The phenomena of soil liquefience in the bases of hydraulic structures

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Abstract. The article presents the results of an analysis of the potential for the phenomenon of soil liquefaction in the bases of hydrotechnical structures. The authors note that hydraulic structures are usually built in the valleys of watercourses where the soil structure is highly conducive to liquefaction processes. These are finely dispersed non-cohesive soils, usually fine-grained, medium-grained or silty sands, sandy loam. Massifs under pressure hydraulic structures are usually highly water-saturated. At the moment of liquefaction, the sandy soil, which was previously stable due to the friction of the particles, becomes a suspension, i.e. water with soil particles suspended in it, so structures located on such soils sink in this suspension. The article provides examples of accidents in the hydraulic structures of Russia caused by the liquefaction phenomena. The authors note that the main directions of protecting the structures of hydraulic structures from the consequences of liquefaction are to prevent the possibility of liquefaction and reduce the harmful effects of liquefaction. There are several methods and ways of reducing the possibility of liquefaction - the creation of an effective drainage system consisting of vertical drainage piles or vacuum wellpoints in the bases of structures where there are prerequisites for liquefaction. The paper proposes the construction of reinforced structures by resting on dense layers of soils below layers liable to liquefaction. It notes the need to consider the phenomena of soil liquefaction of bases in the normative literature on the design and operation of hydraulic structures.

1. Introduction

The phenomenon of soil liquefaction is well known in soil mechanics. Peculiarities of construction and operation of structures erected on soils liable to liquefaction processes are considered in the works of both domestic [1, 2, 3] and foreign authors [4, 5, 6]. However, we find that this process does not receive sufficient attention in hydraulic engineering construction and operation of hydraulic structures (HS).

Hydrotechnical structures - dams, hydroelectric plant buildings, ship culverts (locks, ship lifts) - are usually massive structures that have a significant impact on the soils where they are located. Much of the research into the processes involved in the bases of hydraulic structures focuses on the consolidation of soils [7, 8]. As is known, soil consolidation is the delayed compaction (increase in density) of a water-saturated soil layer over time, occurring due to the squeezing out of water or the convergence of soil particles under load. The duration of the consolidation processes of the soils at the base of the HS depends on the structure of these soils and the consolidation processes can take a
considerable amount of time. For example, the consolidation of soils at the base of the Gorodets locks (Volga Basin Administration) lasted about 50 years, at the base of the Volga locks (Volgo-Don) the consolidation period took about 30 years [7]. As noted in [9], in the presence of clayey soils in the foundations, settlement of structures can develop throughout the life of the structure.

Liquefaction of soils is a qualitatively different process, the transformation of soils into a fluid state as a result of a significant change (up to and including destruction) of the soil structure, irrespective of the cause of this transformation. Such processes develop in non-cohesive soils whose pores are filled with water. The dynamic action on these soils destroys the structural bonds between the particles and the saturated soil acquires the properties of a heavy viscous fluid. As a result, earthen structures are spread and heavy structures located on such soils sink in the liquefied soil. World practice knows many examples of such accidents and disasters.

In 1912 in Revel (now Tallinn) the head of a breakwater made of ribs suddenly shifted by 2 metres in totally calm weather and collapsed, sinking deep into the ground. A similar case occurred with the rowing embankment in the port of Kandalaksha: rowing was installed and soil was backfilled between the rowing and the shore to form the port area. Suddenly the ground turned into a liquid body that spilled over a distance of several dozen metres and within two minutes the embankment was destroyed; the ground took the rows of trees with it and they sank in it. There are also known ground liquefaction-induced failures during the Niigata (Japan) earthquake in 1964 [10], the Bora Peak in (USA) in 1983 [11], the Bhai and Ahmedabad in 2001 [12] and the Christchurch (New Zealand) in February 2011 [13].

An analysis of the location of navigational hydraulic structures in operation in Russia revealed that a large proportion of them are located on weak non-cohesive soils which are subject to liquefaction phenomena under the right conditions. However, the current regulatory literature in the Russian Federation defining the rules for the construction and operation of hydraulic structures does not reflect the liquefaction processes.

