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# Stress-strain state and performance of a high rockfill dam with a grout curtain

# Напряжённо-деформированное состояние и работоспособность высокой грунтовой плотины с инъекционной завесой

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**Ключевые слова:** цементация; инъекционная завеса; каменно-набросная плотина; зацементированный грунт; модуль деформации; напряжённо-деформированное состояние

**Abstract.** This paper examines the results of numerical simulation of the stress-strain state (SSS) in a rockfill dam 235 m tall, with a massive three-tier grout curtain used as the impervious element. Calculations were done for multiple scenarios that reflected the wide range of possible changes of the modulus of deformation for the filler rock and cemented soil (200 MPa to 5000 MPa). The author's original computation program was used in order to consider the elastoplastic behavior of soil. It was discovered that a super-tall dam with a vertical rigid impervious element features an adverse SSS: the hydrostatic pressure in the head reach gives creates a sliding wedge in the upper retaining prism. As the dam has rigid structure, this causes marginal state zones in the filler rock. Only when deforming ability of the filler rock is very low (deformation modulus above 300 MPa), the dam's SSS may be acceptable. Strength characteristics of the grout curtain were evaluated based on stresses that emerge after it is created. The SSS of the grout curtain is mainly described by bending strains. In case of low deforming ability of the screen material, this can give rise to fissures in the screen material where it contacts the bedrock base. Strength and water impermeability of the grout curtain are secured only after bentonite-cement slurry is injected to make the deforming ability of the screen material and filler rock comparable.

**Аннотация.** В статье рассматриваются результаты численного моделирования напряжённо-деформированного состояния (НДС) каменно-набросной плотины высотой 235 м, в которой противофильтрационным элементом является массивная инъекционная завеса, выполненная в три яруса. Расчёты проводились для нескольких вариантов, отражавших широкий диапазон возможного изменения модулей деформации каменной наброски и зацементированного грунта (от 200 МПа до 5000 МПа). Использовалась разработанная автором вычислительная программа, что позволило учесть упругопластический характер поведения грунтов. Было получено, что сверхвысокая плотина с вертикальным жёстким противофильтрационным элементом имеет неблагоприятное НДС – гидростатическое давление верхнего бьефа вызывает в верховой упорной призме образование призмы обрушения. Наличие в плотине жёсткой конструкции приводит к образованию в каменной наброске зон предельного состояния. Только при очень низкой деформируемости каменной наброски (модуль деформации свыше 300 МПа), НДС плотины может оказаться приемлемым. Оценка прочностного состояния инъекционной завесы проводилась по тем напряжениям, которые возникают с момента времени её создания. НДС инъекционной завесы в основном определяется деформациями изгиба. При низкой деформируемости материала завесы они могут привести к образованию трещин в материале завесы и на её контакте со скальным основанием. Только при применении для инъекции бентонито-цементных растворов, при которых деформируемость материала завесы и каменной

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наброски становятся сопоставимыми друг с другом, прочность и водонепроницаемость инъекционной завесы будут обеспечены.

## *Introduction*

Grout curtains are a widely used type of an impervious device at the dam base. They are built by injecting cement and cement-clayey slurries into the pores and fissures of soil. Most frequently, grout curtains are constructed on bedrock base – virtually each tall dam has a cement-grout screen built at its foot [1-6]. Grout curtains are used less often in earthen dams, because soils tend to have hollows, and much greater quantities of slurry have to be injected of fill in the pores. Nevertheless, building a grout curtain in an earthen base frequently is the only possible dam construction option: when the base is composed of large-grain soils at a considerable depth. Numerous dams with grout curtains built into an earthen base exist around the world. A large grout curtain in an earthen base was first used in France in 1954-56 to build the Serre-Ponçon dam [7].

The grout curtain as impervious element has an advantage that it can be built not only at the base but also inside the body of the earthen dam. Quite frequently, the grout curtain built at the foot continues into the lower section of the dam. Construction Regulation SP 39.13330.2012[1] places such dams into the class of combination impervious element dams. As an illustration of such construction design, one may refer to the dams of Atbashi [8], Maina [9], Aswan [10], and the dam of Kambarata HPP-2 [11]. In one case – the Orto-Tokoy dam in Kyrgyz Republic – a grout curtain was built to the full height of the dam [12].

In the light of those experiences, one can regard the grout curtain as an advanced type of impervious design of an earthen dam. This is particularly relevant for construction of super-tall earthen dams, where selection of reliable design of non-soil impervious element still remains an unsolved issue. This paper examines possible construction of a super-tall earthen dam with an injection-built core. Review of science literature demonstrates that performance of a dam with an injected impervious screen had never been researched before.

Our study considered a dam 235 m tall, standing on a bedrock base (Fig.1). The design of the dam included retaining prisms of rock mass on the sides, while the central part of the dam is filled with sand-gravel mix, then cement and bentonite-cement slurry is injected into the center to form a grout curtain. It was assumed that the top of the bedrock base is cemented to strengthen it. Such a dam could be used to build the Kankun HPP in South Yakutia.

