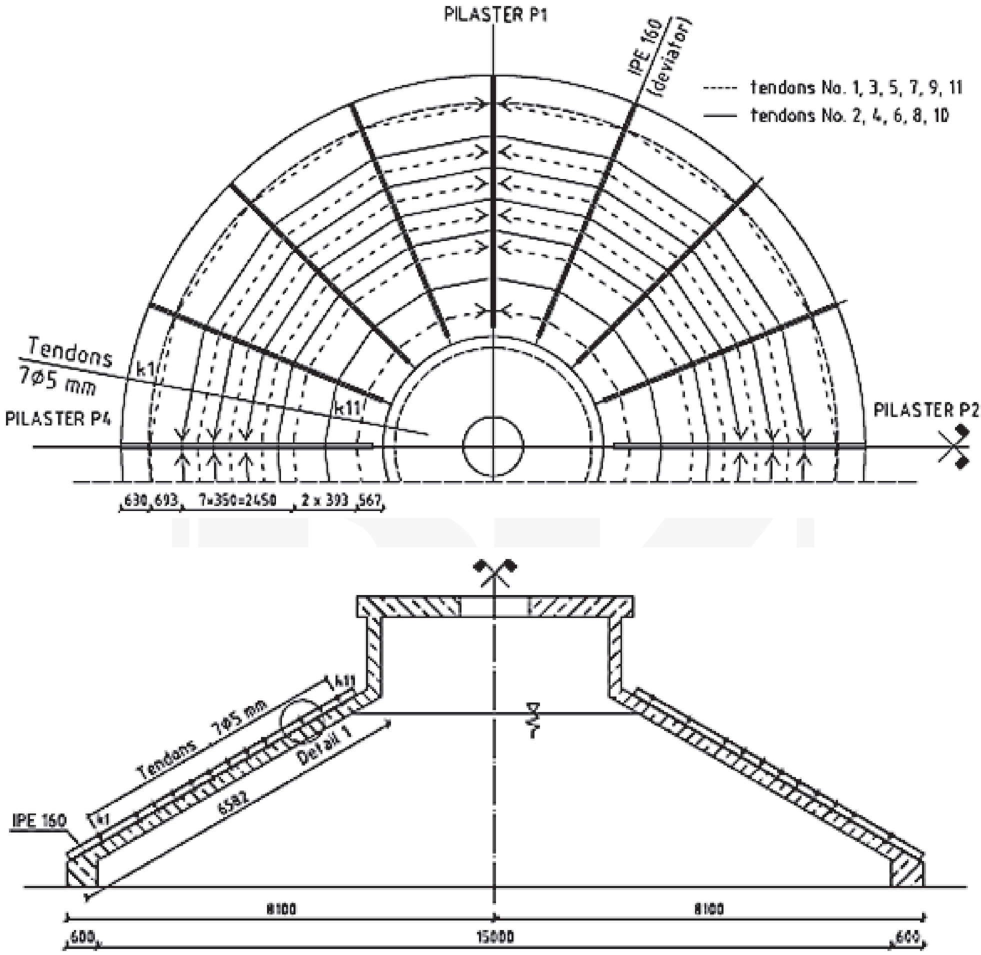

Fig. 12. Arrangement of deviators, pilaster and the tendons

In order to transfer the prestressing force to the dome's cross-section, steel deviators were designed. The deviators were made of 16 steel profiles IPE 160 and four box-section beams. The elements attached to the reinforced concrete dome using anchors. The dome has been divided into 16 parts, each with a cone angle of 22.5°. Four box-section beams were used as pilasters for the anchoring of tendons. The beams were designed symmetrically every 90°. Due to increased losses of prestressing force in two peripheral tendons, each circuit has been divided into 2 sections anchored in four pilasters. The rest of the circuits consisted of only one section anchored in two pilasters.

All tendons are stressed from one side only using a hydraulic jack and are fastened by wedge grips at the live end, as shown in Fig. 13, and swage type anchorages at the dead end. Spacing between the tendons varies from 400 mm in the area of maximum circumferential tensile stresses to 800 mm in the edge zones (Fig. 12). In the places provided for the location of tendons, the web of the profile has been cut and the additional components consisting of teflon coated curved steel sheets have been installed (Fig. 14).

