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Compressible soil thickness and settlement prediction using elastoviscoplastic models: a comprehensive method

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Abstract. The paper is dedicated to developing a comprehensive analysis method of the criteria for defining the compressible thickness critical for estimating long-term settlements in buildings and structures situated on soft soils, focusing on their creep behavior. This study introduces an engineering method grounded on the criterion of soil's undrained condition within the mass, considering both elastic and residual deformations through equivalent creep deformations. Unlike previous methodologies, the proposed method facilitates the assessment of long-term settlements by incorporating creep effects over time, employing undrained shear strength for both normally consolidated and overconsolidated soils. The method enables settlement calculations based on static-sounding data, enhancing predictions' accuracy and reliability. This research endeavors to broaden the application of numerical and analytical calculations in real-world practices, employing elastoviscoplastic soil models to design structures on weak foundations.

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1. *Introduction*

Understanding the intricate behavior of soils and rocks in their natural state is significant in geotechnical engineering, as it aids in designing and implementing earth structures and foundations. A vital aspect of this comprehension is the ability to predict the long-term settlements of structures built on soft soils, which requires a sophisticated understanding of soil behavior under different loading conditions [1,2]. This research builds upon significant advancements in the elastoviscoplastic soil model, incorporating the viscoelastic and plastic deformation characteristics to improve settlement predictions. The model's capability to encapsulate the viscoelastic and plastic behavior of soils over time represents a pivotal advancement in geotechnical engineering, particularly for structures built on compressible soil layers [3].

The necessity of this study is underscored by the demand for accurate and reliable models during the design and construction phases of infrastructure projects on soft soils. Traditional models often fail to capture the intricate and time-dependent behavior of soils, which can result in inaccuracies in settlement predictions. This research aims to enhance the criteria for determining the thickness of compressible soils and apply an elastoviscoplastic model, thus establishing a more robust framework for predicting soil settlements. This effort is expected to contribute to developing safer and more durable infrastructure [4].

Supported by a broad spectrum of literature, the foundation of the proposed approach signifies the continuous evolution of soil modelling techniques and their application within geotechnical engineering. The calibration and validation of elastoviscoplastic soil models have been instrumental in providing critical insights into the behavior of Resedimented Boston Blue Clay under diverse loading conditions, serving as a benchmark for the model's development [3]. The work by Ter-Martirosyan [5] defining the parameters of such models further informs this research's methodology, emphasizing the importance of precise parameterization for accurate soil behavior representation.

Furthermore, research on probabilistic approaches to compressible soil thickness [6] and assessing liquefiable soil layers [7] have introduced a statistical perspective to understanding soil consolidation processes, offering a complementary viewpoint to this study. Studies on the behavior of deformable soil media under compaction [8] enhance the understanding of soil-structure interaction, particularly in road construction. Examination of soil-reinforcement interaction parameters [9] provides valuable context for evaluating the performance of structures built on compressible subgrades, thus significantly enriching the research framework.

Wroth's study [10] laid the groundwork for understanding the natural state of geotechnical materials under various conditions. This foundation is crucial for designing and implementing effective earth structures and foundation systems, enabling engineers to predict, how soil and rock will behave under different loads.

The introduction of a soft soil model by Vermeer and Neher [11], which accounts for creep, represents a significant advancement in computational geotechnics. This model allows for more precise simulations of time-dependent soil deformation, enhancing the ability to predict long-term settlements of structures on soft soils. Such a model is vital for designing and maintaining durable infrastructure on challenging ground conditions.

Schmidt's research [11] into earth pressures at rest and their relation to stress history has deepened the understanding of soil mechanics. By highlighting the influence of past stresses on current geostatic conditions, Schmidt's work aids in accurately assessing the stability and safety of geotechnical structures.

Perzyna's exploration [12] of fundamental problems in viscoplasticity laid the theoretical groundwork for analyzing materials that exhibit viscous and plastic behavior under stress. This theory is crucial for modelling the behavior of geotechnical materials under load, contributing to the design of structures that can withstand the complexities of real-world conditions.

The practical application of theoretical frameworks is exemplified by Khankelov [13] through their modelling of segmental excavator working tools for soil compaction. This work showcases integrating research findings into engineering practices, demonstrating, how theoretical models can be applied to solve practical engineering problems, thereby improving construction techniques and equipment.