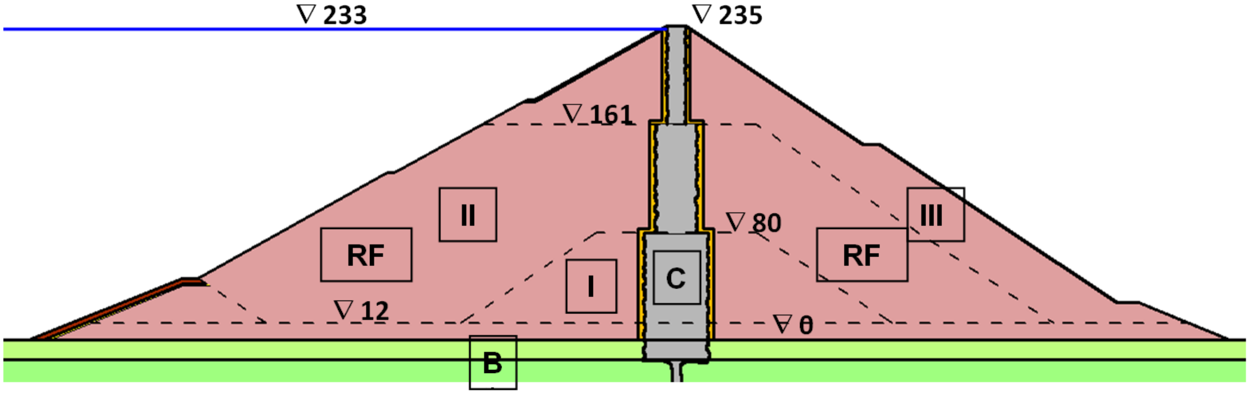

**Figure 1. Design of rockfill dam with an anti-seepage element as grout curtain.**
**C – grout curtain, RF – rock fill, B – bedrock base, I, II, III – construction project phases**

Considering the elevation of the dam, the study examined a 3-phase construction project. Each phase had elevations of 80, 161 and 235 m, respectively. It was assumed that the grout curtain was also to be built in three phases (tiers) – injection is done to full height of the construction phase, while soil is being filled in the next phase of the dam at the same time.

Thickness of the grout curtain was assumed different for each construction phase of the dam depending on the head it has to withstand, based on the condition of required filtration strength. This

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[1] SP 39.13330.2012.Plotiny iz gruntovyh materialov.Aktualizirovannaj aredakcija SNiP 2.06.05-84* [Dams of soil materials.The updated edition of SNiP 2.06.05-84*].Ministerstvo regional'nogo razvitija.Moskva, 2012. – 86 p.

thickness was taken to be some 20% of the head, because Construction Regulation SP 23.13330.2011[2] regulates that to design grout curtains, the critical gradient of the head should be assumed 7.5 for gravel and shingle, and 6 for large-grain sands. Screen thickness was assumed to be 14 m for the dam at phase one, 31 m for phase two, and 48 m for phase three. Therefore, the grout curtain in question is a massive structure best described as an injection core rather than as a grout curtain – the term adopted in the civil construction regulations.

Research was to address the following assigned tasks:

- identifying the conditions of the grout curtain built inside the earthen dam, to know the zones where its strength can be compromised,
- identifying conditions of the embankment body with a massive rigid structure inside it,
- estimating performance of the design of a tall rockfill dam with a grout curtain, and preparation of recommendations on how to select the design of the earthen dam with an impervious grout curtain to ensure its performance.

## *Methods*

Research was conducted by numerical simulation of stress-strain state (SSS) of the dam using the finite-element method, assisted with Nds_N software application developed by the author [13]. The structure and mass of the bedrock base were divided into a grid of 1,065 finite elements, whereof 121 finite elements were contact elements that modeled possible manifestation of non-linear effects on contacts with different materials (such as detachment, slip, etc.). Contact elements were introduced where the grout curtain contacted the dam body soil, bedrock base, and different phases/tiers of the grout curtain.

All finite elements had quadratic settlements approximation, thus producing more regular distribution of stress and deformation in them. The digital model of the structure featured 5,990 degrees of freedom.

The process of building the grout curtain in the dam was simulated by substituting the properties of the material in its location: the properties of normal soil were substituted with those of injection-strengthened soil. Parameters that described strength characteristics of the area were zeroed out as though this was a newly emerged structure, but the original SSS of the area remained without change.

To address the tasks, we researched how the deforming properties of the filler rock and the grout curtain influenced the dam’s SSS.

To simulate performance of the filler rock in the dam, we used two elastoplastic models of soils. One soil model was proposed by Prof. L.N. Rasskazov [14] and modified by M.P. Sainov [15]. It can register plastic, rheological and dilatant properties of soils. The other model was based on the Mohr–Coulomb strength theory, which assumed the soil deformation modulus in the active load area to remain the same regardless the strength condition of the soil.

