Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-1352

Field and laboratory study of deviatoric creep of overconsolidated clayey soils

Rafael Sharafutdinov
Gersevanov Research Institute of bases and underground structures (NIIOSP), Research Center of Construction JSC Russian Federation; linegeo@mail.ru

ABSTRACT: The results of laboratory and field studies of deviatoric creep of overconsolidated clayey soils are presented. Laboratory studies were carried out in axisymmetric triaxial test apparatus; field studies involved load tests on large-scale bored piles. The influence of bored pile installation technology on deviatoric creep parameters during bored pile-soil interaction has been established. The rate of deviatoric creep of piles installed under bentonite slurry protection was 40% higher than that of piles installed using dry technology. It was found that the deviatoric creep index for pile-soil interaction could be determined from laboratory triaxial tests by introducing a coefficient of strength reduction at the "structure-soil" interface. Recommendations on accounting for deviatoric creep when performing analytical and numerical calculations were proposed.

KEYWORDS: Laboratory test; In-situ test; Creep; Deviatoric creep; Numerical analysis.

1 INTRODUCTION

Deviatoric creep essentially affects a long-term bearing capacity of foundation and clayey soil. Either volumetric or deviatoric creep has an effect on foundation displacement and inclination of the structure (Meschyan, 1995).

The volumetric creep parameters are defined through 1-D oedometer testing (ASTM D2435, 2020). Thoroughly studied volumetric creep does not cause problems in geotechnical engineering.

Deviatoric creep parameters can be defined from the laboratory direct and ring shear tests (Meschyan, 1995). However, these tests have some limitations interfering with unambiguous interpretation of the test results and their further application for overconsolidated soils. These limitations potentially include impossibility of reconsolidation, discrepancy between predicted load path and failure mode, and limitations occurring in natural soil conditions, as well as design factors (friction of samples against the walls of the device, stress concentrations in corners), etc. Mirsayapov and Koroleva (2011), He et al (2020) showed that the use of triaxial apparatus (of axisymmetric or true triaxial type) was more feasible.

Another method for assessment of the deviatoric creep is a long-term pile load test. Despite the high cost and time investment, in-situ pile load test is more reliable and reporting for pile foundations (Sharafutdinov, 2025).

However, the significant dependence of the test results on the method of the pile installation should be taken into account.

The article embodies the results of the laboratory and large-scale field studies of deviatoric creep of overconsolidated clays. The laboratory tests were performed using axisymmetric triaxial testing devices. The field investigations involved pile load tests of long duration. The influence of bored pile installation technology on deviatoric creep parameters of the soil interacting with piles was established. The rate of deviatoric creep of piles installed under bentonite slurry protection was 40% higher than of piles installed using dry technology. It was found that the deviatoric creep index of the pile-soil interaction system could be determined from the laboratory triaxial tests by introducing a strength reduction coefficient at the "structure-soil" interface. The application of the study results for analytical and numerical analysis was proposed.

2 METHODS

The laboratory study was carried out using axisymmetric triaxial test. In this study, stress-controlled consolidated drained (CD) triaxial compression was activated at various $\tau = (\sigma_1 - \sigma_3)/2$. To enhance the accuracy of the volumetric strain measurements ε_v, the local LVDT strain gauges were installed to specimens.

The laboratory study included two stages.

At the first stage, an optimum deviatoric creep equation for overconsolidated soil was found. In this regard, power-law, logarithmic, hyperbolic and exponential functions were considered. The shear stresses τ_i applied in the stress-controlled mode were 120, 180, 240 and 300 kPa and were held up to 26 days at each stage. The study considers both shear strain γ and axial strain ε_1. In most triaxial apparatus, shear strain γ is defined via volumetric strain ε_v, whose measurement is based on the volume of pore water squeezed out of the specimen through the system of back pressure tubes, and, thus, is prone to inaccuracies caused by the design features of the apparatus.

At the second stage, the effect of stress state on the deviatoric creep parameters of overconsolidated soils of Moscow and Saint Petersburg was studied. Characteristics of the soils are given in Table 1.

The results of the long-term pile and barrette load tests carried out on two experimental sites in Saint Petersburg (Shulyatyev, 2020) were analyzed. Two piles with 2 m in diameter and two barrettes with the area of 2,8 x 1,4 m were subjected to O-cell testing. Piles and barrettes (hereinafter piles) were divided into ≈2…3 m long lower section and ≈20…25 m long middle and upper sections. Piles were installed using dry drilling, barrettes were constructed under slurry support.

In the case of the large-scale field tests, the deviatoric creep was studied for Vendian deposits (V2kt2) represented by stiff clay (SPb-01 and SPb-02) and stiff clay with sandstone interlayers (SPb-03), see Table 2.

