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Research paper

Expansive soils as the root cause of a historic underground potable water reservoir failure

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Abstract: The paper describes the root causes of damage to an underground potable water reservoir. It is made of reinforced concrete and consists of two chambers founded on subsoil diversified for its age and origin (folds of glacitectonically disturbed septarian clays and postglacial loams). The documented damage results from subsoil deformation caused by volumetric changes in the ground (shrinkage and swelling) due to the facility's exposure to various external factors determining the phenomenon's intensity for over one hundred years. The analysed example presents problems arising from founding the facility on expansive soils. It highlights the hazards resulting from negligence in landscape architecture development and emphasises the role of identifying the subsoil. Comprehensive geotechnical examination and geophysical measurements of the structure and subsoil, making the inventory and evaluating the nature of the damage helped identify the potential factor detrimental to the facility's technical condition. The conclusions are confirmed by numerical simulations, which include the impact of swelling and cyclic loads resulting from the facility's function and surroundings.

Keywords: expansive soils, subsoil examination, geophysical methods, structure diagnostics, historic structure

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1. Introduction

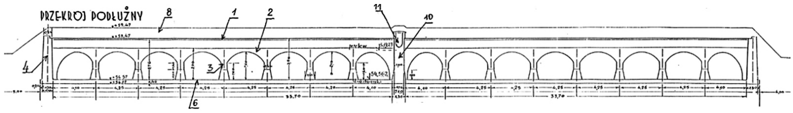
Numerous examples of structures where cracks, fractures and other types of defects that lead to failures are known from geotechnical practice; they result from founding the structures on expansive soils [3, 10, 12, 12]. The examples apply to various situations where the presence of expansive subsoils (e.g. Neogene clays) at the building structure founding level results from the sequence of layers and natural erosion processes. Sometimes, it depends on the current morphological situation (e.g. embankments and slopes) or is related to local uplifts. The latter ones result from glacitectonic disturbance where "older" deposits are subject to spalling, displaced and pushed into "younger" deposits (Pleistocene postglacial formations) [9]. Shallow depths of expansive soils, sensitive to humidity fluctuations determining volumetric changes, often occurring directly at the foundation level, constitute a severe geotechnical problem. The sensitivity of the soils to temperature-humidity balance impairments is strictly related to changes in the stress conditions triggered by such factors as uplift, swelling pressure and shrinkage. In Poland, the concept of expansiveness described in the literature [1, 6, 7] applies primarily to Neogene (Tertiary) marginal and marine clayey formation complexes [9]. The sensitivity in Tertiary clays results from their mineralogy and origin [6]. Expansive properties are triggered by exposure to natural and anthropogenic factors such as climate, plants (trees), drainage, underground infrastructure, and groundworks (excavation). This leads to local volumetric changes in the subsoil. These changes cause vertical displacement of the subsoil and may damage. Furthermore, through the foundation, they can lead to an emergency condition and – potentially, in some cases – to a disaster.

The presented example applies to a situation where the presence of uplifted Oligocene (Septarian) clay at the foundation level is the root cause of damage to the structure of a historic underground reservoir. Various external factors contributed to releasing the subsoil expansiveness potential in the referenced clay.

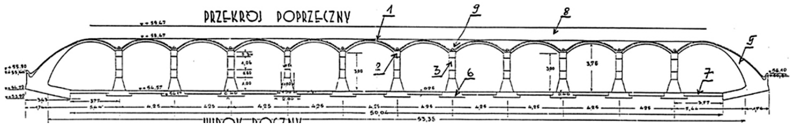
2. Structure characteristics

The analysed structure is an underground reservoir divided into two independent chambers (chamber I – eastern part of the reservoir and chamber II – western part of the reservoir), each 5000 m^3 in volume (Fig. 1 and view in Fig. 5). The reservoir was built as a reinforced concrete structure in the early 19th century (the original design documentation is missing). The date above the entrance to the reservoir, which was initially used as housing for a water-supplying canal, reads 1927.