Both models considered differences in soil deforming ability when actively loads were received and lifted. It was assumed that soil deformation modules off-load were 5 the level registered under active load. In order to delimit the load trajectories, the research used simple conditions based on analysis of changes in the soil condition being loaded. It was assumed that active loading is a fact, if:

- average stress $\sigma$ is rising,
- greatest shift stresses T are growing,
- as $\sigma$ and T are decreasing simultaneously, stresses T decrease slower than they grew at the previous loading stages.

This approach generally corresponds to that of the plasticity theory, which calls for location analysis of so-called loading surfaces [12].

We examined three scenarios of deformed properties of the rock fill. Deforming ability parameters of the rock fill in scenario A were found by this author [15] based on processed results of experimental research by Marsal [16] and Gupta [17], and then adjusted to consider field survey data on construction settlement of rockfill dams [18,19]. Converted to the linear-deformation environment model, scenario A

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[2] SP 23.13330.2011.Osnovanija gidrotehnicheskih sooruzhenij.Aktualizirovannaja redakcija SNiP 2.02.05-84 [Bases of hydraulic structures. SNiP 2.02.05-84Updated edition]. Ministerstvo regional'nogo razvitija.Moskva, 2011.– 111p.

for the dam in question corresponds to the deformation module of E = 85 MPa (for Poisson’s ratio $\nu = 0.25$).

In scenario B, the deforming ability of rock fill was halved (E = 170 MPa), and divided by 4 in scenario C (E = 340 MPa). This interval of rock fill deforming ability corresponds to the dispersion of values observed in real-life dams [20].

The deforming ability of soil in the central zone was assumed permanent in all scenarios: its deformation modulus was approximately 50 MPa.

Material of the grout curtain for computations was assumed to be elastic, with deformation modulus E and Poisson’s ratio $\nu$ assigned. Computation examined multiple scenarios of deformation behavior of injected sand-gravel, covering a deformation modulus in a very broad range. The reason is that by now, deformation behavior of soils strengthened by injecting cement and cement-clayey slurries is unknown: their properties can only be inferred by the properties of similar materials.

Firstly, there are data from experimental research of the properties of soil-cements – soil mixed with cement or cement slurry [21–25]. In [26], these writers processed the data from traxial tests of sand-cement done by Dr. Amanullah Marri [27]. It was found that if 5÷10% cement is added to sand, its deforming ability is only reduced by 2÷3 times. The modulus of cemented sand was 250 to 1300 MPa depending on content of cement (5÷10 %) and swaging pressure (0 to 10 MPa).

Secondly, data are available about deforming ability of the material known as CSG (cement-sand-gravel), super-lean rolled compacted concrete [28]. Its estimated deformation modulus is 5000-10000 MPa.

However, information on the deforming ability of soil-cements cannot fully qualify as injection-strengthened soils due to differences in the technology used to produce cemented material. To make soil-cements and rolled compacted concretes, cement is mixed with the aggregate before concrete is poured into the dam, and then must also be thoroughly condensed.

Conditions of grouting are rather similar to those used to prepare super-lean rolled compacted concrete – slurry-cemented rock. This is another type of cement-soil mix. The material was researched by A.S. Bestuzheva [29, 30], who discovered its deformation modulus to be around 2,000 MPa.

Thirdly, one can use information about the properties of soils secured by the jet grouting method. Still one must bear in mind that the jet grouting technology of soil strengthening differs from the conventional method. In jet grouting, the jet of slurry or water served at high pressure (35÷45 MPa for cement slurry, and 5÷8 MPa for water) erodes the foundation, then the resulting hollows are filled with the slurry [31]. Information about experimental research of sand samples strengthened in field conditions by the jet grouting method is available in [32]. According to this, compressive resistance of strengthened soil varies from 5 to 22 MPa, with deformation modulus from 1000 to 4500 MPa. However, such excellent performance requires a lot of cement, with needed 500 to 1,100 kg of cement per 1 $m^3$ of strengthened soil. Less cement is necessary to create grout curtains in large-debris soils. To quote [31], the strength of sand-gravel soil after jet grouting was approximately 5 MPa with cement consumption rate of 150 kg per 1 $m^3$. No data is available on deformation modulus of such soil.

The above research data on the properties of cemented soil are mere approximations, because injection-strengthened soils differ. The fundamental difference of the latter is that the composition of injected slurries typically contains bentonite or other similar clayey material in addition to pure cement. Cement-bentonite or bentonite-cement slurries are used for injection. Presence of bentonite reduces the deforming ability and strength of processed soils.

In Table 1, we quote the composition of the slurries that were used to build the grout curtains for the dams of Atbashinskaya and Mainaskaya HPP [8, 9]. Analysis demonstrated that bentonite-cement slurry could contain between 15 and 60 % of bentonite.

