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MODELING AND MONITORING OF STRUCTURAL SAFETY OF LONG-OPERATING UNDERGROUND STRUCTURES OF THE SEWAGE SYSTEM IN THE CONDITIONS OF INCREASING ANTHROPOGENIC ACTIONS IN ORDER TO PROVIDE SUSTAINABLE LIFECYCLE OF ENGINEERING INFRASTRUCTURE OF THE MEGACITY (THE EXPERIENCE OF ST. PETERSBURG)

Nickolai A. Perminov [1]***, Rashid A. Mangushev*** [2]

[1] Emperor Alexander I Saint-Petersburg State Transport University, Saint-Petersburg, RUSSIA
[2] Saint-Petersburg State Architecture and Construction University, Saint-Petersburg, RUSSIA

Abstract. Long-term operation in difficult engineering-geological conditions of unique underground sewage facilities creates the danger of violating their structural safety. A long-term study of the dynamics of changes in a technical state of large pumping stations and deep sewage tunnels made it possible to establish patterns of influence of intensive anthropogenic and dynamic actions on this process. The developed discrete and continuous diagnostic models of defect development in tunnel structures allow identifying potentially hazardous areas subjected to manifestation of critical failures and methods of their localization. On the basis of numerical modeling the boundaries of defect-free joint operation of the system “source of impact – geo-mass – sewage underground structure” have been determined. The geotechnical and structural calculations are used to simulate the interaction of the facilities with the soil environment and predict adaptive stress-strain control system parameters. With increasing external anthropogenic and dynamic impacts, modeling zones of urban areas with potentially dangerous sections of underground sewage facilities constitute the basis for development of regulatory documents on monitoring methods and safe development of the geotechnical infrastructure of a megacity.

Keywords: unique underground sewer structures, structural safety, complex ground conditions, modeling and monitoring, man-made impacts

МОДЕЛИРОВАНИЕ И МОНИТОРИНГ КОНСТРУКЦИОННОЙ БЕЗОПАСНОСТИ ДЛИТЕЛЬНО ЭКСПЛУАТИРУЕМЫХ ПОДЗЕМНЫХ СООРУЖЕНИЙ СИСТЕМЫ ВОДООТВЕДЕНИЯ В УСЛОВИЯХ ВОЗРАСТАЮЩИХ ТЕХНОГЕННЫХ ВОЗДЕЙСТВИЙ В ЦЕЛЯХ ОБЕСПЕЧЕНИЯ УСТОЙЧИВОГО ЖИЗНЕННОГО ЦИКЛА ИНЖЕНЕРНОЙ ИНФРАСТРУКТУРЫ МЕГАПОЛИСА (ОПЫТ САНКТ-ПЕТЕРБУРГА)

Н.А. Перминов [1]***, Р.А. Мангушев*** [2]

[1] Санкт-Петербургский государственный университет путей сообщения имени императора Александра I, Санкт-Петербург, РОССИЯ
[2] Санкт-Петербургский государственный архитектурно-строительный университет, Санкт-Петербург, РОССИЯ

Аннотация. Длительная эксплуатация в сложных инженерно-геологических условиях уникальных подземных канализационных сооружений создает опасность нарушения их конструкционной безопасности. Многолетнее изучение динамики изменения технического состояния крупногабаритных насосных станций и туннелей глубокого заложения позволило установить закономерности влияния интенсивных антропогенных и динамических воздействий на этот процесс. Разработанные дискретные и непрерывные диагностические модели развития дефектов конструкций тоннелей позволяют выявить потенциально опасные участки, подверженные проявлению критических отказов и способы их локализации. На основе численного моделирования определены границы бездефектной совместной работы системы “источник воздействия – геомассив – подземное канализационное сооружение. Посредством совместного выполнения геотехнических и конструкторских расчетов моделируются процессы взаимодействия оборудования с грунтовой средой и прогнозируются параметры адаптивного управления напряженно-деформированным состоянием системы. При возрастающих внешних антропогенных и динамических воздействий моделирование зон городских территорий с потенциально опасными участками подземных канализационных сооружений является основой для разработки нормативных документов по методам мониторинга и безопасному развитию геотехнической инфраструктуры мегаполиса.

Keywords: уникальные подземные сооружения, конструкционная безопасность, сложные грунтовые условия, моделирование и мониторинг, техногенные воздействия

1. INTRODUCTION. GENERAL FEATURES OF THE PROBLEM UNDER SOLUTION

At development of big cities unique underground sewage structures require special protection against anthropogenic actions. Sewage pump stations and tunnels as the facilities of an increased level of responsibility should meet the requirements of safe operation excluding the risk of emerging dangerous failures [1, 2]. The analysis of data on a current technical state of large pump stations (the depth of lowering - down to 71 m, the diameter - up to 66 m) and deep sewage tunnels (the total length – more than 2500 km) in more than ten largest cities of Russia with developed historical downtowns allowed developing methods of evaluation of their technical state, make a classification and a catalogue of defects.

In order to identify causes of defects at operation of underground pump stations (violation of integrity of a structure shell, force cracks, corrosion of concrete and reinforcement due to leakages) there were analyzed processes of construction and lowering of a large RC shell. During the sinking of large-size tempering structures, specific conditions of their interaction with the ground massif manifest themselves. Due to the inclusion of the scale effect (factor) (by the hyper size of the side surface area of the shell interacting with heterogeneous soil S=14500m2 and its super large mass G=1,2•106кН) creating a powerful kinetic momentum at instantaneous, most often sudden, landings of the lowering structure [3, 4]. The joint manifestation of these factors is responsible for the specific, non-linear behavior of the structure during sinking and the host soil mass. The strength and deformability of a large-scale massive structure, its geometric variability should be calculated not only for the final stage of construction. Still for the entire history of immersion, taking into account the history of the interaction of the shell with the soil massif during immersion and consequently the effect of stage-by-stage inheritance of the stress. That can only be done using nonlinear problem solving and computer modeling.

The analysis of the experimental results presented in the article showed that the main defects leading to failure of shell integrity and cracking occurred during the erection of the underground part in the soil mass. Thus, the main task is to ensure the operation of the structure in up to the limit modes at the stage of its life cycle during the erection. In order to be able to realise these conditions, it is crucial to assess the actual structural performance taking into account the process of its stage-by-stage erection in the soil mass under the non-linear material properties of the structure and the ground. These conditions can be taken into account to build a correct model of the interaction history of the shell during its step-by-step insertion into the soil mass. Calculated justification of the range of preliminarily changes in the stress-strain state of the mega massive shell when it is immersed in heterogeneous soils will ensure a defect-free life cycle of the underground structure at the stage of erection [5]. The analysis of the shell loading history at the stage of its erection taking into account the effect of VAT inheritance allows to create an adequate calculation and analytical model of the underground structure and to choose a rational calculation method for predicting the dynamics and spatial boundaries of stress-strain state (SSS) changes in the reinforced concrete shell structure, ensuring defect-free structure at all stages of its immersion.

The methodological approach proposed in the article allows you to move from the previously adopted method of calculation on the sinking (Handbook of geotechnics. Edited by V.A. Ilyichev and R.A. Mangushev, 2016) of large-size underground water disposal facilities erected by tempering method to the concept of modeling and prediction of defect-free life cycle at the stage of their construction. The results of experimental and theoretical studies represented in the article convincingly show that modeling and calculated justification of preventive protection parameters by geotechnical methods of underground construction at the stage of erection will ensure its safety and stability to man-made impacts at subsequent stages of the life cycle during long-term operation.