2. Materials and Methods

Studies of liquefaction processes in soils mainly focus on the occurrence of such phenomena under significant dynamic impact with high frequencies - earthquakes, vibrations and other forceful influences [14, 15]. It is known that the ability of soil to liquefy, float and lose its strength completely under the influence of shaking, vibration or other external influences is called thixotropy. But soils can liquefy not only through vibration. The above mentioned accidents in Tallinn and Kandalaksha confirm this.

Liquefaction occurs as a result of the breakdown of structural bonds between particles in water-saturated dispersed soils under different types of stresses. This can cause the soil to lose all or part of its load-bearing capacity and become fluid as a result of structural failure and movement of the particles in relation to each other.

The liquefaction process generally consists of three stages: the destruction of the original soil structure; the transition of the soil into a liquefied state; restoration of the structure and gradual consolidation of the soil. Water-saturated fine sands, dusty sands and sandy loam are the most commonly liquefied. The greater the porosity of the ground, the less dynamic stresses will cause liquefaction. A prerequisite for liquefaction is complete or near complete saturation of the ground with water [16]. When the ground is dynamically stressed, the pore pressure in the soil increases, which leads to shear deformation and volumetric deformation. When there is a large amount of water in the ground, it does not have time to leave the pores it was in, so pore back pressure occurs, which reduces the shear resistance. Pressure dissipation at the ground surface and settling of particles leads to abrupt settling of structures.

The strength and stability of ground structures depend on the following parameters:
- the angle of internal friction, which characterises the frictional force between the ground particles;
- cohesion, which characterises the resistance of soil particles to all movement and depends on the structural bonds of the clay particles.
The angle of internal friction and cohesion together determine the shear resistance of soils. At the moment of liquefaction, the sandy soil, which was previously stable due to the friction forces of the particles, becomes a suspension, i.e. water with soil particles suspended in it, so structures located on such soils sink in this suspension.

Research shows [17] that the natural angle of repose at 13 - 14 % moisture content is the same as in the respective sands and sandy loam, and that the natural angle of repose decreases sharply with increasing moisture content, reaching almost zero at 17 - 20 % moisture content. In addition, the Archimedean weighing force acts on the ground particles, reducing the pressure between the particles and therefore reducing the friction force. Soaking of the clay and dust particles that bind the coarse grains also reduces the bonding force between them.

The coagulation structure formation processes occur simultaneously [18]. These phenomena are typical of "transitional" (from pure sands to clays) soil varieties such as dusty sands, sandy loams and some light loam varieties and are associated with the destruction under dynamic conditions of both coagulation and mechanical contacts. Moreover, the specific composition of these soils contributes to the mutual breakdown of both types of contact. Therefore, such soils in the liquefied state have the lowest viscosity (lower than that of liquefied clays or sands) and the highest mobility of all dispersed soils. The clay particles in the soil are the reason for their ability to remain in a liquefied state for a long time.

After removing the dynamic load, there is a process of gravity compaction of the soil. In this case, the soil particles tend to settle more densely, and this results in system consolidation.

One of the conditions for liquefaction is the presence of layers of granular particles in softer materials in the soil column, which is common in many river sediments. Hydraulic structures generally tend to be located in river valleys and are therefore characterised by this type of soil structure - alternating layers of sand and loam. The construction of retaining structures - dams, dikes, locks - creates level gradients in the watercourse (river, canal) channel, supported by the pressure front of the waterworks, which ensures water saturation of the foundation and creation of filtration flows with sufficiently high head gradients in them. The impeded outflow of groundwater from beneath structures in the foundation soils creates conditions for liquefaction.

In some cases, there is incomplete liquefaction of the ground, called partial liquefaction in work [1], where the overpressure in the ground does not reach the maximum limit value. Some compressive stresses in the soil skeleton are retained and the ground has some load-bearing capacity.

The occurrence of basement liquefaction phenomena can provide a new perspective on the causes of landmark accidents and negative trends in Russian hydraulic structures that have occurred in recent years.