***Table 1. Information on materials consumption to build grout curtains***

| **Dams** | **Injection slurry structure** | | | **Slurry density, ton/m$^3$** | **Consumption per 1 m$^3$ of soil** | | |
|---|---|---|---|---|---|---|---|
| | Cement, kg | Bentonite, kg | Water, kg | | slurry, m$^3$ | solid matter, kg | cement, kg |
| Atbashinskaya | 350–475 | 82–59 | 826–852 | 1.29–1.36 | 0.26 | 181 | 110–160 |
| Mainskaya | 100 | 140–110 | 920–930 | 1.16–1.14 | 0.42–0.60 | 200–240 | 85–110 |
| | 400 | 120–100 | 826–830 | 1.33 | | | 150–190 |
| | 600–500 | 160–120 | 750–800 | 1.5–1.42 | | | 140–220 |

Calculations demonstrated that to build 1 m$^3$ of strengthened soil, one needs 80 to 220 kg of cement, and 25 to 140 kg of bentonite.

Apparently, injection-strengthened soil is closer to clay-cement concrete rather than soil-cement or rolled compacted concrete, but it is weaker and more prone to deformations due to insufficient stirring. As properties of clay-cement concretes can vary over a broad range of values depending on cement content ratio [33], we examined a broad range of soil properties in a grout curtain. The three scenarios for properties of grout curtain material were:

- scenario 1 – E=5000 MPa, $\nu$=0.22,
- scenario 2 – E=1000 MPa, $\nu$=0.25,
- scenario 3 – E=200 MPa, $\nu$=0.30.

It can be said that in scenario 1 soil pores are filled with cement slurry (by jet grouting), while scenario 2 uses injection of cement-bentonite slurry, and scenario 3 uses bentonite-cement slurry. According to [32] and [33], strength of cement (cement-clayey) rock resisting single-action compression was assumed to be around 20 MPa, 5 MPa and 2 MPa respectively for scenarios 1, 2 and 3.

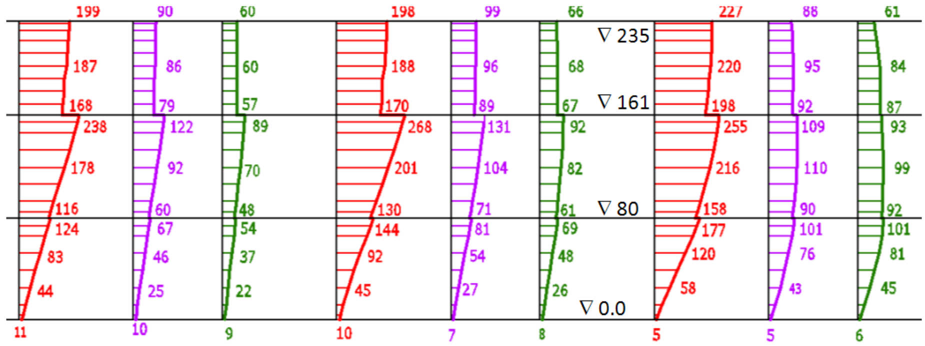
We have examined 9 computing scenarios, each with different deformation properties of filler rock and the grout curtain material. The numeration for the scenarios was dual (e.g., scenario 1A, 2C, etc.).

Calculations addressed own-weight loads of soil and hydrostatic pressure on impervious elements.

It was found that SSS for a tall earthen dam with a massive and rigid impervious element was rather complex and adverse, featuring marginal state zones that develop in the dam soil. Among the causes underlying the adverse SSS of the dam is the abrupt difference in the deforming ability between the soil of retaining prisms and the cemented material of the grout curtain. As the dam carries its own weight, retaining prisms are suspended on the rigid grout curtain, while substantial tangential stress arises at their contact point. In the top and middle tiers of the dam, where such tangential stress is not too high, calculations showed that shear strength of soil was compromised, and as a result the retaining prisms tended to slip relative to the grout curtain.

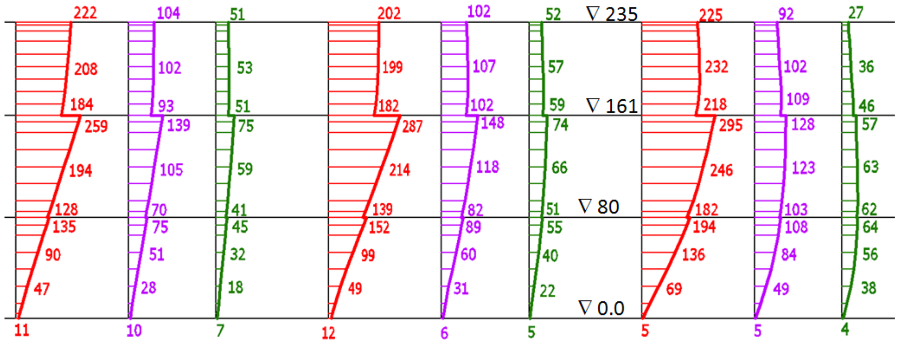
Adverse SSS of the dam body particularly emerges on the borderlines of construction phases, where there are changes in thickness of the grout curtain. Failure zones emerge here, but not on the boundary of prisms and the screen, but inside the body of the soil embankment proper. They are caused by irregular deformations: one part of the embankment can settle freely; while settlement of the other in the bottom tier of the screen is limited. This compromises strength of the soil resisting shear and strain. It can be concluded that use of a staggered profile in the multi-tier grout curtain is undesirable, and one should seek smooth outlines of the impervious element.