The load was applied in steps and was held on for 21 days at each step. This essentially exceeded the recommended conventional stabilization of 0.1 mm/h (The load is applied in 73 steps ranging from 6 to 72 MN, thereby, the average skin friction of the pile segments is in the range of τ = 0,06...2,54 MPa).

3 RESULTS

Figure 1 shows the typical shear γ and axial ε_1 strain of the stiff clay under long-term triaxial loading, plotted versus time. It was observed that the power, logarithmic and exponential functions had the best adequacy. However, the exponential function showed a rapidly damped character. A significant discrepancy between power and logarithmic functions was observed in the beginning of the creep curves. The hyperbolic function scarcely suited the description of the creep behavior of the overconsolidated soils, and the large values of the coefficient of variations confirmed this.

Table 1. Soil properties.

Site	Type of soil	Age	Depth, m	*Void ratio* e_i	*Plasticity index* I_p	*Liquidity index* I_L,	*Overconsolidation ratio* *OCR*	*Preconsolidation pressure* σ'_{vc}, MPa	*Consolidation pressure* σ'_c MPa
M-173876	C-S	Q	35,5	0,62	0,19	0,18	3,7	0,8	0,44
M-173640	C-VS		9,0	0,51	0,22	-0,02	5,1	0,7	0,44
SPb-01	SC-VS	V	25,0-91,7	0,41-0,60	0,12-0,161	-0,45-(-0,08)	3,7	0,4-2,9	0,25-1,7
SPb-02	SC-VS			0,38-0,44	0,097-0,122	-0,87-(-0,19)	2,7	0,11-1,5	0,35-1,9
SPb-03	SC-VS			0,32-0,5	0,089-0,15	-0,7- (-0,46)	1,7	1,4-2,1	0,7-2,3
M RCC-1	SC-S	Q	8	0,47	0,99	0,24	1,2	0,4	0,109
M RCC-1	SC-FS		9	0,80	0,12	0,34	1,2	0,65	0,137
M RCC-3	SC-FS		10	0,48	0,10	0,26	1,3	0,16	0,096

Note: Q – Quaternary period V - Vendian Period; SC-VS — very stiff silty clay; SC-S — stiff silty clay; SC-FS — firm-stiff silty clay ; C-S — stiff clay; C-VS — very stiff clay

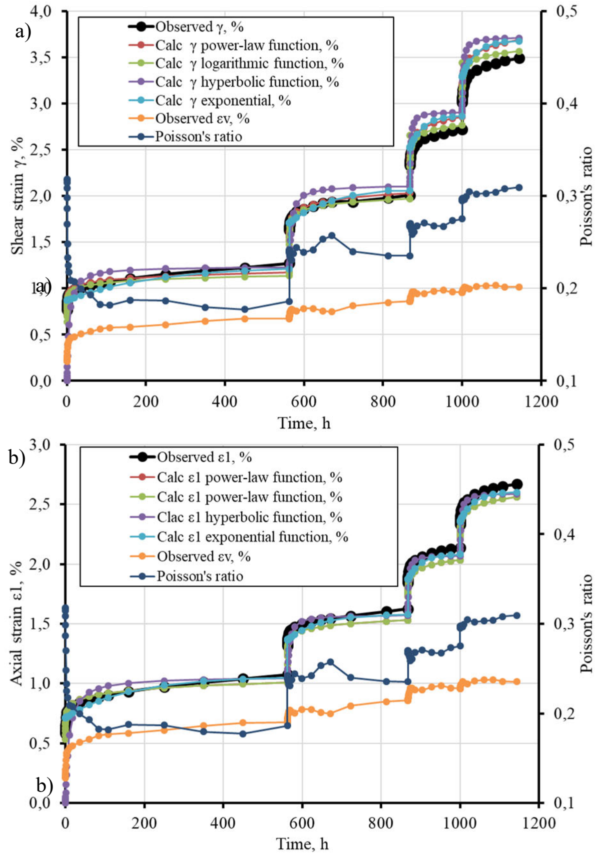

Figure 1. Shear (a) and axial (b) strain versus time.

The test results showed that Poison's ratio ν of overconsolidated clays remained nearly invariable (varied less than 5%) at creeping. The particular and mean values of the creep parameters processed using shear γ and axial strain ε_1 were nearly close, what was consistent with their physical nature.

Given the invariability of ν in creeping, it is recommended to describe the deviatoric creep with power function as universal and conservative one. The parameters of the deviatoric creep under triaxial compression should be defined via axial strain ε_1:

$$\varepsilon_{1t} = \varepsilon_{10}\left(\frac{t_q}{t_0}\right)^{\alpha_1} \quad (1)$$

where, α_1 – deviatoric creep index obtained on the base of triaxial test data; ε_{10} – relatively instantaneous strain within the time of relative stabilization t_0, at the very least 24 h; t_q – duration of the load action.