3. Results

Ship lock No. 5 of the Volga-Don Shipping Canal

Sedimentation of the chamber and head sections of the Volga-Don Shipping Canal (VDSC) lock No. 5 that started from the beginning of operation (1952) continued until 2016, reaching over 100 mm, with the sedimentation of sections IV and V beginning to increase dramatically from 2006, the rate of sedimentation increased to 10 mm/year, the intensity of sedimentation of other sections to 3.60 mm/year. The total precipitation of Sections IV and V from 2006 to 2016 reached 125 mm. The increase in the settlement was accompanied by an increase in the horizontal deformation of these sections, which differed markedly from that of the rest of the lock chamber. External signs of deformation of the lock chamber foundation manifested in the form of settling cracks forming along the chamber walls in the backfill. In April 2005, a griffin formed in the left-hand drainage basin with quite an intense outflow of water and the formation of a soaking zone.

Based on the adopted safety criteria, the condition of the lock chamber was deemed to be pre-emergency.

A set of research works in 2008, 2019 - 2020 revealed significant decompaction of the lock chamber foundation soils, especially in the area of sections IV and V. Most of the chamber floor is cut
into the light water-saturated sandy loam of the Lower Bakinsky horizon, which is similar in composition to dusty fine-grained sands. Fine sands with a particle size of up to 0.1 mm make up the bulk of the soil almost all the way down to the bottom of the chamber (up to the clay buttress - 12 m). The percentage of these sands in the samples is between 85-95%.

The water level in the ground of the lock is set at or above the downstream level, the bottom of the chamber is significantly below this level, the ground below the chamber is constantly saturated with water with significant head gradients (the head at the hydroelectric station is 9.5 m). The porosity coefficients for all soil types (coarse, medium, fine and dusty sands) are determined by surveys as $e > 0.8$ p.u., which corresponds to a friable condition.

Therefore, the base of the lock chamber creates conditions for liquefaction of the soils, which causes significant continuous settlement of the structure.

**Ship lock No. 2 on the Volga-Baltic Waterway**

The current final technical status of the Volga-Baltic Waterway’s shipping lock No. 2 is assessed as limited operational capability with a reduced level of safety. A major factor reducing the technical condition and safety level of the structure is the occurrence and development of a crack at the top edge of the left abutment and in the left lower head emptying gallery.

The problems that led to the crack in the concrete structure of the left abutment of the lower head were already evident in the construction phase of the lock. The excavation of the lower head excavation took place in waterlogged interbedded soft soil variations in the presence of pressurised and unconfined aquifers. Excavation work under these conditions required special dewatering, drainage and the selection of appropriate excavation patterns. It was not possible to reduce the underbalanced surface below the bottom mark by deep-draining wells. The over-moistened soil lost its adhesion under slight dynamic influences and turns into a floating mass, and leads to the suspension of the basement preparation works. The executive survey of the excavation of the lower head on 01.06.1956 indicated the presence of liquefied base soil below the level of the excavation bottom up to 2.0 m. An area of liquefied soil has spread below the base of the lower head with a transition to the adjacent section of cell no. 13.

To lower the level of the depression surface, it was decided to delineate the concrete preparation of the lower head with wellpoints plants; only after these measures the groundwater level in the middle part of the excavation was lowered. In addition, we had to install a collector with needles every 0.6 m in the middle of the excavation of the lower head, only then the level in the middle part dropped 1 m below the bottom of the excavation, which made it possible to start the concrete preparation work.

The base of the lock head was a clay packet of rocks underlain by sand. During construction, the loamy soils uncovered at the base elevations of the bottom head of the lock were completely replaced by sandy soils due to their low thickness and possible change of properties in the open trench. The creation of the pressure front of the hydroelectric installation, the Belousovsky reservoir and the rise of the groundwater level in the lock area resulted in the formation of seepage flow in the base soils from the upstream and from the upstream to the Vytega river bed.