The reservoir was made in an open trench and backfilled with the excavated soil (clayey fill). The backfill layer above the key course is ca. 1 m thick. Drain pipes (0.15 m diameter) run along the spandrels at the abutments and then along the side outer walls (in axes I). Their role is to drain rainwater. The ground above the reservoir is well-maintained and covered with grass. The entrance and ventilation holes are the only elements seen on the surface. The terrain slopes gently towards the south and southwest at the reservoir's southern part. The slopes around the reservoir do not reveal any signs of impaired stability or other mass movements (e.g. creeping). A retaining wall made in 2011/2012 in relation to the adjacent street's extension runs within



(a)



(b)

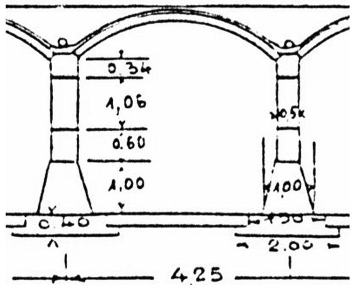
Fig. 1. Cross-sections of the reservoir structure: (a) lengthwise section in the binding joist axis (chamber I exposed); (b) cross-section according to a drawing from 1949 based on [4], describing the structure components: 1 – arch vault, 2 – arch binding joint, 3 – column, 4 – outer wall, 5 – outer arch-like wall (vault abutment), 6 – continuous footing, 7 – bottom plate, 8 – cover layer (fill) above the reservoir, 9 – drainage (stoneware pipes) in the spandrels, 10 – central wall, 11 – unused canal supplying water to the reservoir chambers

the plot's border at the west. The area around the reservoir (which is significant from the viewpoint of the described phenomena) is overgrown by (fruit) trees at the slope.

According to measurements from 1960, the reservoir's inner dimensions are 71.0×56.09 m (projection). The reservoir's bottom plate foundation level is at 54.3 m ordinate, while under the continuous footing, it is at 54.2 m above sea level. The structure's ceiling level at the highest vault points is ca. 58.5 m above sea level. The reservoir's essential components include cylindrical vaults, binding joists, columns, outer walls and the bottom plate. The load-carrying structure consists of reinforced concrete cylindrical vaults resting on reinforced concrete binding joists (intermediate supports) and extreme walls formed as a massive abutment for the extreme vault. The walls are 0.24 m thick; the column centre-to-centre span is 4.25 m (4.10 and 3.77 m, respectively, in the longitudinal and cross-section at the walls), and the height does not exceed 3.76 m. The binding joists, which have a 4.25 m centre-to-centre span, rest on reinforced concrete columns and arch pilasters in the walls (Fig. 2a). The columns, forming a steep head, are founded on a reinforced concrete continuous footing that is 0.40 m thick and 2 m wide (cross-section). The column's specific width is 0.50 m and 1 m at the base; the vault is 3 m high, and the height within the binding joist's clearance is 2.66 m (Fig. 2b).

The outer reinforced concrete walls, in axes 1 and 13 (lengthwise against the chamber) forming arch abutments of the vaults, serve as retaining walls due to water and earth pressure. The outer reinforced concrete wall in axis I (crosswise) are formed as massive reinforced concrete walls that take over the earth pressure owing to the upper support of the vaults (0.3 m wall thickness) and lower support of the bottom plate (1.3 m wall thickness) – Fig. 1. The reservoir is separated by a 1 m thick middle wall (axis A) with an unused filling channel in the upper part (Fig. 5). A 0.25 m thick reinforced bottom plate resting directly on the ground is the reservoir's bottom. Towards axes 1–13, the plate rests on continuous footing running under the columns. The reservoir's interior finish is a mineral layer of water-tight cement rendering (fired and rough floated). The apparently sophisticated method of developing the structure and its items was practical rather than purely aesthetic. Its construction period coincided with the

(a)



(b)

Fig. 2. (a) Sample general view of the reservoir's essential structural components (photo by T. Godlewski); (b) dimensioned diagram on the binding joist axis according to a drawing dating back to 1949 in [4]

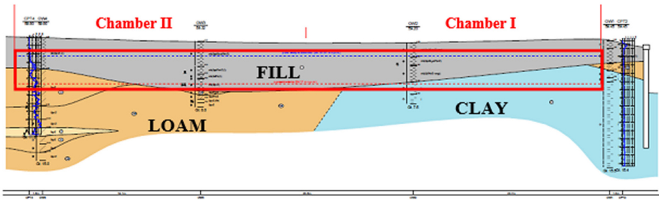


Fig. 3. Characteristic geotechnical cross-section of the reservoir, W–E direction [4]

global financial crisis, resulting in limited availability and high steel prices. Arches and vaults were meant to solve the problem by maximizing concrete capability and measurable savings on reinforcement due to variable cross-section.