Rather strong dependence of α_1 on stress state was received (fig. 2a). Large values of α_1 were observed at $\tau < 150$ kPa. This was due to the compression of the insufficiently compacted irregular end surfaces of the specimens at the reconsolidation stage and due to the "healing" of cracks and defects of poorly collected specimens. At the subsequent loading stages, α_1 increased with increasing τ_i through to ultimate load application. Additionally, a stable relationship between α_1 and void ratio e was established.

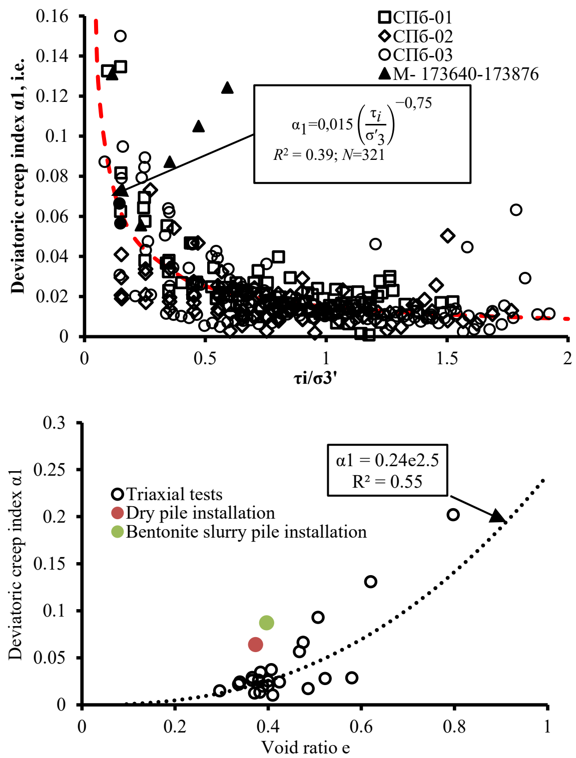

Figure 2. Deviatoric creep index α_1 versus τ_i/σ'_3 (a); void ratio e (b).

The relationship similar to those assumed above was used to analyze the deviatoric creep:

$$S_t = S_0\left(\frac{t_q}{t_0}\right)^{\alpha_p} \quad (2)$$

where, α_p –; deviatoric creep index obtained on the base of pile loading test data S_0 – relatively instantaneous pile settlement in the time of relative stabilization t_0 according to GOST 5686, at the very least 24 h.

The pile installation technology (dry or under slurry support) has a significant effect on the pile behavior under the load (fig. 3). In dry drilling technology, the mean value of α_p is 43% lower when compared with the excavation under slurry support. For piles installed dry in stiff Vendian clay, the received $\alpha_p = 0.039$ exceeded laboratory test result of $\alpha_1 = 0.025$ by 1,5 times. The creeping index α_p at the pile-soil interface can be defined as:

$$\alpha_p = \alpha_1 / R_{\text{int}}^2 \tag{3}$$

where, R_{int} is a coefficient of stiffness reduction at the contact “structure-soil”.

The value of R_{int} might be defined from the laboratory and field tests results.

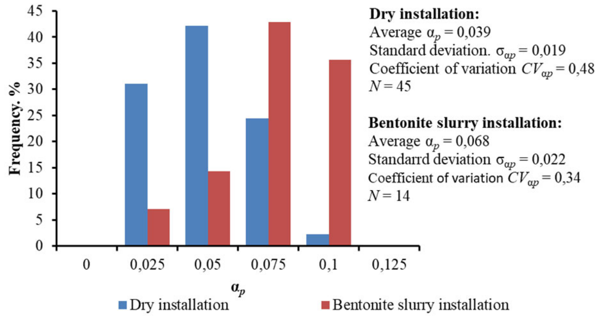

Figure 3. A diagram of α_p distribution in relation on pile installation method.

4 NUMERICAL ANALYSIS OF DEVIATORIC CREEP

Numerical analysis is an advanced tool to calculate long-term settlement. An accurate combined assessment of both volumetric and deviatoric soil creep is necessary, when using this approach. However, far from all soil models implement this. In this case, an approach utilizing a time-varying soil stiffness can be applied.

Numerical analysis may use both a well-known viscous-elastic model and hardening soil models.

When using elastic and Mohr-Coulomb model, the applied approach can utilize a time-varying deformation modulus.

The volumetric creep is customary to describe with logarithmic function. Changes in deformation modulus over time can be expressed as follows:

$$E(t,t_0) = \frac{E(t_0)}{1 + \frac{c_\alpha}{\varepsilon_1(t_0)} \lg\left(\frac{t}{t_0}\right)} \tag{4}$$

where, c_α is a coefficient of the secondary consolidation defined with compression apparatus in accordance with GOST 12248.4; $\varepsilon_1(t_0)$ is a strain at the time of conventional completion of the filtration consolidation t_0.