The paper draws special attention to investigation of structural safety of deep sewage tunnels, long-operating in the bulk of unstable soils of different strength at growing anthropogenic and dynamic actions. For almost most of the cities under consideration, the network of tunnel collectors has an average value of the physical deterioration degree more than 60% with a development dynamic of 0.6-1.2% per year. It was found that for the cities where the operation of the engineering infrastructure is carried out in complex engineering and geological conditions typical, for example, for St. Petersburg, the degree of the tunnels wear is significantly greater, reaching 83% with a higher development dynamic of up to 1.6-2.1 % per year. Development of a methodology for identifying the potentially dangerous sections of tunnels operated for a long time in difficult soil conditions, with their subsequent modeling and monitoring, will ensure their structural safety with increasing man-induced dynamic impacts.

2. THE METHODS OF MONITORING AND GEOTECHNICAL EVALUATION OF NON-STATIONARY INTERACTION OF A LARGE SHELL WITH HETEROGENEOUS SOIL MILIEU

According to the results of field and calculated-experimental works and data of complex system of geotechnical monitoring (Figure 1) of large-sized (D=50÷60m and H=55÷71m) sinking wells the peculiarities of their interaction with heterogeneous soil medium during sinking were studied. The heterogeneity of soil strata is characterized as follows: the upper stratum is represented by quaternary strata to a depth of 14.0-25.0 m (dusty sands of medium density, water-saturated, E = 11 MPa, C = 0.005 MPa, φ = 30°; dusty loamy sandy loam, E = 4 MPa, C = 0.01 MPa, φ = 15°; dusty loamy layered fluid plastic, E = 9 MPa, C = 0.025 MPa, φ = 16°; dusty loamy semi-solid with gravel, pebbles, E = 14 MPa, C = 0.028 MPa, φ = 28°), the lower one - the roof of Proterozoic clays of dislocated solids (E = 19 MPa, C = 0.04÷0.06 MPa, φ = 18÷21°).

The geomonitoring structure included: 1) program complex of calculations and the geomassive stress under different erection modes; 2) technical means of instrumental observations and SSS control of the separate elements of the system "structure - geomassive"; 3) information-measuring system of gathering, processing, storage and identification of parameters (data) of observations and control; 4) geotechnical methods of the influence on the geometric massif and soil and structure stress.

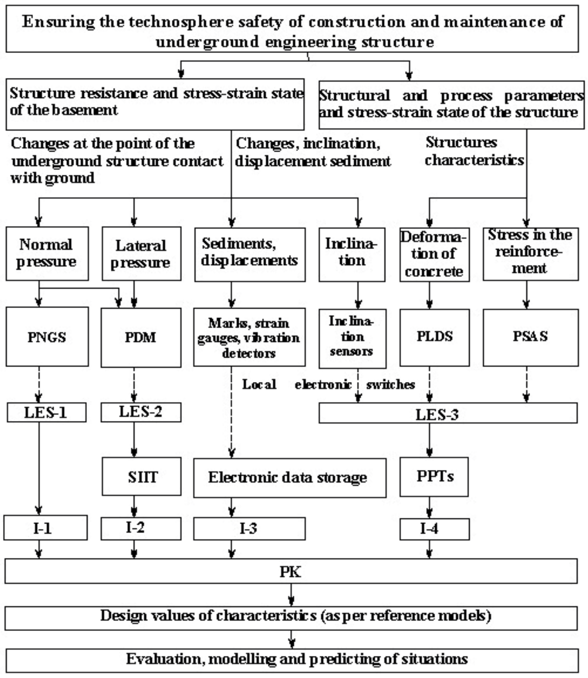
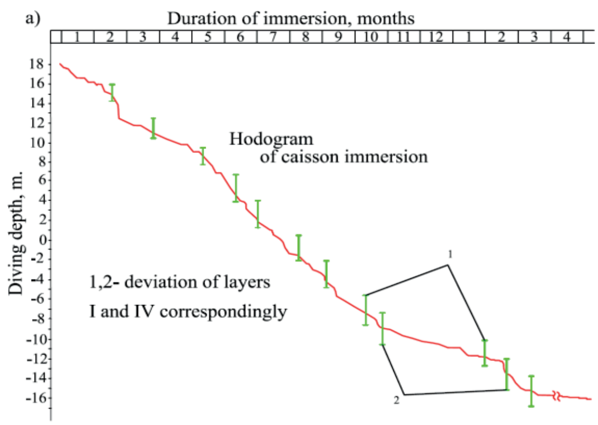


Figure 1. Geocomplex of monitoring and regulating stress-strain behavior of an embedding geobulk.

The monitoring established the peak values of horizontal stresses at the moment of "roll", exceeding the calculated values by more than 2.5 times. This can cause the appearance of microcracks in the concrete structure, which will inevitably lead to violations of waterproofing of the structure. The consequences of this circumstance were noted after 15-20 years of culverts' operation by the inevitable failure of their airtightness (Figure 2).



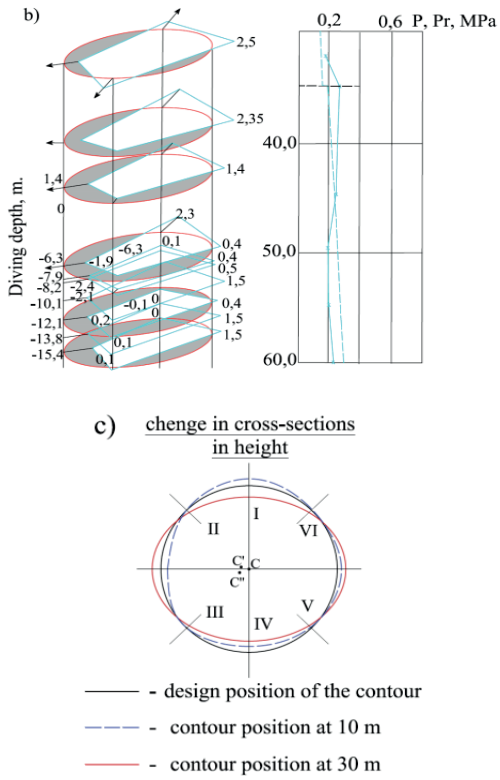


Figure 2. Sinking stages and monitoring of the stress-strain state of the large-sized sinking well: a - sinking hodogram; b, c - volume diagram of the shell displacement and the stress-strain state (SSS) of the shell

According to the analysis of sinking process and stress-strain state of large-sized shell, different, even alternating, stress-strain state of "large-sized shell-soil mass" system and different types of pressures (SP22.13330) of ground medium on the shell, including resting pressure, active and passive pressure (see Figure 3) are observed at different sinking stages.

The strength and deformability of a large-scale massive structure, its geometric variability, must be calculated not only for the final stage of construction, but for the entire history of immersion, taking into account the history of the interaction of the shell with the ground massif during immersion.

According to the general theory the ground pressure on the walls of the well at rest can be determined from the expression:

$$\sigma_o(z) = \sigma_x(z,u_x)\big|_{u_x=0} = \lambda_0 \gamma z \tag{1}$$

where λ_0 - coefficient of lateral pressure of the ground at rest; γ - specific weight of the ground; z - distance from the ground surface to the point in question.