Filtration of fine clay particles from the base of the lower head, as well as suffosion of fine fractions from the sandy soils began during the operation of the hydroelectric installation, which, judging by the comparative analysis of the survey data up to 2020, continues. In addition to the seepage flow, the operation of the lock (filling and emptying of the lock chamber, operation of the working gate mechanisms) also had a dynamic effect on the characteristics of the soils. This created a decompacted soil layer in the basement rock roof of the lower head.

As noted above, liquefaction of soils is caused by the following factors: the fine fractional composition of soils, their high porosity and complete or close to complete saturation of the soil with water.

Research showed that the content of particles smaller than 0.25 mm for the different layers of the base soil of the lower head of lock no. 2 ranged from 93.3 % to 97.3 %. The porosity of the soils is
between 34.77 % and 41.04 %. The soils are fully water saturated as the downstream level is above the base of the downstream head.

The process of weighting fine-grained soil particles occurs when the soil mass is saturated with water. The "floating" soil particles reduce their gravitational properties by decreasing the force of friction between them. The angle of internal friction at a natural humidity of 12.7 % is 31.5°; at a humidity of 37.47 %, when all the pores in the ground are filled with water, the angle of internal friction loses its value. There is almost no adhesion between the "floating" particles, the water-saturated fine-grained soil turns into a "heavy" liquid.

This led to uneven settling of the left and right abutments of the lower head, with the abutment settlement vectors pointing in different directions: the left abutment to the south-east; the right abutment to the north-west. This inconsistent operation of the left abutment and the bottom of the downstream end of lock No.2 resulted in a crack in the concrete. As the processes in the base soils have not stopped, the crack in the concrete also continues to develop.

**Moscow Canal flood-breaking dam between navigation locks No. 7 and No. 8**

On 10 January 2019, the slope of the Moscow Canal embankment between shipping locks Nos. 7 and 8 swept away and liquefied soil started flowing into the Volokolamsk Highway. The official cause of the accident was the inflow of water from the canal under the loam screen along the break between the sheet pile and the tunnel, with further loam decompaction on the western dyke and the creation of a failure on the outer slope. The reason is the poor quality of the construction and installation work.

In our opinion, the main cause of the accident was the liquefaction of the sandy soil in the tunnel cavity structures. The dyke enclosing the canal consists of an impervious loam screen and a retaining sand prism, and there is no drainage for filtered water. Water entering the sand prism from the channel through the contact joint between the tunnel cover and the screen filled its entire volume. The absence of contour drainage between the tunnel cover slab and the sand embankment contributed to the complete water saturation of the prism, i.e. there was no good drainage (diversion of water) from the dam. During the winter, the slope of the dyke with the plant soil and grass froze, stopping the outflow of water from the prism and its evaporation. The ground "floated" when the traction forces exceeded the thresholds.

**Beloporozhskaya MHPP-2 unpaved dam**

On March 22, 2020, there was a 7.1 to 20.5 m wide break in the crest at the base of the Beloporozhskaya MHPP-2 channel earth dam (Kem River, Republic of Karelia). Part of the downstream bank is destroyed and the ditch is filled with water. According to the experts' conclusions, the breach was caused by an incorrect assessment of seepage strength during the geotechnical survey, improper selection of materials and thickness of backflow preventers to the drainage prism. However, after considering the experts' recommendations, when the reservoir was re-filled on 24 October 2020, a breach occurred again in the left bank section of the earthen dam.

Soil liquefaction processes are also responsible for the breach of the Beloporozhskaya HPP-2 groundwater dam. The stability of the dam was calculated from the physical and mechanical properties of the soil according to the current design standards. The seepage coefficient adopted in the design ensures the seepage strength and stability of the structure when the dam body is backfilled with moraine soils. The weir body does not have any impervious elements and the weir body is drained by means of volumetric stone prisms located in the upstream and downstream reaches. However, the filtration coefficient of moraine soils including fine sand and dust particles has a value of 0.005 - 0.0003 m/day. At these values, drainage is ineffective, resulting in water saturation of the earthen soil. In addition, the drainage capacity is reduced if the drainage prisms are not properly executed (clogging with other soils). The water-saturated soil of a dam acquires the properties of a "heavy" liquid.
4. Discussion
To prevent the occurrence of zones of liquefaction of unbound soils dangerous to the stability, strength and integrity of hydraulic structures, it is possible to propose several measures and structures. The main areas of structural protection against liquefaction and its effects can be divided into two types:
- preventing the possibility of liquefaction;
- reducing the harmful effects of liquefaction.