On the ground that the Poisson’s ratio is constant at creeping, a bulk compression modulus can be defined as:

$$K(t,t_0) = \frac{E(t_0)}{\left[1 + \frac{c_\alpha}{\varepsilon_1(t_0)} \lg\left(\frac{t}{t_0}\right)\right](1-2\nu)} \tag{5}$$

Deviatoric creep can be regarded as:

$$G(t,t_0) = G(t_0)\left(\frac{t}{t_0}\right)^{-\alpha_1} \tag{6}$$

Soft Soil Creep model (hereinafter SSC) can be used to assess creeping, however, it indirectly takes into account deviatoric creep.

The approach considering the variable deformation modulus in the Hardening Soil can be used through the following:

$$E_{50}(t,t_0) = E_{50}^{ref}(t_0)\left(\frac{t}{t_0}\right)^{-\alpha_1} \tag{7}$$

$$E_{oed}(t,t_0) = E_{oed}^{ref}(t_0)\left[1 + \frac{c_\alpha}{\varepsilon_1(t_0)} \lg\left(\frac{t}{t_0}\right)\right]^{-1} \tag{8}$$

The above relationships may have an application for analytical and numerical analysis.

The applicability of enumerated models implemented with PLAXIS software was assessed by laboratory tests on volumetric and deviatoric creep (fig. 4).

It was found, that volumetric creep assessed with HSS and SSC models showed the results closed to test results. The deviatoric creep in the HSS model assessed with respect to (7)-(8) well agreed with experimental evidence.

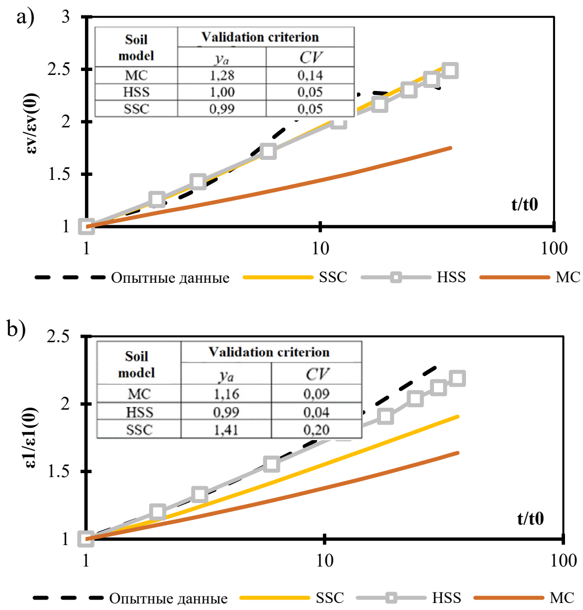

Figure 4. Typical results of the back analysis of the triaxial test on deviatoric creep: a relative volumetric (a) and axial (b) deformation.

5 DISCUSSION AND PRACTICAL APPLICATION

In practice, it is advisable to assess a decrease in stiffness over time.

The solutions proposed were used to modify the equation of the long-term settlement of a single pile (Sharafutdinov et al, 2024):

$$S(t,t_0) = \frac{N\tau_f(t,t_0)}{PG_i(t,t_0)\left(\tau_f(t,t_0) - \frac{N}{UL}R_f\right)} \tag{9}$$

where, U is a pile perimeter; $G_i(t,t_0)$ is a stiffness of the“pile-soil” system; $\tau_f = R_{\text{int}}\left[K_0\sigma_g \tan\varphi(t,t_0) + c(t,t_0)\right]$.

The proposed approaches can be validated using the data of the long-term observation of the settlement. Since the deviatoric creep most typically accompanies soil-pile interaction, the validation was performed regarding pile foundation. The shortage of the full-value data of the long-term observation complicated the validation.

Settlement of the single pile. Single piles and barrettes described in the section 3 and interacting with Vendian clay were used in validation. The validation results of the proposed approach (9), performed on the base of both pile and barrette loading tests data, demonstrated a sufficient engineering accuracy of the developed methods (fig. 5).

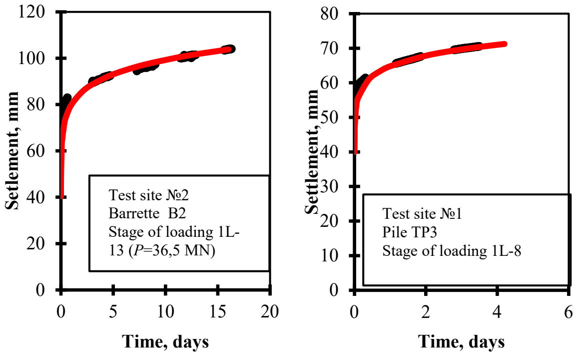

Figure 5. A comparison of the calculation results with monitoring data: red – calculation results, black – observation data.