Advancements in unified constitutive models have ultimately contributed to safer and more efficient infrastructure designs. These developments closely align with ongoing efforts to refine elastoviscoplastic models, which are crucial for enhancing the accuracy of settlement predictions in soft soils. As the field continues to evolve, these sophisticated models are becoming indispensable tools for geotechnical engineers, enabling them to address complex soil-structure interactions and ensure the stability and longevity of constructions built on challenging ground conditions [14-18].

The current study aims to develop an elastoviscoplastic soil model to refine settlement prediction accuracy.

2. Methods

2.1. Basic equations of the elastoviscoplastic model of the soil environment and the criterion of compressible thickness

The deformation components for elastoviscoplastic soil models are determined by the sum of elastic ε_{ij}^{e} and viscoplastic deformations ε_{ij}^{vp} :

$$\varepsilon_{ij} = \varepsilon_{ij}^{e} + \varepsilon_{ij}^{vp}. \tag{1}$$

Let us consider the criterion for limiting the compressible thickness under the conditions of a one-dimensional problem.

For the rate of viscoplastic deformation in the one-dimensional case, the general relation is valid:

$$\dot{\varepsilon}_{v,c} = \frac{\mu^*}{\tau}\left(\frac{p^{eq}}{p^{eq}_{p,new}}\right)^{\beta} = \dot{\varepsilon}_{NC}\left(\frac{p^{eq}}{p^{eq}_{p,new}}\right)^{\beta}; \tag{2}$$

$$\beta = \frac{\lambda^* - k^*}{\mu^*}. \tag{3}$$

By integrating equation (2), subject to constant effective stresses, we can obtain expressions for the increment of relative volumetric strain in the form:

$$\Delta\varepsilon_{vc} = \mu^* \ln\left(1 + \int_0^t \left(\frac{p^{eq}}{p^{eq}_{p,new}}\right)^{\beta} \frac{1}{\tau_{ref}} d\tau\right) = \mu^* \ln\left(1 + \frac{1}{\tau_{ref}}\left(\frac{p^{eq}}{p^{eq}_{p,new}}\right)^{\beta}\right). \tag{4}$$

By equating the logarithm of the second term to zero in equation (4), we can obtain a criterion for limiting the compressible thickness (hereinafter, $\tau_{ref} = \tau$):

$$\frac{t}{\tau}\left(\frac{p^{eq}}{p^{eq}_{p,new}}\right)^{\beta} = 1. \tag{5}$$

The value of the current equivalent volumetric pressure will be:

$$p^{eq} = p^{eq}_{p,new}\left(\frac{\tau}{t}\right)^{\frac{1}{\beta}}; \tag{6}$$

$$p^{eq}_{p,new} = p^{eq}_{p} \exp\left(\frac{\varepsilon^c_v}{\lambda^* - k^*}\right), \tag{7}$$

ε^c_v – volumetric plastic deformation at the loading step.

The resulting defining equation turns out to be expressed through a new value of over-consolidation pressure $p^{eq}_{p,new}$, which, according to (7), in turn, depends on the accumulated value of viscoplastic deformation. Meanwhile, the equation for the components of the viscoplastic deformation rate can be expressed through the reference value of the over-consolidation pressure p^{eq}_{p} (determined during standard compression tests) considering (31) in the form of the equation:

$$\dot{\varepsilon}_{v,c} = \frac{\mu^*}{\tau}\left(\frac{p^{eq}}{p^{eq}_{p,new}}\right)^{\beta} = \frac{\mu^*}{\tau}\left(\frac{p^{eq}}{p^{eq}_{p}}\right)^{\beta} \exp\left(\frac{-\varepsilon^c_v}{\mu^*}\right). \tag{8}$$

Since when deriving equation (6), it was assumed that the increments of creep strains should be reduced to zero, the indicated equation can be reduced to the form:

$$p^{eq} = p^{eq}_{p,t} = p^{eq}_{p}\left(\frac{\tau}{t}\right)^{\frac{1}{\beta}}. \tag{9}$$