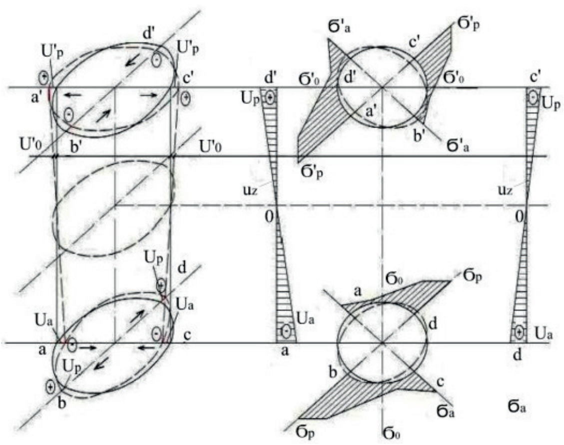
At displacements of the manhole shell wall > 0.005h (SP22.133 p.9.21) from the ground at depth z, the active pressure on the enclosure, σ_a which corresponds to the minimum pressure value, is realized. The passive pressure σ_p, is realized at much larger displacements of the wall on the ground ($U_p = 0{,}01 - 0{,}02h$) and corresponds to the maximum value of pressure.

If there is no load on the ground surface, the expressions for determining the active and passive pressures are as follows:

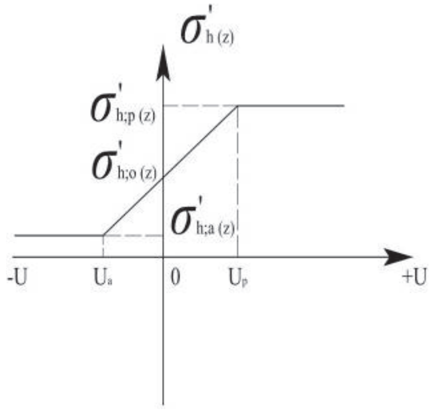
$$\sigma_a(z) = \lambda_a \gamma z - c\lambda_{ac} \quad (2)$$

$$\sigma_p(z) = \lambda_p \gamma z + c\lambda_{pc} \quad (3)$$

where: λ_a - coefficient of active ground pressure; - λ_{ac} coefficient for the influence of ground cohesion on the active pressure; λ_p - coefficient of passive ground pressure; - λ_{pc} coefficient for the influence of ground cohesion on passive pressure; c - specific ground cohesion.



<u>Figure 3.</u> Asymmetric deformation (displacement) of the KGOK shell



<u>Figure 4.</u> Dependence of lateral soil pressure on the shell on displacements $u_x \in (u_p, u_a)$ according to clause 9.21 (SP 22.13330)

The dependence of the effective horizontal pressure of the ground on the retaining structure in the interval u_x (u_p, u_a) has a complex character (Figure 4).

The active and passive pressures of the ground on the enclosure constitute the pressure limits, that is, the effective pressure is always in the range:

$$\sigma_a(z) \le \sigma_x(z, u_x) \le \sigma_p(z) \quad (4)$$

The dependence of the effective horizontal ground pressure on the holding structure in the interval $u_x \in (u_p, u_a)$ has a complex character (Figure 4).

In Figure 3 and considering Figure 4 we show the character of asymmetric deformations (displacements) of the shell contour according to the diagram of dependence of horizontal soil pressure on the walls of the well depending on the character of its displacement (asymmetric contraction-expansion of the shell in its upper and lower parts) and the diagram of lateral soil pressure, when approximating it by piecewise linear function.

The function of change of pressure value σ_x at some depth z from displacements can be represented as follows:

$$\sigma_x(u_x) = \begin{cases} \sigma_p, & u_x \le u_p \\ f(u_x), & u_p < u_x < u_a \\ \sigma_a, & u_a \le u_x \end{cases} \quad (5)$$

With some assumptions, the function

$$f(u_x) = \sigma_0 - k u_x \tag{6}$$

where k is the stiffness coefficient of the ground; σ_0 - ground pressure at rest.

The ground stiffness coefficient can be used as the ground stiffness coefficient.

The resulting pressure along the bottom and top sections of the well wall is the sum of the effective pressures on both sides of the enclosure. Let us present in the form of two graphs the effective ground pressure on the wall of the well from the ground (left) and the excavation (right) depending on the horizontal displacement of the well shell (Figure 3).

Construct the function as a $\sigma_x(z,u_x)$ piecewise given function for any value of z.

To describe the effective pressures $\sigma_x(z,u_x)$ for individual sections of the diagram between the active and passive pressure limits $\sigma_a(z) = \lambda_a \gamma z - c\lambda_{ac}$ $\sigma_p(z) = \lambda_p \gamma z + c\lambda_{pc}$, instead of (a), (b), (c), (d) we will use (1), (2), (3), (4), adding indices "l","r" for the terms relating to the axis of contraction and expansion of the well diameter. In the case where the knife part of the wall of the well is surrounded on both sides by the soil mass $\sigma_x(z,u_x)$ will take the form of:

$$\sigma_x(z,u_x) = \begin{cases} \sigma_p^l(z) - \sigma_a^r(z-h_k), & u_x \le u_1 \\ \sigma_0^l(z) - \sigma_a^r(z-h_k) - u_x k^l, & u_1 < u_x < u_2 \\ \sigma_0^l(z) - \sigma_a^r(z-h_k) - u_x(k^l + k^r), & u_2 \le u_x \le u_3 \\ \sigma_a^l(z) - \sigma_0^r(z-h_k) - u_x k^r, & u_3 < u_x < u_4 \\ \sigma_a^l(z) - \sigma_p^r(z-h_k), & u_4 \le u_x \end{cases} \tag{7}$$

If we separately consider the resultant pressures on the shell up to the face ($z \le h_k$), expression (7) will take the form:

$$\sigma_x(z,u_x) = \begin{cases} \sigma_p^l(z), & u_x \le u_1 \\ \sigma_0^l(z) - k^l u_x, & u_1 < u_x < u_3 \\ \sigma_a^l(z), & u_3 \le u_x \end{cases} \tag{8}$$

Let us substitute expressions (1), (2), (3) in (7) and (8):

$$\sigma_x(z,u_x) = \begin{cases} \lambda_{pl}\gamma z + c\lambda_{pcl}, & u_x \le u_1 \\ \lambda_{0l}\gamma z - k_l u_x, & u_1 < u_x < u_3 \\ \lambda_{al}\gamma z + c\lambda_{acl}, & u_3 \le u_x \end{cases} \tag{9}$$

$$\sigma_x(z,u_x) = \begin{cases} \lambda_p^l \gamma z - \lambda_a^r \gamma (z-h_k) + c\lambda_{pc}^l + c\lambda_{ac}^r, & u_x \le u_1 \\ \lambda_0^l \gamma z - \lambda_a^r \gamma (z-h_k) + c\lambda_{ac}^r - u_x k^l, & u_1 < u_x < u_2 \\ \lambda_0^l \gamma z - \lambda_a^r \gamma (z-h_k) - u_x(k^l + k^r), & u_2 \le u_x \le u_3 \\ \lambda_a^l \gamma z - \lambda_0^r \gamma (z-h_k) - c\lambda_{ac}^l - u_x k^r, & u_3 < u_x < u_4 \\ \lambda_a^l \gamma z - \lambda_p^r \gamma (z-h_k) - c\lambda_{ac}^l - c\lambda_{pc}^r, & u_4 \le u_x \end{cases} \tag{10}$$

The analysis of formulas (9) and (10) describing resultant pressures shows that practically independent of properties of the host soil mass, the sum of effective pressures on the asymmetric deformed shell (Figure 3) both in the opposite axial directions and in the lower and upper parts of the shell exhibit a high degree of nonuniformity. As comparative calculations show, the non-uniformity of the resulting pressures can be of the order of one unit (see Figure 5) or more, either on both sides of it, or along the formations of the lower and upper sections of the manhole walls. It is impossible to predict such character of the stress-strain state (SSS) of the "sinking structure-soil massif" system using the recommended calculation approach, as it was noted, and it is also impossible to ensure verticality and uniformity of sinking by applying previously known methods of geotechnology [6], as evidenced by hodograms (see Figure 2a).