3. Soil and water conditions

The reservoir is located in a young glacial landform area within a moraine edge and documented glacitectonic uplifts. This determines the subsoil structure – a moraine hill of Pleistocene glacial deposits (loam and sand), within which folds, xenoliths and floats of Oligocene Septarian clays are present (results of glacitectonic displacements). The initially planned test numbers and locations (CPTU probing and drilling) were extended to cover additional drilling meant to precisely identify the course of glacitectonically displaced Septarian clays. They were completed with field tests using a flat dilatometer (DMT) and laboratory identification of the swelling pressure due to the need to determine the soil characteristics for its deformation range (potentially expansive soils). In order to determine the dampness of the made ground, shallow (1 m) drilling and trenching were made around the reservoir walls and on its ceiling. The geotechnical tests revealed that fine (cohesive) marginal soils represented by rigid-flexible Oligocene clays and postglacial soils in the form of rigid-flexible sandy loams and semi-cohesive clayey sands are the direct foundation of the analysed underground reservoir. Postglacial soils are deposited in the structure's western part. At the same time, Oligocene Septarian clays form the subsoil of the reservoir's eastern part (Fig. 3). Severely damped areas at the reservoir slope base were discovered within the fills.

During drilling, a continuous groundwater level was not discovered in the reservoir's bottom (within the exploration depth up to 15.5 m below ground level). Groundwater was only discovered as filtration in the sandy interbedding and fill within clayey sands and loams. Measurements done with the P1 piezometer (in clays) and the determined chemical composition suggest that the water originates from rainwater infiltration and/or infiltration from the reservoir through the fill levels. The mean water level in the P1 piezometer is close to the ordinate corresponding to the water level in the reservoir, potentially suggesting hydraulic contact. Measurements in the P2 piezometer (in loam) reveal a stable water level. According to the soil profile, the water comes from intra-loam filtration and precipitation infiltration. There is no apparent link between the water level in the P1 piezometer and that in the filled tank. Chemical tests revealed that the water composition corresponds to standard groundwater composition in urbanised areas. The documented soil under the ground can generally be described as a load-carrying substrate for the structure. Still, considering various characteristics, including diversified strain characteristics and expansion potential, in the context of geomorphological and geological (glacitectonic displacement) determinants and the potential geodynamic processes (risk of mass movements in the area), the reservoir subsoil conditions shall be considered complicated. According to the literature [9] and own study results [4], Septarian clays have low values of the internal friction angle. In case of damping or contact with water, the soils are easily plastified due to water absorption). In such a case, considering a very high potential expansiveness (PE), a very high degree of expansion (DE) and swelling potential ($> 25\%$), as well as the swelling pressure (30–300 kPa according to literature [6] and 25–50 kPa according to own spot tests), the soil volume can increase several times as a result of its swelling. Field and laboratory tests helped determine geotechnical parameters for further analyses of the geotechnical conditions of the reservoir foundation.

The issue of building defects related to their founding on Septarian clays is well-known and documented [11, 12]. Non-uniform deformation of swelling or shrinking clays leads to deformation and damage to building structures. According to Tarnawski [11], the common practice of adjusting and loading the slopes with fills whose permeability exceeds that of clays several times leads to the accumulation of groundwater at the fill bottom and plasticising the clay floor, along which phenomena leading to landslides in extreme cases occur. The analysed area is predestined to phenomena related to mass movement risks. It results from the coexistence of several passive and active factors where geological structure (the presence of clays), slope of the land and weather phenomena play the most significant, though not the only, role. Weather factors, vegetation in the area, rainwater infiltration rate, and anthropological land development will influence the intensity and rate of the phenomena (creeping, landslide). Mass movements in the areas at risk can be caused by natural and anthropological factors, e.g., defective engineering works such as drains, slope undercutting, and slope profiling, poorly performed construction works (e.g., water and sewer systems), slope loading with building infrastructure, or depriving large areas of permanent vegetation [11]. In this context, when evaluating the potential causes of the analysed facility's defects in relation to the geotechnical founding conditions and in addition to the complex arrangement of the subsoil layers, one should consider current geodynamic processes and periodical and cyclic active phenomena related to subsoil expansion, e.g. as a result of slope exposure or hydrological draught when plants are present (tree roots).