Settlement of the small group of piles. The validation was performed using the data received from the monitoring of the frame building settlement, described by Bartolomey et al, 1994.

The piles of 6 m long, with an area of 25x25 cm, were installed in a clayey soils in groups with a spacing of 3d. The maximum load applied to a pile group was 580 kN. The accommodating clayey soil had the following parameters: plasticity index of I_L = 0.55-0.75; void ratio e =0.68-0.85; deformation modulus E = 6.7 MPa; angle of internal friction φ = 16-18°; specific cohesion c =2-3 kPa. The index of deviatoric creep was 0.05-0.07, according to the data of the section 3.

The data of 32-year-long observation (1968-2000) showed the average increase in foundation settlement ranged from 14.2 to 18.5 mm (by 1.3 times) after application of 100% load. The settlement mode and increase over time was comparable with the settlement of the pile groups discussed in this paper.

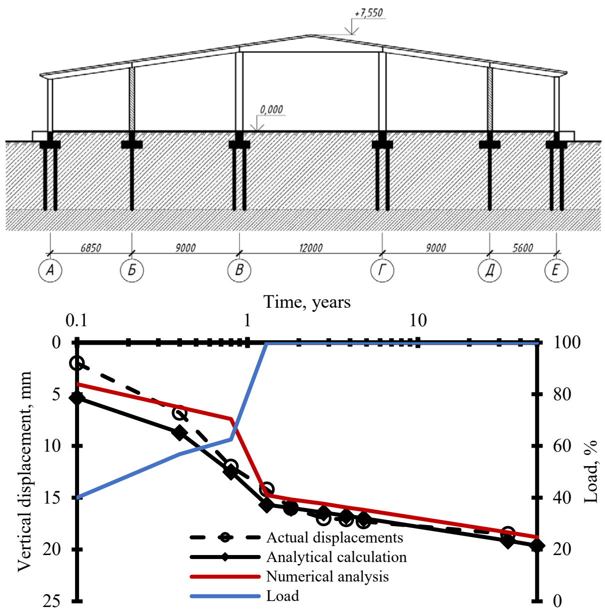

Figure 6. Designed and actual settlement of pile groups over time.

The results of calculations performed according to analytical approach showed that at the time of full load application, the conditionally instantaneous settlement of the pile-group was 15.7 mm, which was by less than 10% at odds with monitoring data. According to numerical approach, the discrepancy was 4%. After 30 years' time of the building operation, the designed settlement was 18.4-19.2 mm, meanwhile the monitoring results showed 18.5 mm.

The values of settlement received from the developed approach and the monitoring data showed good agreement both at the time of 100% load application and while operating.

Settlement of the large pile group. Validation of the large pile group was based on the long-term monitoring of some structures.

Residential buildings of the Moscow region. The buildings were constructed in 2004-2005 year period (fig. 7). The settlement has been observed for 20 years, since the construction start-up.

The residential buildings are 16-storey (No 10) and 32-storey (No12B) prefabricated flat blocks. The buildings rest on reinforced concrete piles with the length of 14 m (No 10) and 5 m (No 12B), and with a cross section of 30x30 cm. The amount of piles is 1380 and 980 respectively. Foundation bottom pressure is 250-330 kPa. The piles interact with clayey and sandy soils (fig. 8, 9).

In 20 years' time after construction start-up, the designed settlement was 70-110 mm. The comparisons of the calculation and observation results (fig. 10) showed good agreement. The discrepancy of the calculated and actual settlement was less than 10%.

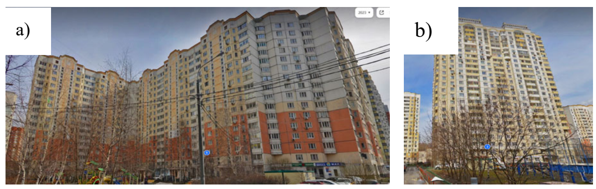

Figure 7. Photos of the residential buildings 10 (a) and 12B (b) in Moscow region.

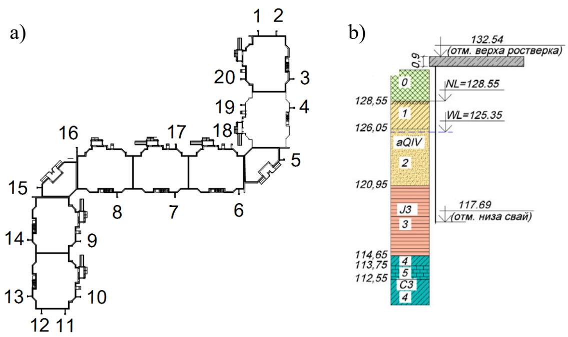

Figure 8. The residential building No. 10's scheme with geodetic marks (a) and subsoil profile (b).