The analysis of the conditions of interaction between a massive large-sized shell and the ground massif when immersed in heterogeneous strata testifies to the manifestation of non-stationarity effects in the parameters reflecting this process. In order to be able to study the regularities of manifestation and conditions for preventing their prohibitive development in our further studies, it is necessary to use simulation of shell immersion modes, solving for this purpose the problems in linear and nonlinear formulations.

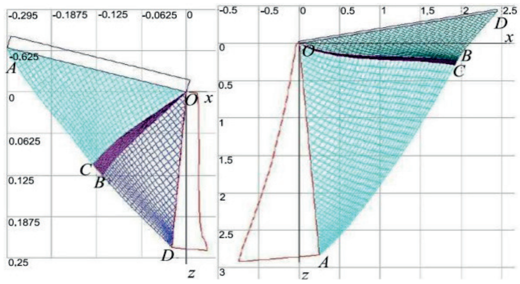


Figure 5. Slip lines for active and passive pressures on the retaining wall (by Korolev K.V.) 10Ea<<En

The strength and deformability of a large-scale massive structure, its geometric variability should be calculated not only for the final stage of construction. Still for the entire history of immersion, taking into account the history of the interaction of the shell with the soil massif during immersion and consequently the effect of stage-by-stage inheritance of the stress. That can only be accomplished using nonlinear problem solving, nonlinear models, and computer-based nonlinear modeling.

3. THE RESULTS OF MODELING AND ANALYSIS OF REGULATED MODES OF LOWERING OF A MASSIVE SHELL IN HETEROGENEOUS SOIL USING THE METHODS OF GEOTECHNOLOGY

3.1 Numerical modeling of the process of correcting a tilt of a massive embedded structure

In engineering practice, it is known that during the construction of large-diameter manholes there are often problems associated with the deviation of the structure from the design position. The causes of uneven sinking of the well, as it was found in section 2, are peculiarities of interaction of the large-sized shell with heterogeneous soil medium at the stage of its sinking and non-stationary nature of the stress-strain state of the system "large-sized lowering shell - the host soil mass".

By means of numerical geotechnical calculations it is proposed to choose technically possible geotechnological methods for controlling the sinking process: change of the geomassivation in the base of the structure and on the side surface, for example, by methods of prestressing the soil mass, by means of loading leader screens, regulation of the soil resistance on the side surface and other protective geotechnological measures,

In order to assess the effectiveness of geotechnical measures to correct the roll, several series of calculations were carried out on the ground massif with a buried structure. In the initial series of calculations, the soil base was modeled as a linearly deformable medium. In the subsequent series, the nonlinearly deformable material. As a linear medium, a model with a Hooke coupling between stresses and strains was used [7, 8]. An incremental model based on generalized Hooke's law was used to simulate nonlinear material.

Description of the nonlinear ground model

An incremental strain-type model was used as the computational ground model to solve the nonlinear problem. The relationship between stresses and strains in the model is taken separately for the volumetric and shear components of the stress tensor:

$$\left.\begin{aligned} dS_{ij} &= 2G^{\partial} \cdot de_{ij} \\ d\delta_{cp} &= 3K^{\partial} \cdot d\varepsilon_{cp} \end{aligned}\right\} \qquad (11)$$

Where: dS_{ij} - increment of the deviatoric component of the stress tensor; de_{ij} - increment of the deviatoric component of the strain tensor; $d\delta_{cp}$ - increment of the average stress; $d\varepsilon_{cp}$ - increment of the average strain; K^{∂} - differential volume strain; G^{∂} - differential shear strain.

The mathematical approximation of deviator loading is taken as a linear polynomial of two variables:

- under the condition of loading by the deviatoric component of the stress tensor

$$G_H^{\partial} = A_0 + A_1 \cdot \delta_{cp} + A_2 \cdot \tau_i \qquad (12)$$

- under the condition of unloading by the deviatoric component of the stress tensor

$$G_P^{\partial} = A_0 + A_1 \cdot \delta_{cp} \qquad (13)$$

Approximation of the differential volume strain modulus under the condition of loading by the spherical component of the stress tensor is carried out by a second-order power polynomial:

$$K_H^{\partial} = B_0 + B_1 \cdot \delta_{cp} + B_2 \cdot \delta_{cp}^2 \qquad (14)$$

at "unloading": $K_P = const$

where: τ_i - tangential stress intensity; δ_{cp} - average stress; $A_0;A_1;A_2;K_p;B_0;B_1;B_2$ - model design parameters.

Parameters of the computational model $A_0;A_1;A_2;K_p;B_0;B_1;B_2$ (12-14) were determined from the data of three-axis tests in the stabilometer. The tested soil is a sandy soil of medium coarseness with density Pd=1.65g/cm3 and humidity W=10%.

The procedure for solving the nonlinear problem was reduced to the well-known method of variable stiffness [9, 10], according to which the stiffness matrix was reshaped at each step of the solution according to the current level of SSS and the orientation of the overload vector.

As measures for leveling the roll of a buried structure can be chosen the method of regulation of ground resistance by electroosmosis or the transfer of horizontal pressure on the ground, based on immersion in an array of soil elastic shell, in which by special technology creates excessive pressure, transmitted through the walls of the shell on the ground [11]. The elastic casings are placed to some depth along the wall of the buried structure on the outer side. Then pressure is transferred to the inner cavity of the shell, which is transferred to the wall of the structure on one side and to the ground on the other side.

The calculation scheme is a soil mass measuring 296.0 m (horizontal) by 115, 0 m (vertical). In the central part of the scheme is a rigid buried structure having a length of 50.0 m in plan and is buried at 45.0 m.

The computational scheme is discretized into 246 quadrangular isoparametric elements. The total number of nodes was 282. The computational domain is represented by two groups of elements with different deformation characteristics. The elements of the first group (rigid buried structure) are represented by an elastic material with an elastic modulus E=30000.0MPa and Poisson's coefficient V=0.18.

The surrounding structure space is represented by a group of linearly deformable elements No. 2 with strain modulus E=30.0 MPa and Poisson's coefficient V=0.33.

In solving the problem, it is assumed that the structure has an initial roll, as shown in Fig. 6 (left to right). To correct for uneven subsidence on the right side of the structure is applied additional load intensity Q on the section of length L = 12.5 m.

The load Q in the solved problems was taken equal to 0.3; 0.6 and 0.9 MPa.

Numerical solution of geotechnical problems allowed to obtain the following results. Moving the contour of the dip well in a continuous elastic medium while adjusting the action of the lateral additional load Q and simultaneously adjusting the soil resistance on the lateral surface provided predicted prevention of rolls when sinking in heterogeneous soils. The displacements of the structure according to the solution of the nonlinear problem are shown in Figure 6. The greatest prevention of absolute horizontal displacement at a load of Q=0.9 MPa is as follows:

1st series of calculations - 48 cm; 2nd series of calculations - 67 cm; 3rd series of calculations - 80 cm; 4th series of calculations - 16 cm.