4. Measurements and observations outside the facility

The outside observation covered measurements on the previously mounted inclinometer columns around the reservoir. An analysis of the collected data revealed a displacement of the facility and its direct surroundings. A significant ca. 20–30 mm increase was reported in two years. The resultant horizontal displacement is oriented southwards and southwestwards according to the natural slope inclination. Setting out of the reservoir, displacement (settling) was made available, and the retaining wall was measured for assessment purposes [4]. The reservoir's horizontal and vertical displacement measurements revealed a linear shift of 1–9 mm and settling by 1–3 mm. One should note that the measurements applied to the structure on the surface and ground surface benchmarks (on the reservoir's ceiling). Still, the resultant displacement direction and values comply with the inclinometer measurement results for the same measurement period.

The direction and successive reported values of the benchmark replacement measurements for the retaining wall correspond to the expectations for such structures (bracket displacement). Linear displacements for the benchmarks located on the retaining wall reveal 1–6 mm differences and settling up to 6 mm. The analysis of the collected data suffered from a certain cognitive dissonance as the measurements were relative. It was impossible to refer the measurements to the construction period of the retaining wall, and hence the information was missing on the total displacement and the adopted acceptable values. Nevertheless, the currently observed traces of the retaining wall repairs (Fig. 4) testify to the structure's excessive strain (exceeding the acceptable values).

Fig. 4. Traces of repairing the retaining wall (ca. 5–6 m high) defects (vertical cracks); the wall is located along the reservoir's eastern part (photo by T. Godlewski)

5. Measurements and observations inside the facility

The works inside the facility were aimed to make an inventory of defects and verify the findings concerning the reservoir's structure condition. Setting out results referring to the measurements of the reservoir structure gap opening (bottom plate, vault and arch lintels) on the applied benchmarks (bolts on both sides of the gap – Fig. 7a) were made available for the assessment purpose [4]. Based on the measurements (four sessions during four years), it was hard to unequivocally determine the size of the measured changes due to the missing information about the measurement accuracy. Nevertheless, all benchmarks indicated changes suggesting crack closing and opening within the $-0.30 \div +0.31$ mm range for the bottom plate and max. $-0.24 \div +0.61$ mm for the vaults and lintels.

A detailed inventory of each chamber defects (made temporarily available after pumping out the water) was made before the geophysical measurements. Sketches were developed, openings measured and detailed photos taken [4]. Based on the above, the reservoir's technical condition was determined to be diversified. Referring the inventory results to the findings of 2004 (for chamber I, left) and 2016 (for chamber II, right), one can observe the progressing degradation of the reservoir's structure, though of various origins and intensity – Fig. 5. New fractures (not previously described) and cracks in the repair areas were discovered in chamber I. The cracks continue in the bottom plate along the reservoir's entire length in the eastern part (axes I, H, G to F) and crosswise on the axes 10–13 and between axes 2–3. The crack opening characteristic widths reach 1.5–2 mm (up to several mm locally) and are often filled with crushed material with numerous fine branches (Fig. 6a). Cracks on the binding joists, running towards the vault top, typically in the repair areas and often with intense lime efflorescence, up to 1.4 mm opening width, are characteristic of the chamber I (e.g. Fig. 6b).