As the reservoir is filled, processes of strength failures emerging in the body of the earthen embankment get more intense. Driven by hydrostatic pressure, the earthen embankment shifts towards the tail reach (Fig. 2, 3). Because the dam is very tall, settlements are considerable. In Series A scenarios, greatest core settlements amount to 238÷268 cm respectively. Such settlements in the top prism creates a sliding wedge, even as the top prism slides along the vertical pressure face of the impervious element, because lack of compressive stress $\sigma_x$ reduces soil resistance to shear dramatically. Due to the resulting sliding wedge, the top of the dam's ridge settles by more than 1 meter in Series A scenarios.



a) var. 1A b) var. 1B c) var. 1C d) var. 2A e) var. 2B f) var.2C g) var.3A h) var.3B i) var.3C

**Figure 2. Grout curtain settlements (cm) in various scenarios (estimated using the soil model by prof. L.N. Rasskazov)**



a) var. 1A b) var. 1B c) var. 1C d) var.2A e) var.2B f) var.2C g) var.3A h) var.3B i) var.3C

**Figure 3. Grout curtain settlements (cm) in various scenarios (estimated using the Coulomb-Mohr soil model)**

A sliding wedge also emerges in Series C scenarios, where greatest settlements of the grout curtain amount to some 100 cm (Figs. 2, 3); however, settlement of the top prism becomes less manifest. One can conclude that to ensure performance of a super-tall rockfill dam, very high consolidation of the filler rock is needed so that its modulus of deformation is at least 300 MPa.

Now let us consider SSS (stress-strain state) of the impervious element: the grout curtain. As we examine SSS of the grout curtain, we need to consider an important factor: stress in the cement rock, i.e. hardened mortar can develop stress only after injection is completed. Therefore, material of the grout curtain evidently should be correctly regarded as a compound of two components: rock skeleton and cement rock located in the hollows of the skeleton. Each component carries loads of different levels. Strength of the grout curtain should be evaluated based on stresses that exist only in the cement rock. To measure such stresses, we need to take total stresses borne by the grout curtain's material, and subtract from the figure the stresses borne by the soil before it is filled with injected slurry.

As an illustration, Figure 4 compares SSS of the grout curtain in scenario 1B for the case of total stress, and stresses existing in the cement rock. Obviously, differences are material in terms of strength analysis of the grout curtain. If we evaluate strength by stresses borne by injection-filled soil as a whole, then existence of stretching stresses registers only in the bottom tiers of the screen (Fig. 4a). And when we evaluate stresses borne only by the cement rock, we can find crack formation in all three tiers of the screen (Fig. 4b).

Figures 2, 3, 5, and 6 show stresses and settlements that emerge in the cement rock of the grout curtain, disregarding the stresses and settlements that existed in different tiers before the grout curtains are built. The settlements curves in Figures 2 and 3 have staggered shapes because each tier appeared at different points of time.

Analysis of SSS demonstrates that working conditions differ for each tier of the grout curtain. The top tier – the last one built – does not carry the dam's own weight, mainly having to withstand just hydrostatic pressure of water. This tear resists the bending force. The bottom and middle tiers, in addition to hydrostatic pressure, have to bear the dam's own weight, created by settling soil and then transferred with friction to the screen. Consequently, the bottom tier and the middle tier not only have to resist the bending force, but also the longitudinal compressing forces.

Bending deformations of the grout curtain are rather complex by nature. This work can be described as that of a console restrained in the rock base. The top and middle tiers bend towards the tail reach, and the lower tiers work towards the head reach. Thus, stretching vertical stress $\sigma_y$ emerges in the lowest facet of the top tier, while the top tier has to resist compressing forces (Figs. 5, 6). In the middle tiers, the bend is faint, as possible stretching stresses are overcome by compressing longitudinal forces.