Table of displacements of corner points of the structure

№ points	Q = 0,3 kg/cm²		Q = 0,6 kg/cm²		Q = 0,9 kg/cm²	
	U_y [cm]	U_z [cm]	U_y [cm]	U_z [cm]	U_y [cm]	U_z [cm]
1	-6,7	9,16	-13,4	18,3	-20,2	27,5
2	-16,0	9,17	-32,0	18,3	-48,0	27,5
3	-6,7	-1,08	-13,3	-2,1	-20,0	3,2
4	-16,0	-1,06	-31,9	-2,1	-47,8	3,2

E = 30 MPa
ν = 0,33

Moving the structure in a controlled immersion mode

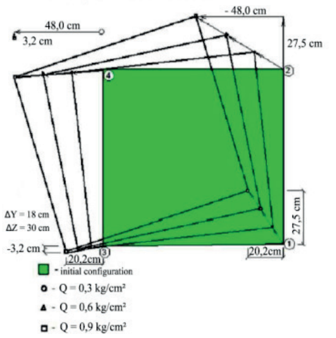


Figure 6. Contour displacement of the dip well in a solid elastic medium under the action of lateral load: Q=0.3; 0.6; 0.9 (MPa)

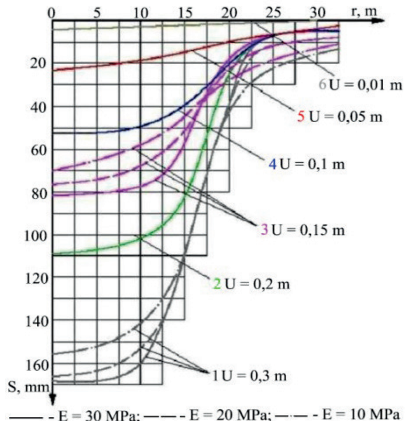


Figure 7. Influence of horizontal displacements of manhole walls U_k on ground surface settlements under roll

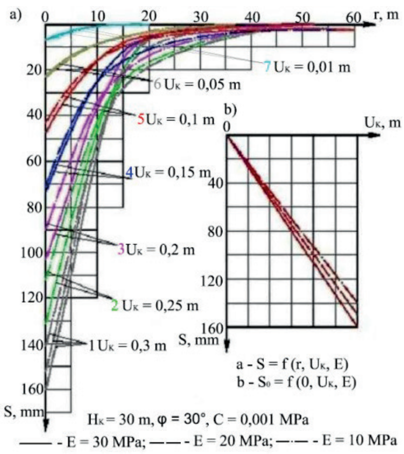


Figure 8. Influence of U_k value control on ground surface settlement in the mode of controlled structure landing

For all solved problems, without the application of geotechnical regulation methods, the irregularity of sinking of the lowered structure was observed: vertical movements (raising (+); lowering (-)) of the left and right contour of the buried structure with the corresponding sign is given below:
Series 1 calculations -3.2 and +27.5 cm; Series 2 calculations -12.5 and +36.4 cm; Series 3 calculations -25.7 and +41.1cm; Series 4 calculations -3.2 and +15.9cm.
When the zones with a reduced deformation modulus are taken into account in the calculations (the drilling zone), the horizontal displacement of the upper part of the structure has increased almost 2 times.
By comparing the results for different sizes of the drilling area, it can be seen that for the same E*, the increase in size by 2 times leads to a displacement increase of about 15-30%.
The geometric dimensions and configuration of the SSS control zones were selected by analyzing the displacement calculation data (Figure 7, 8). When solving the nonlinear problem (4th series of calculations), the displacements were obtained significantly less than when solving elastic problems. This fact can be explained by the considerable deformation heterogeneity in the soil surrounding the buried structure when solving the non-linear problem. The strain modulus when solving a nonlinear problem depends significantly on the stress state. Because of this, the strain modulus increases with depth. We also note that for the nonlinear solution there was no drilling zone, as well as the thixotropic jacket located around the structure was not modeled

3.2 Modeling of conditionally instantaneous failure of a massive shell when it is immersed in an inhomogeneous soil medium

Using the software package Autodesk Robot Structural Analysis Professional [12], we analyzed the performance of the casing structure during its sudden uncontrolled slip (fall) to the bottom of an open soil cavity from a height of 1.3 - 1.5 m at angles of deviation from the vertical axis of 0.5°-5°.
In developing the calculation model (Figure 9) it was taken into account that the structure of the shell consists of two cylinders stacked on each other: Upper cylinder: outer radius R = 36 m, inner radius R = 30.5 m, height H1 = 46 m; Lower cylinder: outer radius R = 36 m, inner radius R = 30 m, height H2 = 25 m. Thus, the outer diameter of the shell was D = 72 m, the height of the shell was H = 71 m. Concrete class B30.
To simulate the magnitude of the impact force at failure of the shell in the model, the cylinder fell from a height H = 150-250cm. under the action

of its own weight with an angle of inclination of 0.5 °-5 ° on the compliant soil (clay greenish-gray: φ = 21 °, C = 0,04 MPa, E = 19 MPa). Spatial shell design scheme was modeled: weight G = 210000 t; number of knots 16944; volume finite elements 12496; number of static degrees of freedom: 50828; number of loads 27; free fall acceleration g = 9.81 m/kV.s; fall time t=√(2*H/g); Δt=0.30-0.54 sec. Because of the angle of the slope, the frictional forces were applied at the top of the well on one side and at the bottom on the opposite side.

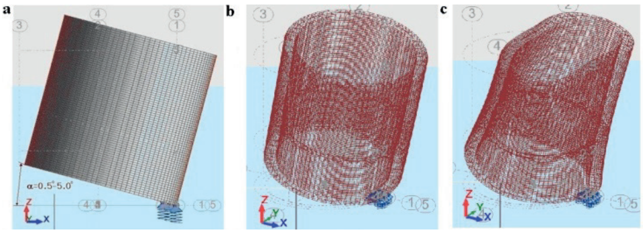


Figure 9. Schematics of calculation models and submersible shell simulation results at different angles of its deviation from the vertical axis: a - static support at roll; b,c - failure and slippage at deviation from the vertical axis (roll), respectively: calculation form "N4" at a=1°; ΔH=1.25m (n=0.56, Δ=26.1cm-limited VAT), calculation form "N22" at a=3.5°; ΔH=2.5m (n=1.94, Δ=183. 4cm-limited SSS)

During the analysis of the shell's deflectivity, we used the coefficient of forbidden state n, defined as the ratio of the equivalent stress of the shell structure according to Mises to the ultimate resistance of concrete of class B30.
Since the simulation of the stalling processes at different deflection angles of the shell from the -ɑ-axis and the drop heights -ΔH was performed in a rather large range, Figure 9 shows only fragments of the calculated forms "NN4 and 22" and the most characteristic results that were taken for analysis. The total calculation table of the integration results of the motion equations of the shell at stall (fall) at velocity VZ, VX, VY (cm/sec), acceleration AZ, AX, AY (cm/sec2) and displacement UZ, UX, UY (cm) was 186385 lines.