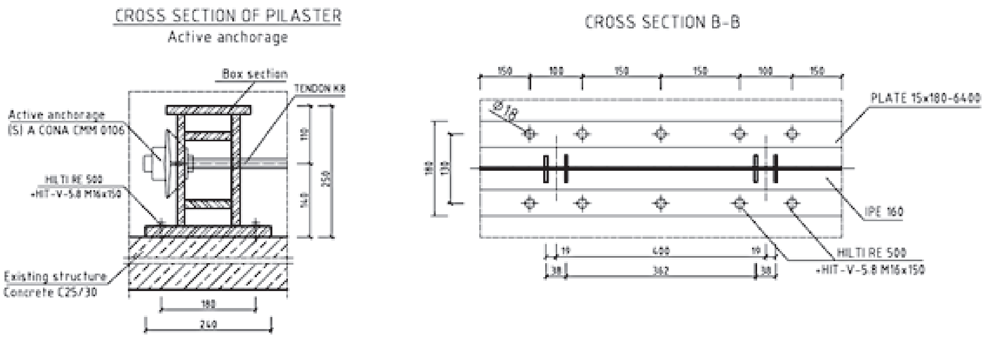

Fig. 13. The detail of the anchorage design solution

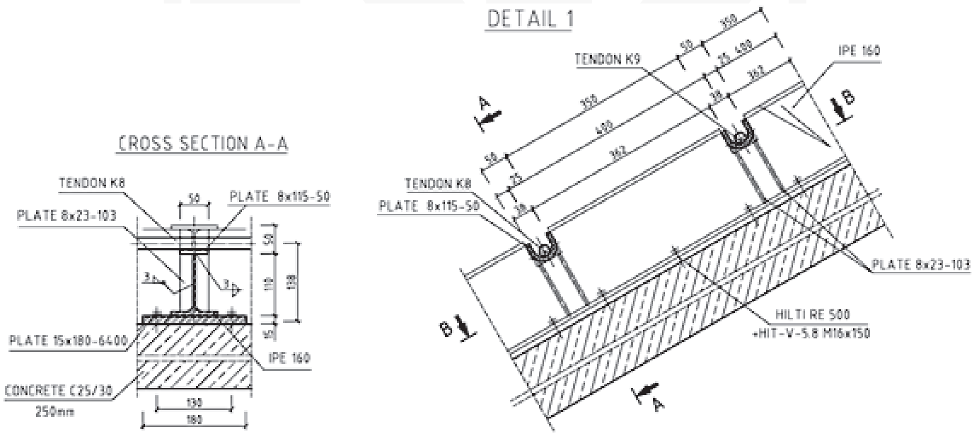

Fig. 14. The detail of the steel deviator

It has been found that the maximum tensile stresses occur in the emergency situation (during tank overflow) in the centreline on the outer surface of the dome and are equal to σ_{ct} = –2.02 MPa (in a radial direction). Tensile stresses in other parts of the dome vary from

–0.10 MPa to –1.45 MPa in the initial situation and from –0.32 MPa to –1.86 MPa during the tank's operation. The maximum compressive stresses are σ_{cc} = = 3.74 MPa (in a circumferential direction).

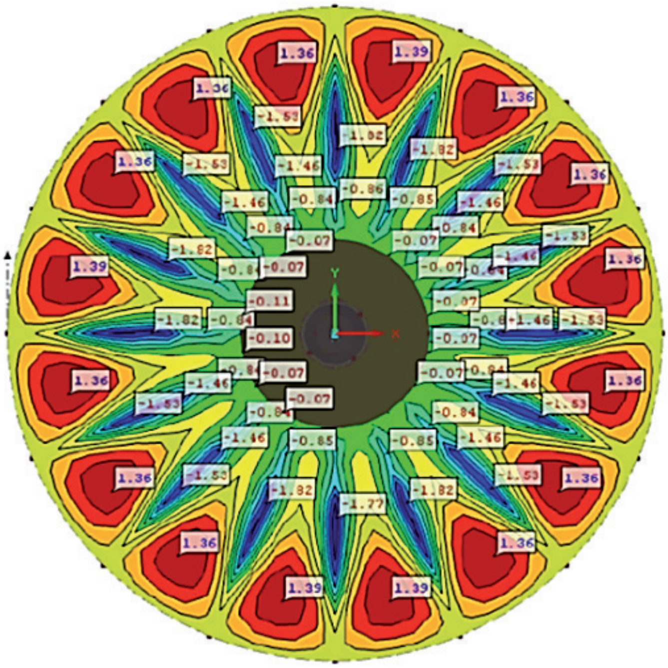

Fig. 15. The distributions of circumferential stresses on the outer surface of the tank's dome in the emergency situation