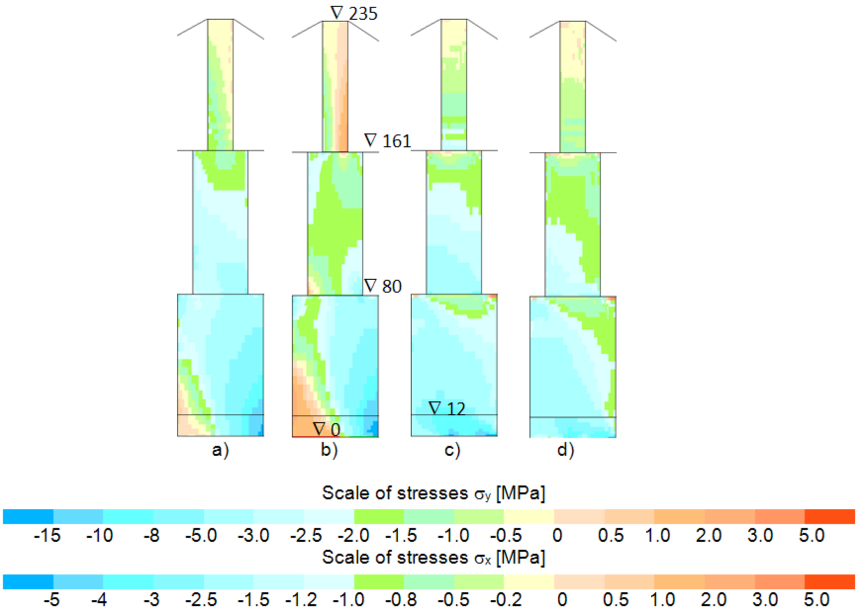


**Figure 4. Comparing stresses (MPa) in the grout curtain for scenario 2C: a, b – $\sigma_y$, c, d – $\sigma_x$, a, c – total stresses borne by grout curtain material; b, d – stresses borne by cement rock.**

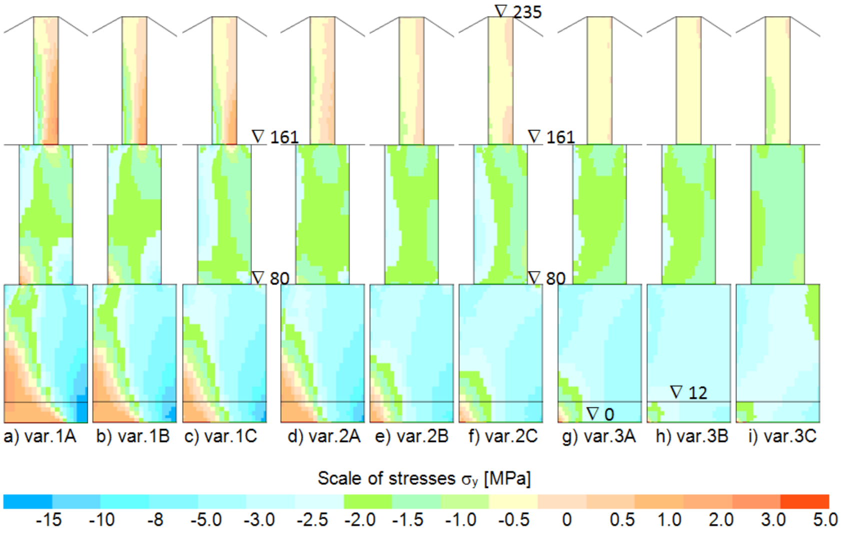


**Fig. 5. Vertical stresses $\sigma y$ in the grout curtain, building up after construction and until the reservoir is filled with water (estimate based on soil model by Prof. L.N. Rasskazov)**

The strongest bend deformations bear on the bottom tier of the screen. This occurs because its deformations are limited by the rigid bedrock that underlies it. Despite serious longitudinal compressing forces existing in the bottom tier, its top facet has a zone of stretching stresses $\sigma_y$.

Bend deformations are especially notable in the contact section, where the injection core contacts the bedrock. On the side, on the bottom facet in the contact section, there is a zone of concentrated compressing stresses $\sigma_y$, while at the top the contact seeks to open due to straining strength failure.

The above quality image of SSS of the grout curtain is typical for all estimation scenarios, and for any of the two soil models. Differences in the SSS for estimated scenarios are merely quantitative. The grout curtain has a more favorable SSS in the event of low deforming ability of filler rock and high deforming ability of the grout curtain material.

For scenarios, where the grout curtain is made of rigid material (Series 1 scenarios with high cement content), its SSS is extremely adverse under any option of deforming ability of filler rock (Fig. 5a–c, 6a–c). In the top and middle tiers, stretching stresses $\sigma_y$ can be up to 2÷3 MPa, greater than the straining force in the cement rock. This force opens up the seam between the upper and middle tier from below. The middle tier is in better strained condition, it also may develop stretching stresses yet. In the lower tier, the material is prone to both compression and tension. In each Series 1 scenario, the contact seam opens to considerable length that can compromise the dam's reliable operation.

In scenarios that have the grout curtain made of a more pliant material (Series 2 scenarios with cement-bentonite slurry injected), SSS is much better, but still adverse (Fig. 5d–f, 5d–f). In many zones of the impervious element, the material is not strong enough to resist strength either compression or tension, and the contact seam opens rather wide. Only scenario 2C with failure zones of local sizes (Fig. 5f, 6f) can be considered as relatively robust.