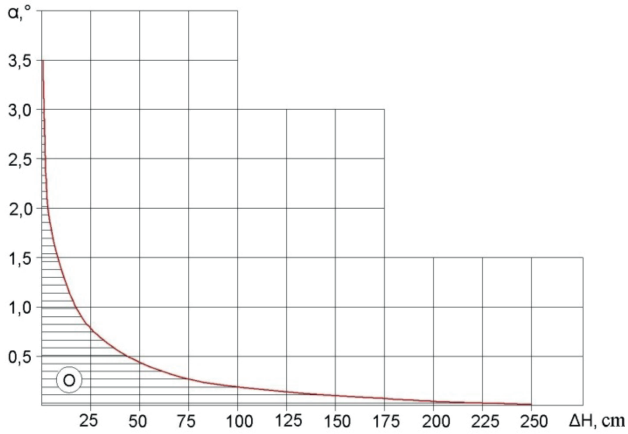


Figure 10. Area of maximum permissible values of conditionally instantaneous landings (failures) ΔH of the shell with diameter D=61m, height H=71m, weight G=210000t, at various angles of deviation of the structure from the vertical axis a° (concrete class B30; φ = 21°, C = 0,04 MPa, E = 19 MPa)

According to the results of modeling (Figure 10), the acceptable parameters of the spatial position of the shell and the range of its conditionally instantaneous disruptions, providing up to the limit value of the shell's SSS were established.
The simulation results show that for large-sized shells, the recommendations of normative documents have limited application and need to be confirmed by computational modeling.

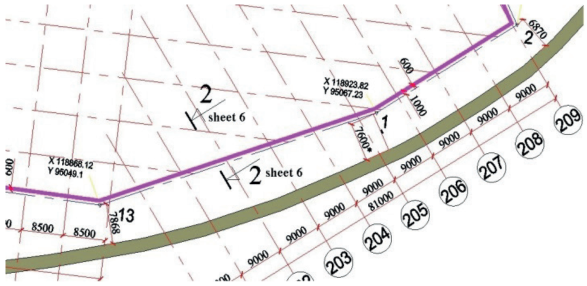
4. MODELING AND MONITORING OF POTENTIALLY DANGEROUS PARTS OF SEWAGE TUNNELS

4.1 The methods of studying potentially dangerous parts of sewage tunnels

The most interesting from the point of view of studying potentially dangerous sections of sewer tunnels is the system and network of tunnel collectors of St. Petersburg, which, with an undeveloped redundancy scheme, has a length of about 275 km. The system of sewer tunnels consists of pipelines with a diameter of 1.2 to 5.6 m and a depth of 8 to 70 m. Most (up to 75%) of the waste line length is located in the central historical part of the city in extremely difficult technogenic and engineering-geological conditions. The main part of the territory is covered by a stratum of Quaternary deposits (Q) that are unstable to technogenic impacts. Among the latter, it should be especially noted water-saturated clay soils belonging to lake-sea, lake-glacial and moraine deposits. Up to a depth of 30-120 m, soil strata are represented by silty sands of medium density, water-saturated E=7-11 MPa, C=0-0.005MPa, j=27-30°; silty plastic sandy loam E=3-5 MPa, C=0.01-0.02 MPa, j=12-17°; silty layered fluid plastic loams E=5-8 MPa, C=0.015-0.025 MPa, j=10-16° [13]. Long-term waste line operation in these conditions negatively affects their technical condition.

This factor is especially true for the continuously operating tunnel sewer collectors under the conditions of increasing man-induced impacts, first of all, to static from the large-sized complexes under construction in the influence zone with a developed underground part (see Figure. 11) and vibro-dynamic from the construction and transport equipment [14]. To identify the potentially dangerous areas, instrumental surveys of tunnels are carried out using a special technique. Technical instrumental surveys included: full-scale tachymetric survey of the spatial position of the tunnel in the intervals between mines, scanning the inner surface conditions of the tunnel with an assessment of its continuity with a GPR; taking cores and carrying out tests using the pull-off method with scanning to determine the strength characteristics of concrete, taking samples for chemical and biochemical analyzes, assessing the degree of corrosion and reinforcement by non-destructive methods, vibro-dynamic testing of vibrations of internal tunnel structures from external and construction vibration effects [15].

The uniqueness of the observation materials for the tunnels' state lies in the fact that the technical inspection of the structures has been carried out for a long time from the end of the 70s up to the present (2021). During this period, the same reservoir intervals have been surveyed for several times. At the same time, as a rule, after the examination, their conditions were monitored for several years. Thus, it became possible to trace the dynamics of the defects' development.



<u>Figure 11.</u> Scheme of a potentially dangerous section of a sewer tunnel in the zone with man-induced impacts from a complex under construction in St. Petersburg

The observation time range was divided into 3 periods:

a) 70-80s; b) 80-2000s; c) 2000-2020s. The most typical revealed defects affecting the operational reliability and bearing capacity of the tunnel were grouped into 7 classes: d1 - shrinkage cracks in the concrete jacket; d2 - signs of gas corrosion; d3 - drip leaks; d4 - force cracks in the arch and on the lateral surface of the tunnel; d5 - signs of biological corrosion of concrete; d6 - reinforcement corrosion, tray abrasion; d7- the presence of pressure leaks.

Analysis of the defects' development manifestation and dynamics structure show that in the initial period of the tunnel collectors' operation, defects were observed in the form of shrinkage cracks in the concrete jacket with the manifestation of drip leaks and signs of gas corrosion. The nature of the defects prevailing in the first 15-20 years of the tunnels' operation and their influence on the bearing capacity and operational reliability of structures can be taken as insignificant, and their technological state can be recognized as workable according to the RF "GOST" and "BC" regulations.

Defect-free waste line functioning in these conditions requires a calculated justification of the structural safety of the tunnel and monitoring its technical condition when choosing a method and a mode of carrying out the measures to restore the bearing capacity and operational reliability of the structure.

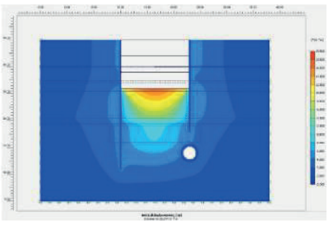
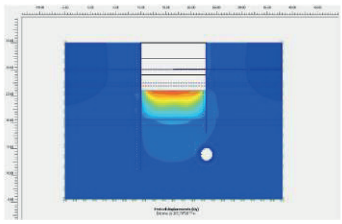
Within the framework of this study, we faced the task of the safe level external anthropogenic impacts' geotechnical provision on the tunnel structure, taking into account its residual bearing capacity.

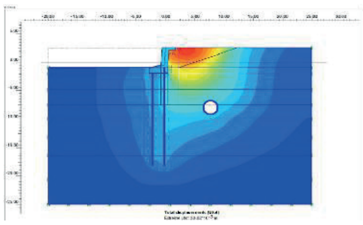
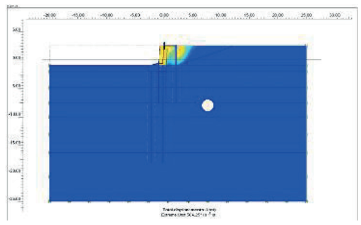
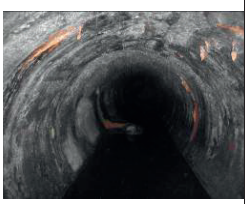
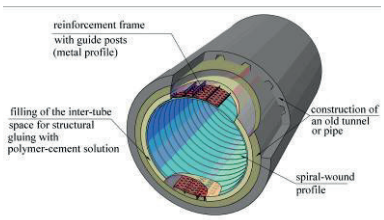
4.2 Modeling, monitoring and geotechnical substantiation of protective measures for potentially dangerous parts of deep sewage tunnel

The measures to protect potentially dangerous sections of tunnel collectors and ensure their reliability and structural safety were proposed on the basis of modeling and determining the boundary of the defect-free joint operation of the system "source of impact – geo-mass - sewer tunnel", but the main requirement that they must certainly meet, is the possibility of preventive use, justified by geotechnical and design calculations.