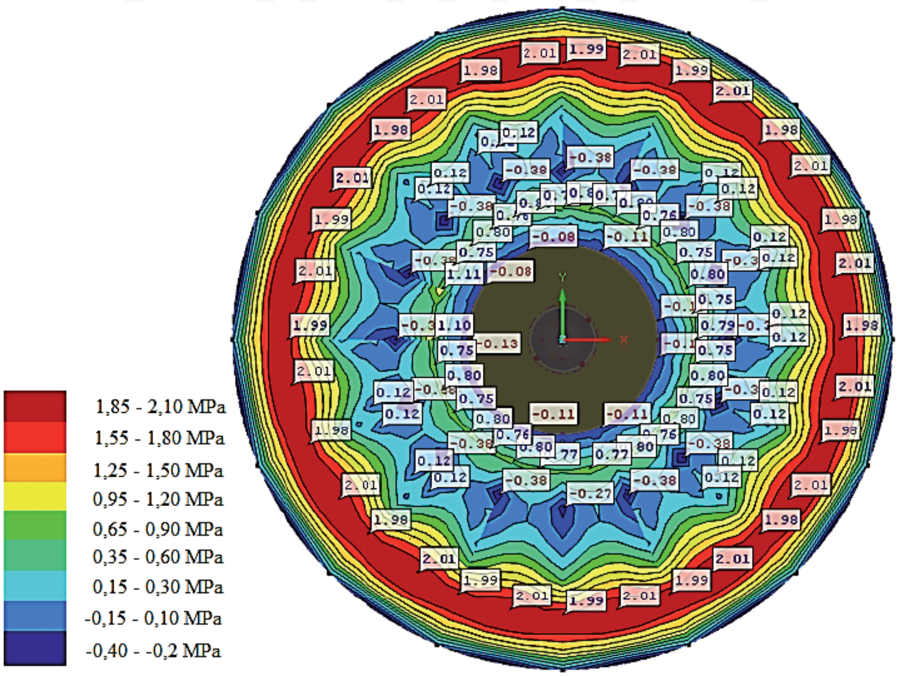

Fig. 16. The distributions of radial stresses on the outer surface of the tank's dome in the emergency situation

The distributions of circumferential and radial stresses on the outer surface of the tank's dome in the emergency situation are shown in Fig. 15 and Fig. 16.

The tensile stresses are lower than the medium concrete tensile strength. Therefore, reopening of the cracks is unlikely.

6. Conclusions

The assessment of the technical condition of the existing structure revealed an underestimation of the required reinforcement at the design stage. The reinforcement ratio did not meet the requirements of minimum reinforcement according to EC2. Moreover, an insufficient thermal insulation layer created additional stresses in the dome cross-section. The proper strategy of thermal insulation in terms of its thickness and materials plays a vital role in the static-strength behaviour of the tanks subjected to temperature gradients. In order to improve the thermal properties of the partition, thermal insulation layer was increased to 120 mm for both versions of the repair.

Maintaining tightness throughout the life of the structure is a problematic task in the case of pouring an additional RC shell due to the occurrence of tensile stresses in the cross-section. In the post-tensioned dome, prestressing forces produce compressive stresses in the dome cross-section. On the other hand, the additional shell method is simpler in terms of contractors' qualifications and the required equipment. However, concreting can take place only under favourable weather conditions (absence of rainfall and air temperature similar to the temperature of the tank – elimination of thermal stresses at the connection surface). This is an important factor because it determines the period of the tank's downtime – this has significant financial implications for the investor.

In both versions, special attention should be paid to proper sealing execution. This is particularly important in the case of pouring an additional RC shell. Thus, the works related to the sealing of the structure must be completed with tank tightness testing.

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