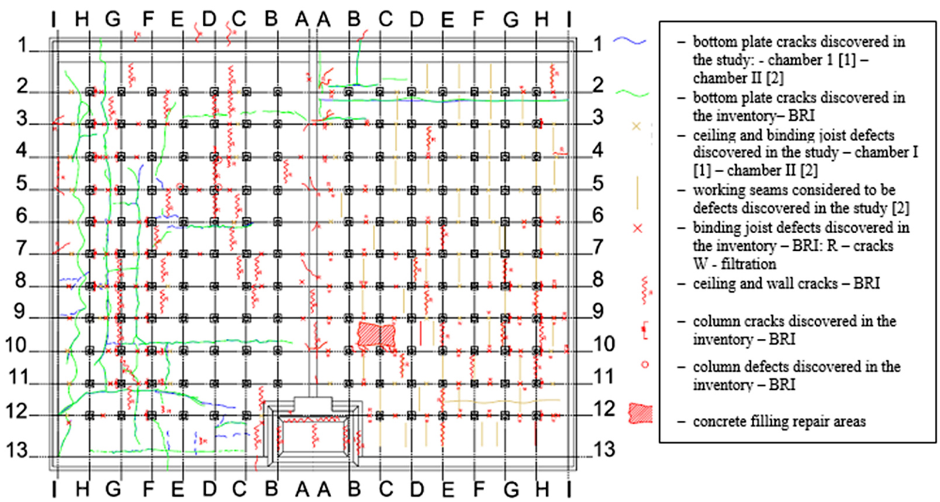


Fig. 5. Inventory of the reservoir defects [4]

(a)

(b)

Fig. 6. (a) Bottom plate in chamber I, slanting cracks, locally with crushed material and branching, opening width ca. 0.9 mm; (b) binding joist in the H-G/8 axis in chamber I – a horizontal crack in the repair area, with efflorescence, opening < 0.3 mm, running vertically towards the vault (photo by T. Godlewski)

In some areas, mutual vertical displacement of the crack edges (offset) was visible – Fig. 7a. Characteristic tight cracks could be observed in the column areas at the column head widening; they reached no further than half the column width (Fig. 7b). The defects came in pairs on two adjacent columns in the direction of the numerical axes and applied to almost all columns in the H, G, and F axes (Fig. 5). Vault defects occurred all over column I, prevailing in the central part. In most cases, they are very fine cracks with lime efflorescence and in the repair areas.

(a)

(b)

Fig. 7. (a) Bottom plate in the G-F/9-10 axis in chamber I – monitored gap in the repair area, opening width > 2 mm, vertical displacement (offset) visible; (b) column in the H/2 axis (view from axis 3 side) in chamber I – a horizontal tight crack at the column widening area, reaching no further than half the column width (< 25 cm), crack opening ca. 4 mm wide with efflorescence (photo by T. Godlewski)

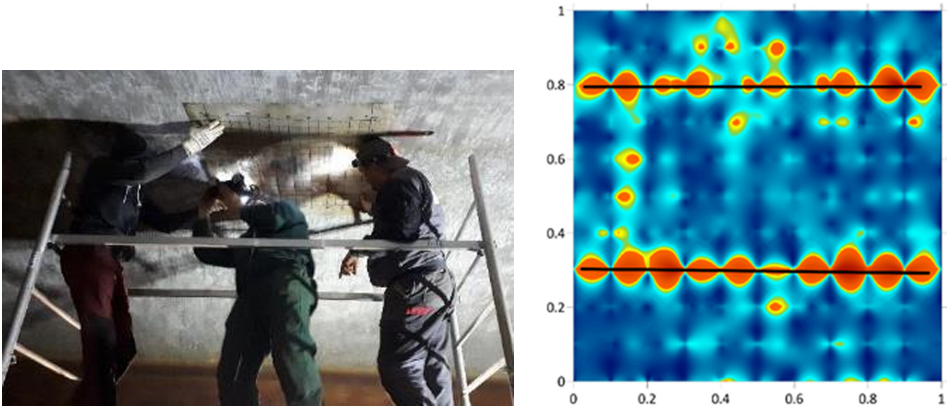
Cracks were also observed on the outer walls at the vault or pilaster edges. A significant number of cracks in the bottom plate, visible traces of shearing and vertical movements of the cracks, combined with the measurement results on the applied benchmarks and the nature of the column defects (compression?) suggest intense work of the structure in this part of the reservoir under stress with variable impact vectors.