The results of geotechnical modeling to ensure the waste line structural safety typical for a large city with a developed engineering and transport infrastructure under difficult engineering and geological conditions of construction and operation are presented below (Table 1) as one of the examples [16].

Table 1. The results of the calculation substantiation of geotechnical measures to protect potentially dangerous sections of tunnels from unacceptable impacts

<table>
<tr><th>Characte of technogenic impacts</th><th colspan="3">Geotechnical and structural measures</th></tr>
<tr><td colspan="4">Arrangement of a protective screen made of fixed soil to prevent the foundation pit bottom from elevation</td></tr>
<tr><td rowspan="2">Unloading the soil mass when trenching the excavation for the tunnel</td><td>Vertical deformations of the soil mass after the excavation of the construction pit, without preliminary fixing - 37mm</td><td colspan="2">Vertical deformations of the soil mass after excavation of a construction pit with soil fixing above the collector using Jet Grouting technology (3.0 m thick) - 3.2 mm</td></tr>
<tr><td></td><td colspan="2"></td></tr>
<tr><td colspan="2">Calculation option</td><td>Construction stage</td><td>Collector deformation, mm.</td></tr>
<tr><td colspan="2">Without Propping</td><td>-</td><td>+37</td></tr>
<tr><td colspan="2" rowspan="2">Propping the foundation pit using Jet Grouting technology. Power 2.0m.</td><td>soil reinforcement</td><td>-2.1</td></tr>
<tr><td>excavation</td><td>+10.1</td></tr>
</table>

<table>
<tr><td colspan="3" rowspan="2">Propping the foundation pit using Jet Grouting technology. Power 3.0m.</td><td>soil reinforcement</td><td>-3.0</td></tr>
<tr><td>excavation</td><td>+3.6</td></tr>
<tr><td colspan="3" rowspan="2">Propping the soils above the collector using Jet Grouting technology. Power 2.0m.</td><td>soil reinforcement</td><td>-2.6</td></tr>
<tr><td>excavation</td><td>+8.3</td></tr>
<tr><td colspan="3" rowspan="2">Propping the soils above the collector using Jet Grouting technology. Power 3.0m.</td><td>soil reinforcement</td><td>-2.9</td></tr>
<tr><td>excavation</td><td>+3.2</td></tr>
<tr><td colspan="5">Structural waste line reinforcement winding technology to increase the maximum permissible tunnel displacements</td></tr>
<tr><td rowspan="4">Increasing the zone of maximum permissible waste line displacements with one-sided unloading of the soil mass during the embankment reconstruction.</td><td colspan="2">General calculated displacements during excavation under the protection of sheet piling</td><td colspan="2">General calculated displacements during excavation with the device of a geotechnical barrier</td></tr>
<tr><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td colspan="2">Additional horizontal deformations of the collector were 54mm</td><td colspan="2">Additional horizontal deformations of the collector were 7mm</td></tr>
<tr><td colspan="4"></td></tr>
<tr><td colspan="5">Structural waste line reinforcement by lining and spiral-wound technology to restore the bearing capacity of the tunnel to the design level</td></tr>
<tr><td rowspan="2">Increase in static and dynamic loads on waste line from the action of heavy vehicles and trams</td><td></td><td colspan="2"></td><td></td></tr>
<tr><td>Condition of a potentially dangerous waste line area before renovation</td><td colspan="2">Structural scheme and solution to strengthen the tunnel section</td><td>Technical condition of the tunnel after restoration and repair</td></tr>
</table>

The second example of the numerical implementation of measures to protect the collector from external influences is the potentially hazardous area noted above (see Figure 11).

The customer set the task to ensure the safe operation of a sewer collector located in soft soils, near which the construction of a high-rise building had started.

The modeling task was to determine the permissible horizontal displacements of the tunnel sections when performing work near the structure.

For normal operation of the collector tunnel, it is necessary to exclude the formation of cracks in the structure of the lining caused by its displacement towards the pit during the work.

The criterion for the structure safety is the maximum permissible tensile stresses of concrete at the characteristic points of the lining.

The design of the collector tunnel lining is a two-layer cylinder. The outer layer is a prefabricated reinforced concrete structure made of tubing. The inner layer is a monolithic reinforced concrete jacket (see Figure 12).

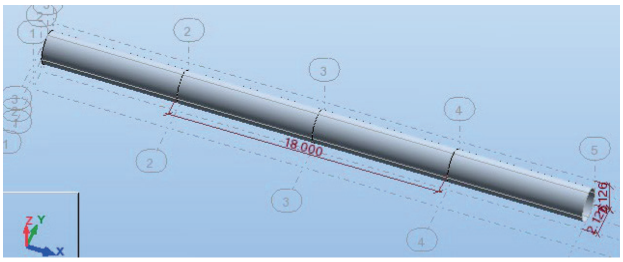


Figure 12. Design of the collector tunnel potentially dangerous section's lining

The tunnel sections with different lengths of the influence zone and the structure reinforcement degree were modeled (see Figure 13). Geotechnical calculations simulated the measures to reduce the impact on the displacement of the tunnel using a wall in the ground between the tunnel and the wall under construction in the ground made of low-modulus material.

a)

b)

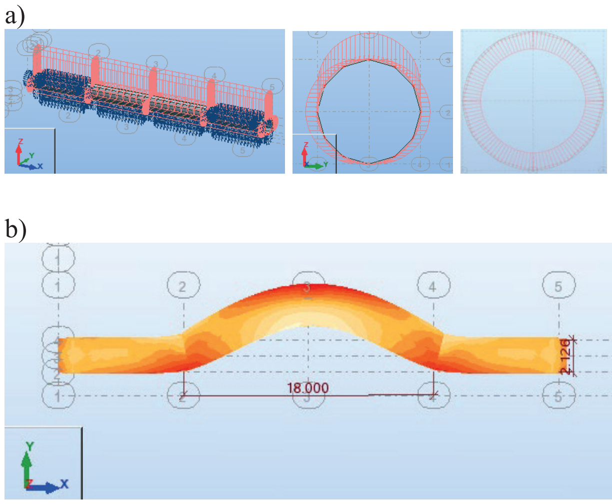
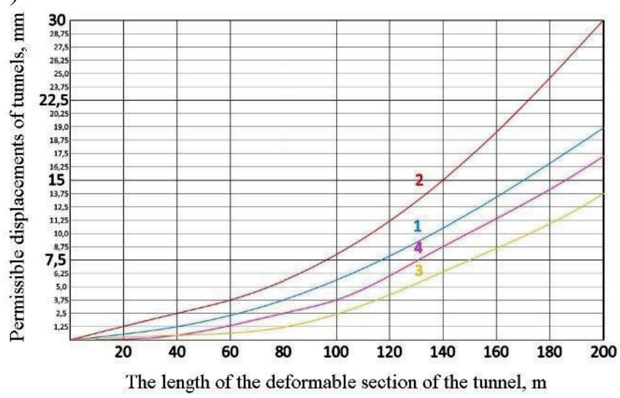


Figure 13. Fragment of the calculated potentially dangerous section of the collector with a length of 18 m: a) with loads and elastic rebound; b) the transcendent SSS tunnel zones: 1 - in the middle; 2 - at the ends of the displacement section

Numerical calculations were used to obtain the permissible displacement values of the lining, taking into account the presence of a screen made of low modulus material and depending on the tunnel deformable section length. Carrying out of work related to man-induced impacts, the project provided monitoring the tunnel and geo-massif structure [17].