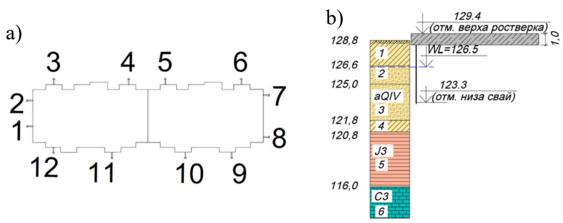

Figure 9. The residential building No. 12B's scheme with geodetic marks (a) and subsoil profile (b).

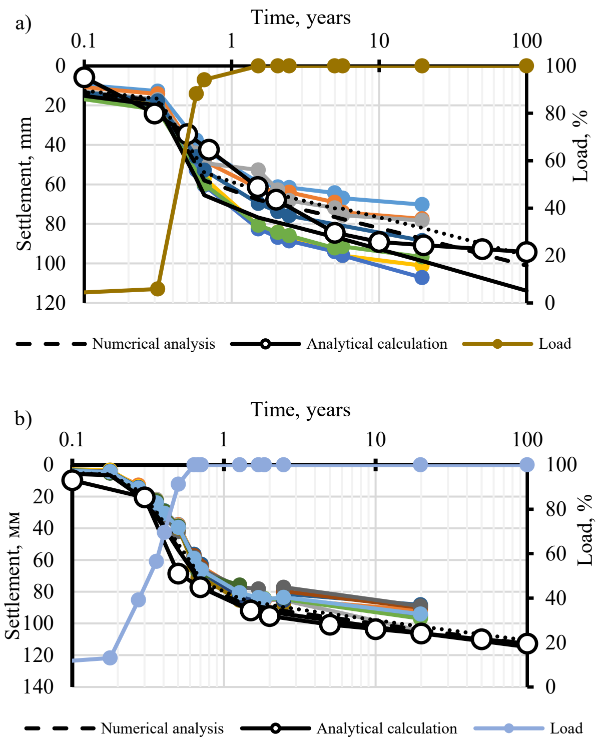

Figure 10. Comparison of the calculated and observed settlements of the residential building No 10 (a) and 12B (b)

The main pier of the Garigliano bridge in Southern Italy.

Mandollini, 2005 presented the results of the long-term (for 13 years) observation over the settlement of the foundation of the piers of the cable-braced bridge. The project comprised rectangular 10,6x19.0 slab resting on 144 steel tubular piles of 48 m long. An excavation supported by 12 m long bored piles of 0,8 m in diameter surrounded the piers. The construction of the bridge started in October 1991. Total load was applied in July 1994. The observation of the settlement was carried out from 1991 to 2004. The maximum load applied to the pier foundation was 118 MN. Figure 11 represents the scheme of the pier foundation of the bridge.

The piles are interacting with clayey soils with occasionally occurring sand layers. The piles were embedded to a depth of 50 m.

Analytical and numerical approaches to assess settlement were applied. The scheme of numerical approach is shown in figure 12.

The analysis results showed the discrepancy with the observation data was 20% in 13 years after load application, which can be considered satisfactory for engineering tasks (fig. 13)

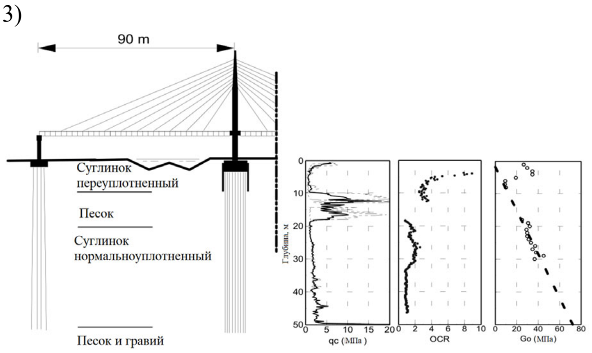

Figure 11. Subsoil profile at the location of the main pier of the Garigliano bridge (Mandolini et al, 2005).

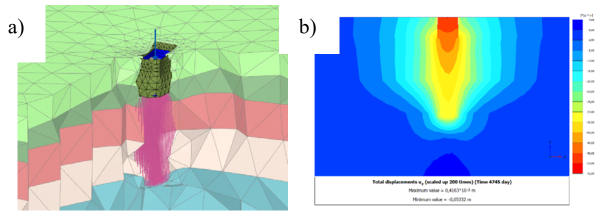

Figure 12. General view of the calculation scheme (a) and computed settlements in 13-year time after load application (b) to the foundation of the main pier of the Garigliano bridge in Southern Italy.