New cracks versus the 2016 inventory were discovered in chamber II, but their scale is minor. The confirmed defects can be attributed to the structure's nature and age. Due to the structure's continuity, some defects are likely to be linked to the situation in chamber I (e.g., shared cracks on the inner wall or new defects of the columns). Fine cracks running across the vault axis were observed in cylindrical vaults. Typically, they are a continuation of the cracks in binding joists, frequently with severe efflorescence and visible damping, occurring at technological gaps (working seams). Considering the need to maintain the facility's function, a decision was made to measure the geometry, material strength and geophysical parameters of the reservoir structure's selected components using non-invasive methods. They confirmed previous findings on defects' existence and nature, the structural components' geometry and their low strength. Sclerometer tests (a collection of ca. 700 measurements) revealed insufficient homogeneity of the tested concrete (variability index > 20%). The binding joists and vaults are characterised by the highest estimated strength (ca. 34 MPa on average), whereas it is ca. 20–24 MPa for the walls and columns. The lowest strength value (ca. 14 MPa) applies to the floor and bottom plate. The quoted mean values for the bottom plate are similar to the strength value determined based on non-destructive tests carried out under the previous assessment (ca. 11 MPa).

Georadar tests (GPR) were performed to verify the reinforcement rate of the structure components; for the bottom plate and the subsoil, they were meant to identify non-homogenous and excessive damping areas. The concept of the GPR method is described in detail, e.g. in [2], [5] and [8]. The signatures of GPR anomalies in the subsoil include reflective horizons, diffraction hyperbolas and areas of the recorded wave signal amplification. The GPR measurements were performed with 1.6 GHz and 750 MHz antennas all over the bottom plate's surface and in selected areas of the ceiling (vault), columns, floors and binding joists. A total of 700 GPR profiles were made in the reservoir. Three hundred had cruciform arrangements, 0.5 m spacing, and a prospection range of ca. 1.2–1.4 m (linear measurements). Four hundred profiles also had cruciform arrangements, 0.1 m spacing and a prospection range up to 0.5 m (spot measurements) – Fig. 8.

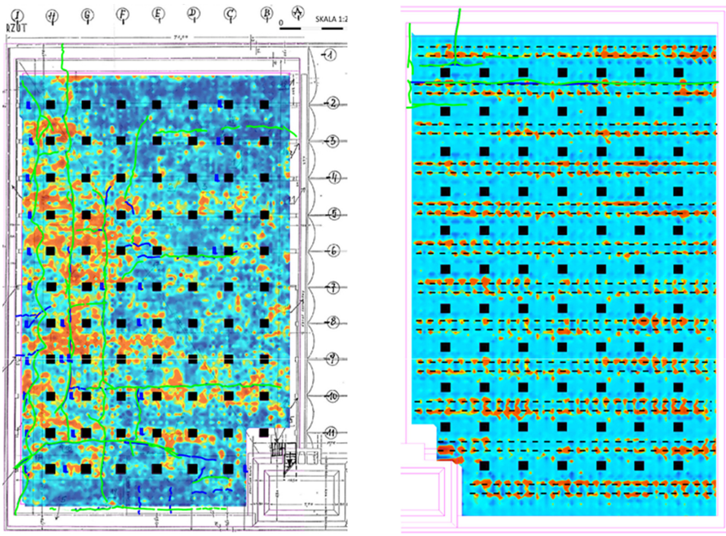
Numerous superficial areas of electromagnetic wave signal amplification were observed in the reservoir's bottom plate in chamber I, which is a testimony to the plate's non-homogeneity and cracking. It is confirmed by observations, whereby the GPR anomaly zone is much broader than the charted cracks – Fig. 9a.

This can suggest damping of the plate and material deterioration (concrete corrosion) much more advanced than expected based on observing visible cracks. The increased dampness zone in the bottom plate overlaps with the location of the prominent cracks identified in the reservoir structure. The GPR image shows linear, regular anomalies originating from the reinforcement. The echogram interpretation revealed that the reinforcing bars are not regularly spaced. The structure is reinforced but to a minimum extent. GPR measurements indicated the reinforcement



(a) (b)

Fig. 8. (a) View during the vault's GPR test (1.6 GHz antenna) in a cruciform arrangement, 0.1 m spacing, prospection range up to 0.5 m (spot measurements), (Photo T. Godlewski); (b) sample imaging result (prospection at 0.1 m depth) as a map of anomalies, regular anomalies (electromagnetic wave signal amplification areas) represent reinforcing elements [4]