**Preventing the possibility of liquefaction**
The hydraulic structures include large areas of unbound fine-grained soils at the base. It is therefore effective to prevent liquefaction by compacting, consolidating soils at the base of structures.

1. One method known in practice for compacting fine-grained soils is drainage by diverting water from the ground. In the Elbe floodplain in Germany, for example, Bauer prepared the base for an airfield runway on muddy soils using vertical drains. The paper pipes were buried for 10 - 12 m by paper tape indentation at 1.5 m intervals (Fig. 1, a).

![Figure 1. Using paper pipes for drainage:](image)

*a - scheme of the arrangement of drenches; b - scheme of seepage and compaction of the ground mass*

When in the wet ground, the paper tape, saturated with moisture, straightened out and became circular in cross-section. A filtration gradient was formed on the surrounding saturated ground and the ground gave up water to the paper tube. Around the pipe, there was a zone (column) of dewatered compacted soil (Fig. 1, b). This principle of capillary movement of groundwater towards the drainage pipe is widely used in dewatering practice.

2. It is known to increase the gradient for water extraction by means of vacuum wellpoints [19]. The vacuum wellpoint sinks to a depth of 25 m and is vacuumed in soils with a low filtration coefficient ($K_f$ less than 0.5 m/day). Such soils include fine-grained dusty and clayey soils. The vacuum dewatering unit consists of a 4K-8 centrifugal pump and a GV-5c water jet pump. At a sinking depth of wellpoints of 8.5 m, the capacity of the plant is 40 m$^3$/hour. Similar vacuum dewatering systems were used in the construction of the Volga-Kama cascade of hydroschemes.

3. Since the thickness of the saturated soils is usually of the order of 8 - 15 m (e.g. at the Volgo-Balt No. 2 and Volgo-Don No. 5 locks), it is also advisable to install vertical drainage. Drainage includes a drainage column in the form of a perforated pipe, with the interior filled by a material that absorbs groundwater. Two options are possible: a single-use or multiuse column.

The single use involves the construction of a hollow column in the ground (Fig. 2), which creates a pressure differential between the water in the surrounding ground and the cavity inside the column.
Figure 2. Diagram of the single-use operation of a depth drain:

\[ a \] - plan; \[ b \] - vertical section

1 - perforated pipe; 2 - water outflow boundaries into the pipe (drainage influence zone) 3 - porous filler;

This difference causes the filtered water to move from the surrounding unbound soil mass into the column cavity. Once the column is filled with water, the inflow of water from the soil stops, but a zone of compacted soil with low water content is formed around the culvert - a zone of accelerated consolidation. It is possible to fill the inside of the pipe with absorbent material: mineral wool, wood fibre wool, etc. The purpose of this filling is to prevent the inner cavity of the drainage pipe from being filled with fine-grained sediment.

Instead of completely filling the inner cavity with moisture-absorbing material, it is possible to use internally lined pipes, with the porous filler material arranged as a layer on the inner surface of the perforated pipe (Figure 3).

Reusable vertical drainage involves drawing filtered water from the surrounding soil mass into the drainage pipe and pumping it out periodically. The most reliable design in this case is the twin-cylinder tube, consisting of two co-axial perforated cylinders (Figure 4). It is necessary to pump the water out of the inner cylinder as it fills up. The most convenient way to do this is to use vacuum devices.