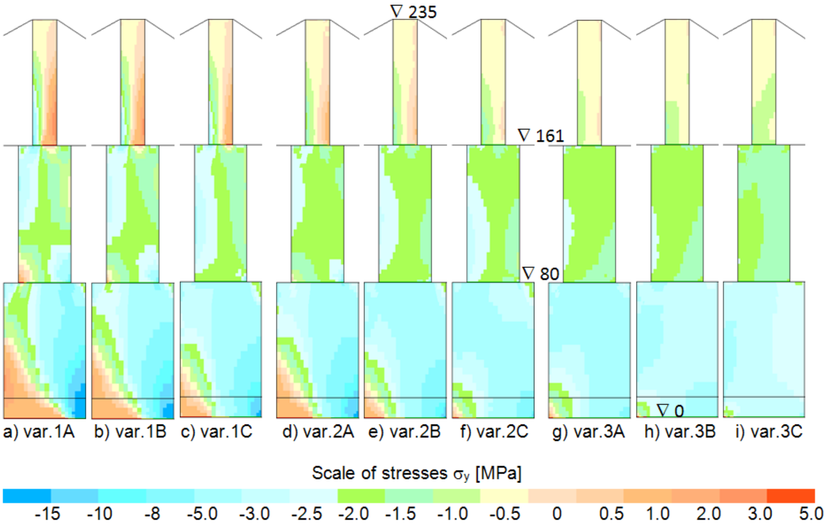


**Figure 6. Vertical stresses $\sigma_y$ in the grout curtain, building up after construction and until the reservoir is filled with water (estimate based on the Coulomb-Mohr soil model)**

SSS of the grout curtain can be regarded favorable only when bentonite-cement slurry is injected (Series scenarios 3) (Fig. 5g–i, 6g–i). In these scenarios, no stretching stress emerges in the top and middle tiers, and compressing stresses roughly match the adopted material strength for single-axis compression. Given that strength of plastic materials rises along with lateral reduction, one may expect that compression strength is assured, because the screen's material withstands compression on every side all around. In most of the screen's bottom tier, compression and straining strength is in fact assured, yet singular zones exist where failures are possible. Scenarios 3B and 3C (Fig. 5h–i, 6h–i) can be regarded as usable, since their failure zones are local by size and cannot jeopardize the structure's integrity, and their contact seam only opens over an insignificant length. In these scenarios, the deforming ability of the grout curtain material roughly matches that of the filler rock.

Scenario 3A can be regarded as conditionally usable. In it, compressing stress $\sigma_y$ of 2÷3 MPa embraces virtually the entire bottom tier of the core, and this level of stress is approximately equal to or greater than the strength of clay-cement rock withstanding single-axis compression (Fig. 5g, 6g). At the same time, strength can be assured, if the compression stress is considered.

***Table 2. SSS of the grout curtain for estimated scenarios (based on Prof. L.N. Rasskazov soil model)***

| **tier** | **value** | **scenarios** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | 1A | 1B | 1C | 2A | 2B | 2C | 3A | 3B | 3C |
| top | max $\sigma_y$, MPa | 3.3 | 2.4 | 2.6 | 1.0 | 0.7 | 0.9 | 0.1 | 0.02 | 0.0 |
| | min $\sigma_y$, MPa | -3.7 | -2.4 | -3.5 | -0.9 | -0.9 | -0.8 | -0.4 | -0.5 | -1.0 |
| middle | max $\sigma_y$, MPa | 2.3 | 1.4 | -0.5 | -0.1 | -1.1 | -1.1 | -1.2 | -1.3 | -0.8 |
| | min $\sigma_y$, MPa | -8.1 | -7.0 | -5.1 | -4.1 | -2.5 | -3.3 | -2.3 | -2.3 | -1.9 |
| bottom | max $\sigma_y$, MPa | 3.3 | 2.3 | 2.1 | 2.1 | 1.4 | 1.3 | 1.7 | 0.8 | 0.7 |
| | min $\sigma_y$, MPa | -54.4 | -34.9 | -29.4 | -21.5 | -14 | -12.2 | -6.0 | -4.2 | -4.0 |
| contact | L, m | 36 | 26.5 | 18 | 21 | 8 | 6 | 2,5 | 1 | 1 |
| | P, mm | 365 | 106 | 52 | 172 | 18 | 7.0 | 11.6 | 0.9 | 3.3 |

Note: L – opening length over contact between grout curtain and base,

P – greatest opening at contact between grout curtain and base.