The calculation substantiation of geotechnical protective measures was carried out according to the algorithm: collection of loads and impacts, determination of physical and mechanical characteristics, determination by geotechnical calculations of the permissible level of external anthropogenic impacts on the tunnel, taking into account its residual bearing capacity. Figure 14.a shows fragments of the tunnels' maximum permissible displacements computational modeling results (before the application of protective measures and after the implementation of protective measures) and Figure 14.b shows the data of monitoring observations.

a)



b)

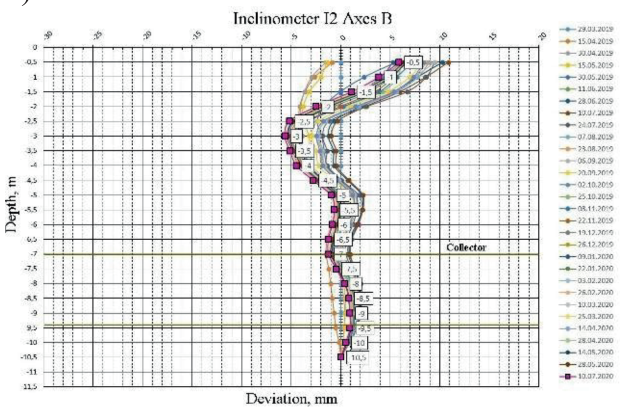


Figure 14. Maximum permissible displacement of a tunnel falling into the zone of man-induced influences: a) calculated values of the maximum permissible displacement of sewer tunnels D=1.5 and D=2.5 before (1.3) and after (2.4) strengthening the structure using winding technology; b) data of monitoring control by the inclinometers readings to prevent exceeding the maximum permissible displacement of the tunnel D=2.5m with the length of the deformable section 70m

To ensure the bearing capacity of the tunnel, the SATURN winding method was used, developed and adapted for the specific conditions of St. Petersburg: intervals between mines up to 1000m and more; irregularity of the working section along the length of the collector associated with the dynamic influences and subsidence of the tunnel in weak thixotropic soils. The sewage tunnel with inter-shaft spacing up to 850m and diameters D= 2,5 m and D= 1,5 m is embedded at the depth down to 17 m, it has been operating for more than 40 years and, according to the survey results, had a wear rate of more than 79%, subsidence at the intersection of streets up to 25 mm. Based on the GPR scanning results, it was found everywhere that the concrete jacket was peeled off from the tubing lining with the formation of pressure leaks. The scope of work operations included: tunnel cleaning and surface preparation; structural bonding of the concrete jacket and tubing lining by injecting SikaDur; reinforcement of the surface of the vault with structural reinforcement with SikaWrap carbon fiber; lining the surface of the tunnel with a winding profile made of PVC; polymer cement mortar injection (P_{comp}=65MPa) into the annular space for structural bonding of the shell made of PVC with tunnel construction.

This method was applied to make a geotechnical prediction of dangerous parts of tunnels long-operating in difficult soil conditions (see figure 15), the numerical modeling defined a rational method of geotechnical protection of these sections in order to provide their structural safety at growing anthropogenic and dynamic actions.

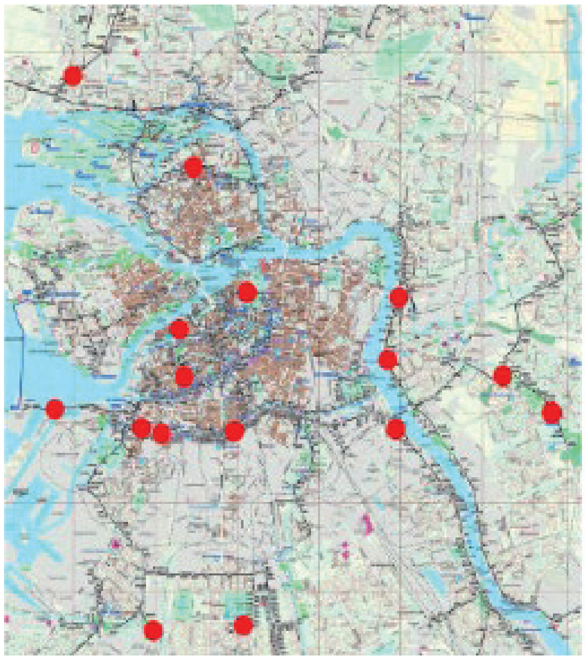


Figure 15. Geotechnical prediction of potentially dangerous sections of sewage tunnels in St. Petersburg which require protection of structural safety

According to the monitoring carried out on the restored potentially dangerous sections of the tunnel, it was established: vibration dynamic tests of the tunnel before and after repair showed the changes in the period of natural collector vibrations of 0.54 s. up to 0.19 s. so, by 58%, and the amplitude of natural vibrations decreased from A = 300 μm to A = 15 μm, i.e., by almost double. This indicates the structure integrity restoration and the joint work of its layers during continuous operation.

5. CONCLUSIONS

Difficult engineering and geological conditions and the increasing influence of technogenic factors have led to the wear of long-operated tunnel collectors in large cities of Russia up to 66%. For St. Petersburg, characterized by vibration-resistant enclosing waste line massifs of soils, the level of wear reaches 83% with a high dynamic of development up to 1.5 hours 2% per year.

Potentially dangerous sections of the tunnels have been identified by the methods of diagnostics and modeling of technical conditions

Proposed and geotechnically sound waste line protection methods, including technologies of structural reinforcement and rehabilitation in the conditions of wastewater transportation, accompanied by a monitoring system, ensure structural waste line safety and their operational reliability.

Methods for ensuring structural safety developed and substantiated by waste line geotechnical modeling are recommended for use in large cities with difficult soil conditions, with heavily worn-out sewers with potentially dangerous sections of tunnels and, as a result, to increase their reliability during long-term operation in conditions of urban infrastructure development.

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Nicolay A. Perminov, C.T.Sc., Chief specialist of the Institute SUE "Lengiproingproekt", docent «Emperor Alexander I St. Petersburg State Transport University», 9, Moskovsky prospect, St. Petersburg, 190031, Russia; phone +7 (812) 315-26-21; fax: +7 (812) 315-26-21; E-mail: perminov-n@mail.ru.

Mangushev Rashid Abdullovich, Corresponding Member of the Russian Academy of Architecture and Construction Sciences, Professor, Doctor of Technical Sciences; Head of Geotechnical Department, Saint-Petersburg State Architecture and Construction University, Russia, Saint-Petersburg, 2-ya Krasnoarmeyskaya 4; E-mail: ramangushev@yandex.ru.

Перминов Николай Алексеевич, кандидат технических наук, главный специалист института ГУП «Ленгипроинжпроект», доцент «Петербургского государственного университета путей сообщения Императора Александра I»; 190031, Россия, г. Санкт-Петербург, пр. Московский, д. 9, тел. +7 (812) 315-26-21; факс +7 (812) 315-26-21; E-mail: perminov-n@mail.ru.

Мангушев Рашид Абдуллович, член-корреспондент РААСН, профессор, доктор технических наук; заведующий кафедрой геотехники Санкт-Петербургского государственного архитектурно-строительного университета (СПбГАСУ), Россия, Санкт-Петербург, 2-я Красноармейская 4; E-mail: ramangushev@yandex.ru.