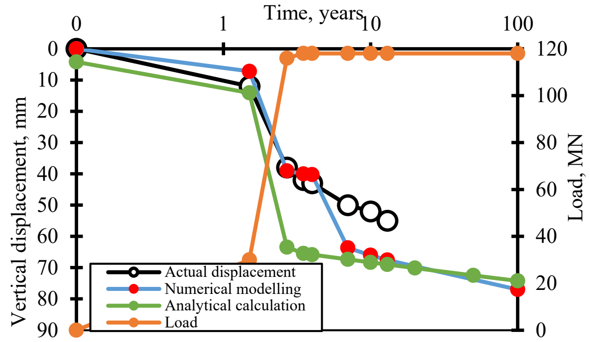

Figure 13. Estimated and actual foundation settlement of the main pier of the Garigliano bridge in Southern Italy (Mandolini et al, 2005).

High-rise building of the "Lakhta-Center", St. Petersburg.

The building has a frame-core structure. Basic bearing structures involve a 26-meter-diameter central concrete reinforced core, which carries 70% of the total vertical load, and 10 steel-concrete composite columns connected with a central core by four outrigger trusses, and a distributing slab (fig. 14). The total design weight of the building is 5010 MN.

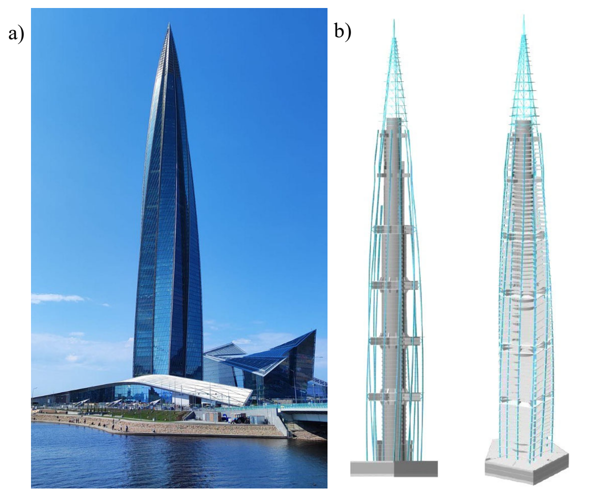

Figure 14. High-rise building of the "Lakhta-Center": a – general view; b – fundamental design solution of the superstructure (Shulyatyev O.A., 2020).

A box foundation comprises upper and lower monolithic concrete reinforced slabs with a thickness of 3.6 and 2.0 m respectively, and 2.5 m thick shear walls (Shulyatyev O.A., 2020). The foundation consists of 264 bored piles, with a diameter of 2 m and a length of 55 and 65 m, embedded from the depth of 72 and 82 meters respectively with a spacing ranging from 4 to 6 m. The different length of the piles bespeaks to reduce differential settlement of the box foundation. The piles are embedded into very stiff Vendian clay of the Kotlinsky age underlain by the sandstone of the same origin. Very stiff

Vendian clay overlapped with glaciolacustrine, marine and lacustrine deposits (fig. 14) (Sharafutdinov et al, 2024).

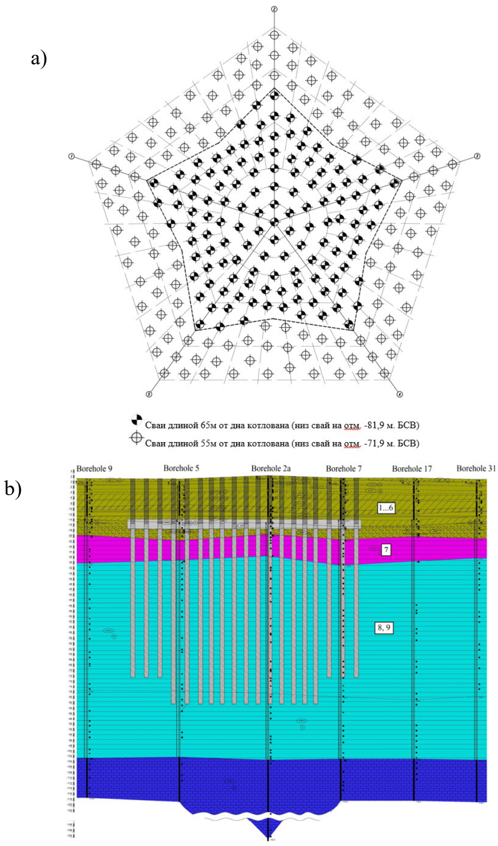

Figure 14. Layout of bored piles (a) and cross-section with the bored piles (b): XX - piles length of 65 m (the elevation of the pile end is 81.9 m) (Sharafutdinov et al, 2024)

The building is constructed from the 17 m deep foundation pit, which is supported by a 1.2 m thick and 31 m deep diaphragm wall. Temporary monolithic concrete slabs arranged in four levels brace the diaphragm wall.