***Table 3. SSS of the grout curtain for estimated scenarios (based on Coulomb-Mohr soil model)***

| **tier** | **value** | **scenarios** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | 1A | 1B | 1C | 2A | 2B | 2C | 3A | 3B | 3C |
| top | max $\sigma_y$, MPa | 4.4 | 3.3 | 2.5 | 1.7 | 1.1 | 0.5 | 0.2 | 0.06 | 0.04 |
| | min $\sigma_y$, MPa | -6.5 | -4.6 | -3.6 | -1.8 | -1.5 | -1.7 | -1 | -1.2 | -1.2 |
| middle | max $\sigma_y$, MPa | 2.3 | 1.3 | -0.9 | 0.3 | -1.1 | -0.9 | -1.2 | -1.2 | -1.1 |
| | min $\sigma_y$, MPa | -8.7 | -6.9 | -3.1 | -5 | -2.7 | -2.9 | -2.4 | -2.4 | -2.0 |
| bottom | max $\sigma_y$, MPa | 3.2 | 2.2 | 1.8 | 2.2 | 1.5 | 1.1 | 1.5 | 1.1 | 0.3 |
| | min $\sigma_y$, MPa | -61.7 | -41 | -28.1 | -24.9 | -16.6 | -12.1 | -7.6 | -5.3 | -4.1 |
| contact | L, m | 38 | 29.5 | 18.5 | 21 | 12 | 4 | 2.5 | 2 | 1 |
| | P, mm | 424 | 151 | 43 | 194 | 42 | 9 | 24 | 3.3 | 1.0 |

Therefore, slurries with high bentonite content should be used to build grout curtains as this helps to bring the properties of manmade material's deforming ability close to soil in the dam body, thus resulting in better SSS. In addition, banked soil in the dam body needs to be better compacted to reduce its deforming ability. According to calculations, SSS of the grout curtain is favorable only where its material's deformation modulus is below or nearly the same as that of the soil in the dam body.

The study has also revealed the danger of fissures emerging in the grout curtain material as the dam is built, even before it is exposed to any hydrostatic force. The upper part of the screen's tier were discovered to develop stretching stresses $\sigma_x$ that may cause vertical fissures in it. Such fissures can subsequently compromise water tightness of the impervious element. Stretching stresses $\sigma_x$ in the upper part of the tier may emerge when the next phase of the dam is added on top of it. Under its own weight, the earthen embankment works to expand on both sides, but this is prevented by the rigid grout curtain. This however causes the upper part of the grout curtain's tier to stretch.

Greatest stretching stresses $\sigma_x$ were observed on the tier edges, where they can exceed 2 MPa (Table 4). The zone in which the two tiers of the core are subsequently going to close have weaker stretching stresses $\sigma_x$ at around 1 MPa. Such stresses are the same or greater than the stretching strength of cement rock, and so vertical fissures are quite probable. Stretching stress $\sigma_x$ does not appear in scenarios 3A and 3B that use bentonite-cement slurry to build the grout curtain.

***Table 4. Highest stretching stress $\sigma_x$ (MPa) in bottom tier of the core being built***

| **value** | **scenarios** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1A | 1B | 1C | 2A | 2B | 2C | 3A | 3B | 3C |
| total maximum | 2.3 | 2.4 | 2.0 | 2.1 | 2.0 | 1.4 | 0.5 | 0.1 | - |
| maximum at contact with tier No. 2 | 1.1 | 1.2 | 1.0 | 1.0 | 0.9 | 0.7 | 0.9 | - | - |

## *Conclusions*

1. Adverse SSS is typical for super-tall earthen dams with central impervious elements (vertical screens or cores). Hydrostatic pressure causes horizontal settlements in such dams, and they can be strong enough to create a collapsing soils in the upstream retaining prism. To ensure reliable operation of such dams, filler rock must be compacted very dense to achieve the deformation module of at least 300 MPa.

2. Working conditions for a massive impervious element such as a grout curtain are similar to those required for thin-wall impervious cores built using the diaphragm wall method. Its SSS is mainly the result of bending deformations, but also longitudinal compressing forces created by settling surrounding soil. Yet, unlike a thin-wall core, massive injection cores themselves can bear most of the hydrostatic load that acts on the dam. Besides, exposed to bending deformation, massive injection cores may develop serious stretching stresses that bring the risk of fissures with loss of water tightness in the impervious element.

3. Strength analysis of the grout curtain material is necessary for the part of stress that is applied to hardened mortar in pores of the injection-processed soil. These stressing forces emerge as soon as the grout curtain is built; they include the compressing stresses created in the injection-processed soil under its own weight. This is important because SSS of cement rock is more adverse than that of integral material of the grout curtain.

4. According to the estimate, SSS of the grout curtain will be favorable only when its material's deformation modulus is under 5 times that of the soil in the dam body, which in turn should be at least 200 MPa. To ensure reliable performance of the impervious screen, one needs, on the one hand to inject clayey-cement slurries that increase the deforming ability of the screen material, and on the other hand to try and reduce the deforming ability of filler rock by making it more dense.

5. While selecting the profile of the impervious element as a multi-tier grout curtain, one should not use the staggered or stepwise profile (with screen width changing abruptly), because this will cause irregular deformation of the dam body and may create strength failure zones in it.

6. The results of SSS estimated for earthen dams, produced using two soil models, are fairly close, particularly in terms of grout curtain SSS evaluation. First of all, this suggests accuracy of the results; and secondly, it demonstrates that in terms of SSS of manmade structures, the non-linear nature of soil deformation is not too critical to consider as the structure bears loads. To estimate SSS for earth structures, it is most important to correctly discriminate the deforming ability of soil actively loaded and unloaded.

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