Figure 3. Drainage pipe:
1 - perforated drainage shell; 2 - porous material lining; 3 - collecting cavity for filtered water

Figure 4. Cross section of a drainage pipe with water pumped out:
1 - outer perforated cylinder; 2 - porous filler (filter); 3 - inner perforated cylinder; 4 - inner cylinder mounting (fixation) ribs

Such drainage devices should preferably cover the entire area under the structure, built up of liquefiable soils, so that the zones of influence of the drains overlap. But often the installation of such a drainage grid beneath the foundation is technically problematic, accessing it, much less drilling holes for vertical drainage is very difficult, and sometimes simply impossible. However, usually the pressure of massive hydraulic structures creates a very significant load on the base so the most dangerous areas with regard to liquefaction potential are the unloaded peripheral areas of the structure, within which it makes sense to organise dewatering and consolidation of the subsoil.
In earthen dams, to prevent liquefaction, it is advisable to install drainage dikes or seals on the underwater part of the slopes and where the underbalance curve is close to the downstream slope surface. It is possible to use geotextiles, geogrids or rock fill as filter bedding.

**Reducing the harmful effects of liquefaction**

As noted above, a condition for liquefaction is the presence of a layer (mass) of fine-grained unbound water saturated soils directly beneath the base of the hydraulic structure. However, the characteristics of the substrate often make it possible to propose ways of partially or fully supporting the concrete mass of the structure on layers of soil with a higher load-bearing capacity below the fine-grained differences. As a rule, the depths from 10 to 20 m from the base of the structures have more dense layers - clay, dense sands, etc.

It is possible to install a pile-deck to transfer the load from the concrete structures of the structure to the stronger subsoil. A scheme of such a pile cap that can be proposed to strengthen the lower downstream end of the Volga-Baltic Waterway lock No. 2 is shown in Figure 5. It includes 6 piles that are driven along the periphery of the structure. The pile structure must rely on dense soils and transfer the load from the concrete mass of the structure to them. The transfer of loads requires a special mating unit (Fig. 6).

![Figure 5. Schematic diagram of the piling arrangement at the lower head of navigable lock No. 2 of the VBWW: 1 - lower downstream end of a lock; 2 - bored piles; 3 – breakstone piles](image)

![Figure 6. Scheme of the interface node: 1 - pile tube; 2 - pile reinforcement frame; 3 - anchors; 4 - concrete head structure](image)

There are two pile variants suitable for waterborne erection conditions:
- bored piles;
- breakstone piles.

The design of these piles allows them to be built both from dry land and from the water. Drilling piles after laying the concrete mixture assume the extraction of the pipe. Both concrete and breakstone piles allow an increase in bearing capacity due to the friction between the pile and the surrounding sand mass. Using breakstone piles can increase the number of piles from six to ten by installing them in the junction area of the abutment with the flood bed.

To transfer the load from the foundations to the heads of the breakstone piles, it is necessary to install a single concrete beam (foundation) at the heads of all piles, adjacent to the concrete bottom of the structure (Fig. 7). The connection can be made using a steel beam fixed with anchors to the face of the base of the structure.
Figure 7. Scheme for supporting the structure on breakstone piles:
1 – breakstone pile; 2 - reinforced concrete beam; 3 - anchor in beam; 4 - anchor in concrete base;
5 - steel beam; 6 - temporary excavation for installation work

We recommend the following characteristics for breakstone piles:
- pile depth is up to 12 m;
- pile diameter is 0.8 m;
- crushed stone in fractions is 20 - 30 mm;
- anchor rods in flood bed $d = 70$ mm, length 0.8 m;
- anchor spacing is 1.0 m.

5. Conclusions
Thus, we should note that the phenomenon of soil liquefaction in bases is very often the cause of non-design displacement and deformation of hydraulic structures. The presence of unbound fine-grained soils at the base of structures, usually located in the valleys of watercourses, contributes to the liquefaction of soils. After the construction of the retaining structures and the creation of the pressure front of the waterworks, the base soils are saturated with seepage water with significant head gradients. In the absence or difficulty of drainage from the soil masses, liquefaction conditions form in the soil masses.

It is important to take these phenomena into account both at the design stage and during the operation of hydraulic structures. The analysis has demonstrated that failure to consider the risk of liquefaction leads to the disruption of structures and even to accidents.

In addition, considering the importance of hydraulic structures and the danger of severe consequences of accidents on them, it is necessary to include the possibility of the phenomenon of liquefaction of soils in the regulatory literature.

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