The maximum calculated settlement and the mean settlement value of the lower slab of the box foundation observed over an 8-year period is 54.5 mm and 40.5 mm respectively, with the soil creep taken in consideration (fig. 15). According to field monitoring data, the maximum settlement is 49.7 mm. The difference between the results of back-analysis and the field monitoring data does not exceed 10%. Insignificant discrepancies relate to some design and technological features: assumptions and inaccuracies in the load readings and their modeling, non-linear behavior of the concrete structures of the box foundation and piles, soil anisotropy, incomplete consolidation and others.

The results of the analytical assessment and numerical modeling performed for the deep soil layers taking into account creep showed good agreement with the data of long-term field observations. The discrepancy with the data of long-term observations of high-rise building settlements is less than 10–20%.

6 CONCLUSIONS

The laboratory axisymmetric triaxial compression test is an appropriate approach to assess parameters of the deviatoric creep. However, the parameters defined via axial strain are more reliable and practical.

Both laboratory triaxial test and in-situ pile loading test show that the power function most appropriately describes deviatoric creep, and gives the more conservative assessment of the deformation over time.

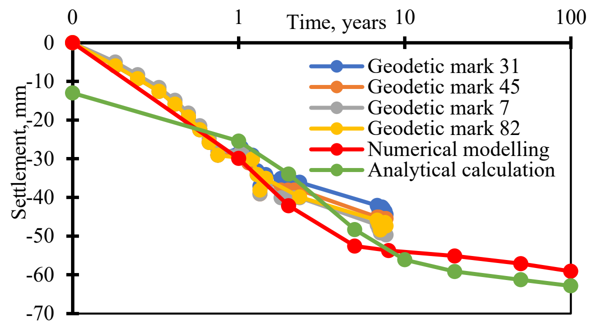

Figure 15. Estimated and actual foundation settlement of Lakhta-Center high-rise building pile foundation in Saint-Petersburg (Sharafutdinov et al., 2024).

The Poisson's ratio varies in overconsolidated clayey soils under additional load application. However, it remains almost invariable at creeping.

Overall, the index of the deviatoric creep for dry technology of pile construction is 40% as much as for the pile construction under the slurry support. However, a short pile-soil interaction (regardless creep) was defined as independent from construction technology. It was also discovered that the deviatoric creep index in relation to a pile-soil interaction could be defined from laboratory tests by introducing the coefficient of stiffness reduction at the contact "structure-soil".

The comparison with the results of durable load tests showed that the developed approach was applicable to predict the settlement of the pile foundation over time.

The results of analytical evaluation and numerical modelling, performed for deepen soil layers with taking into account creep showed good agreement for long-term field monitoring. The discrepancy with data of long-term settlement observation of high-rise building was less than 10-20%.

7 REFERENCES

Bartolomey, Omelcak I.M., Yushkov B.S. 1994. *Prediction of pile foundation settlement* [In Russian]. Stroyizdat, Moscow.

Meschyan, S. R. 1995. *Experimental rheology of clay soils*. CRC Press.

He Z., Xiang D., Liu Y. 2020 Triaxial creep test and particle flow simulation of coarse-grained soil embankment filler. *Front. Earth Sci.,* 8(62), doi: 10.3389/feart.2020.00062.

Mirsayapov, I. T., Koroleva I. V. 2011. Calculation model of long-term nonlinear deformation of clay soils under complex stress state [In Russian]// *Bulletin of KazGASU*. No. 2 (16), 121-128.

Sharafutdinov, R.F. 2025. Laboratory and In-Situ Investigations of Deviatoric Creep in Overconsolidated Clay Soils. *Soil Mech Found Eng* 62, 25–33. https://doi.org/10.1007/s11204-025-10024-0.

Shulyatyev, O. A. 2020 Foundations and bases of high-rise buildings [In Russian]. M: ASV, 442 p.

Sharafutdinov, R. F., Razvodovskii D. E., Zakatov D. S. 2024. Single Bored Pile Settlement Prediction Taking into Account the Elastic-Plastic Behavior of the Soil. *Soil Mech Found Eng,* 61, 213–222. https://doi.org/10.1007/s11204-024-09965-9.

Sharafutdinov, R.F., Shulyatyev, O.A., Shulyatyev, S.O. and Andrushenko, V.O. 2025.: Application of Field Monitoring Data for Validation of the Geotechnical Model for the High-Rise Building, *Soil Mechanics Foundation Engineering,* 61, 507-515. https://doi.org/10.1007/s11204-025-10005-3

Sharafutdinov R. F. 2023. Validation metrics for non-linear soil models using laboratory and in-situ tests // Magazine of Civil Engineering, 2023. - 122(6). DOI: 10.34910/MCE.122.5

Mandolini A., Russo G., Viggiani C. 2005. Pile Foundations: Experimental Investigations, Analysis and Design. *Proc. of the 16th Int. Conf. on Soil Mechanics and Geotechnical Engineering*. — Osaka : Millpress, 177–213.