(a) (b)

Fig. 9. View of the reservoir's bottom plate – map of GPR anomalies, 750 MHz antenna, against the cracks discovered on the surface: (a) image in chamber I, 0.25 m depth – visible irregular anomalies in the electromagnetic wave signal amplification (red), the effect of material cracking or non-homogeneity; (b) chamber II, 0.87 m depth – visible regular* anomalies in the electromagnetic wave signal amplification (red), the result of an under-plate drainage system [4]

position and arrangement, but the reinforcing bar diameters cannot be determined on that basis – invasive tests (drilling in the anomaly area) are indispensable. The origin of regular GPR anomalies (i.e. diffraction hyperbolas) identified below the bottom plate (with the 750 Hz antenna) still needs to be clarified (as not directly confirmed). Remembering the documented drainage system on and around the structure, the anomalies can likely be attributed to the drainage channels made of ceramic or concrete elements under the bottom plate. Unfortunately, the element sizes cannot be determined using the GPR method, but they can be assumed to be identical to the elements confirmed during uncovering in the 1960s (Fig. 9b).

6. Numerical analysis

Assessment of the structure's interaction with subsoil under variable load and consideration of the interaction with the adjacent palisade (retaining wall) required multi-stage numerical analyses. To that end, ZSoil 2018 software based on the finite element method (FEM) was used. The analysis covered all relevant phases and possible variants of the reservoir loading. Additionally, the potential influence of clay swelling under the reservoir and the phenomenon impact range were evaluated in the analysis – model in Fig. 10. Geometric parameters were determined based on in situ test results and the course of "triaxial" tests. A non-linear elastic Hardening Soil model with small strain stiffness (HSs) was used. The reservoir structure was modelled as concrete, assuming a simplified Hoek-Brown condition as the failure criterion. The material parameters were adopted assuming the compressive strength of concrete as 10 MPa and 1 MPa tensile strength. Reduced strength and strain parameter values were adopted for the components (e.g. columns) spaced at set distances, considering the impact of their spacing on the structure's model global behaviour. The palisade at the road was modelled with beam elements, where the assumed parameters corresponded to those of 0.6 m diameter concrete piles spaced every 1.06 m [4].

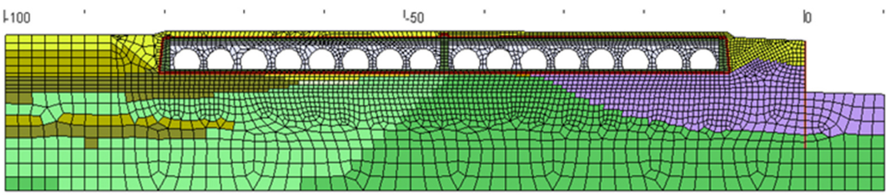


Fig. 10. Computational geotechnical model of the reservoir structure interaction with the subsoil

The water load inside the reservoir chambers is the main load in the analysed cross-section. It was assumed to be a hydrostatic load on the reservoir walls and bottom, with the value resulting from the stored water level. The reservoir bottom level (empty reservoir option) was considered the minimum one. The maximum level was when the reservoir was full during regular operation (full reservoir option). A case of only one chamber filled was also considered. In order to evaluate the impact of clay swelling in the subsoil directly below the founding level, the set swelling pressure values were assumed. The maximum expected displacement (soil uplift) at the total mobilisation of the set swelling pressure can be up to +16 mm, even when the reservoir is entirely filled with water – Fig. 11.

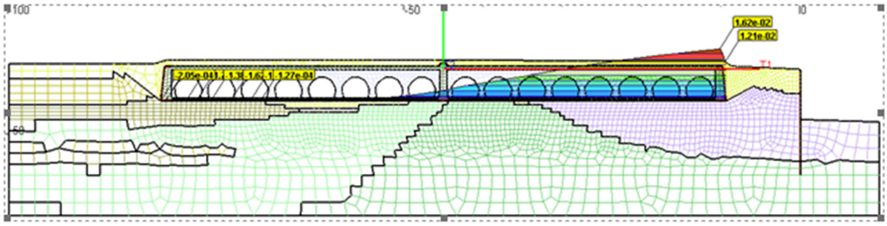


Fig. 11. Displacement distribution and values – visible impact of clay swelling (50 kPa swelling pressure) on the displacement of the reservoir and soil in its vicinity – soil uplift (max. 16 mm)

Based on the numerical analysis, it was concluded that the water level change in the reservoir related to the operating regime (the need to empty the chambers periodically for sanitary and maintenance purposes) has a limited impact on its replacement (max. ±5 mm). Nevertheless, even minor displacement occurring when the reservoir is occasionally emptied and filled during regular use, when the structure is sensitive, poorly reinforced and impaired because of its age, can lead to the formation of new micro failures or contribute to the progressing degradation of the existing ones, compromising the reservoir's integrity with time. Consequently, swelling processes could and can be activated due to the possible migration of water into the subsoil.

Calculations revealed that making a deep trench secured with a retaining structure embedded in the ground could result in displacements up to ca. 6 mm (for the reservoir wall adjacent to the palisade) at the construction stage (despite following the applicable technological practice). Evaluating the direct causes of the phenomenon at the current stage (missing construction period data concerning the displacement of the wall or further continuous monitoring) is a challenge. It is most likely a sum of factors related to the technology (supporting structure, non-anchored palisade), subsoil conditions (bored piles in expansive subsoil, presence of slopes), and environmental conditions (potential static impact of the reservoir and dynamic impact caused by road traffic). This, however, does not undermine the findings of numerical simulations on the interaction between the retaining wall structure (primarily at the excavation stage) and the reservoir (due to the potential impact of subsoil dampness). Subsoil uplift in expansive soils may contribute to extending the active zone (opening of microcracks), consequently leading to subsoil volume increase, dampness fluctuations and intensification of swelling and shrinkage in clays.

7. Summary

In the context of the tests and observations, the analyses indicate the direct determinants and causes of the currently discovered reservoir defects. This is a sum of factors resulting from the following:

– geomorphological situation and complex geological conditions – area at risk of mass movements, presence of expansive soils;
– age and quality of the construction material – expiry of the structure's life (over 100 years old), poor and irregular reinforcement, low concrete strength;

– current situation, development and use of the area near the reservoir – rainwater runoff, presence of trees.

The displacement computationally demonstrated and observed in and around the reservoir is minor and should not cause severe defects in structures fulfilling the applicable requirements (standards and regulations). However, for a historic facility with a sensitive and poorly reinforced structure situated on an "undercut" slope revealing mass movements, its founding partly on expansive soil can be identified as the root cause of the observed defects.

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Grunty ekspansywne jako praprzyczyna awarii zabytkowego podziemnego zbiornika wody pitnej

Słowa kluczowe: grunty ekspansywne, badania podłoża, metody geofizyczne, diagnostyka konstrukcji, obiekt zabytkowy

Streszczenie:

Artykuł opisuje przyczyny uszkodzeń zabytkowego podziemnego żelbetowego zbiornika wody pitnej, składającego się z dwóch komór, posadowionych na podłożu zróżnicowanym pod względem wieku i genezy (fałd zaburzonych glacitektonicznie iłów septariowych i glin polodowcowych). Udokumentowane uszkodzenia wynikają z deformacji podłoża wywołanego zmianami objętościowymi gruntu (skurcz, pęcznienie), w sytuacji różnych czynników zewnętrznych warunkujących intensywność tego zjawiska na przestrzeni ponad 100 lat użytkowania obiektu. Przeanalizowany przykład przedstawia problemy wynikające z posadowienia obiektu na gruntach ekspansywnych, wskazuje na zagrożenia wynikające z zaniedbań w kształtowaniu architektury krajobrazu oraz podkreśla rolę rozpoznania podłoża gruntowego. Poprzez kompleksowe badania geotechniczne i pomiary geofizyczne w obrębie konstrukcji i podłoża oraz inwentaryzację i ocenę charakteru uszkodzeń, wskazano potencjalne czynniki niekorzystnie oddziaływające na stan techniczny obiektu. Wnioski potwierdzają wykonane symulacje numeryczne obejmujące m.in. wpływ pęcznienia oraz obciążeń cyklicznych wynikających z funkcji obiektu i jego otoczenia.

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