The current value of the equivalent volume pressure during loading is determined by equation (9) since the loading vector (one-dimensional case) is located on the K_0 line:

$$p^{eq}_{p} = \frac{\left(1 + 2K_{0,NC}\right)\sigma_{zc}}{3}\left[1 + \left(\frac{\eta K_0}{M_{cs}}\right)^2\right]; \tag{10}$$

$$p^{eq}=\left(p_0+\Delta p\right)\left[1+\left(\frac{\eta K_0}{M_{cs}}\right)^2\right]=\frac{\left(1+2K_{0,NC}\right)\left(\sigma_{zg}+\sigma_{zp}\right)}{3}\left[1+\left(\frac{\eta K_0}{M_{cs}}\right)^2\right]; \tag{11}$$

$$\eta K_0=\frac{3\left(1-K_{0,NC}\right)}{1+2K_{0,NC}}; \tag{12}$$

$$OCR=\frac{\sigma_{zc}}{\sigma_{zg}}. \tag{13}$$

Substituting (10–13) into (9) express the known criterion for limiting the compressible thickness [19,20]:

$$\sigma_{zp}=\sigma_{zg}\left[OCR\left(\frac{\tau}{t}\right)^{\frac{1}{\beta}}-1\right]. \tag{14}$$

The criterion for limiting the compressible thickness, expressed by (14), makes it possible to calculate settlement in one-dimensional conditions in time at the stage of completion of consolidation as for an elastoplastic medium according to the relation:

$$S\left(t>t_{eop}\right)=$$
$$=\left|\begin{array}{l}\int_0^H\frac{C_r}{1+e_0}\lg\left(\frac{\sigma_{zp,i}+\sigma_{zg,i}}{\sigma_{zg,i}}\right)\quad\text{at}\quad\frac{\sigma_{zp,i}+\sigma_{zg,i}}{\sigma_{zc,ref}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c-C_r}}<1\\ \int_0^H\frac{C_r}{1+e_0}\lg\left(\frac{\sigma_{zc,ref}}{\sigma_{zg,i}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c-C_r}}\right)+\int_0^H\frac{C_c}{1+e_0}\lg\left(\frac{\sigma_{zp,i}+\sigma_{zg,i}}{\sigma_{zc,ref}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c-C_r}}\right)\quad\text{at}\quad\frac{\sigma_{zp,i}+\sigma_{zg,i}}{\sigma_{zc,ref}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c-C_r}}\geq 1\end{array}\right., \tag{15}$$

where H is the depth of the compressible thickness, assigned by equation (14).

2.2. Criterion for the undrained nature of soil action

Equation (8), in general form, allows evaluating the criterion for the undrained behavior of soil in a massif under the condition that the volumetric deformation is equal to zero:

$$\varepsilon_v=\varepsilon_v^e+\varepsilon_v^{vp}=0.$$

Accordingly, in this case, the magnitude of volumetric viscoplastic deformation is determined by the expression:

$$\varepsilon_v^{vp}=-\varepsilon_v^e=-k^*\ln\left(\frac{p^{eq}}{p_0^{eq}}\right). \tag{16}$$

Substituting equation (16) into (8) leads to obtaining, by analogy with (5), the criterion equation for the undrained state of a soil mass for the case of a one-dimensional problem in the form:

$$\frac{t}{\tau}\left(\frac{p^{eq}}{p_p^{eq}}\right)^{\beta}\left(\frac{p^{eq}}{p_0^{eq}}\right)^{\frac{k^*}{\mu^*}}=\frac{t}{\tau}\left(\frac{\sigma_{zi}}{\sigma_{zc}}\right)^{\beta}\left(\frac{\sigma_{zi}}{\sigma_{zg}}\right)^{\frac{k^*}{\mu^*}}=$$
$$=\frac{t}{\tau}\left(\frac{\sigma_{zp}+\sigma_{zg}}{\sigma_{zc}}\right)^{\frac{\lambda^*-k^*}{\mu^*}}\left(\frac{\sigma_{zp}+\sigma_{zg}}{\sigma_{zg}}\right)^{\frac{k^*}{\mu^*}}=1. \tag{17}$$

By analogy with equation (14), a criterion for the undrained nature of work in a soil massif could be obtained in the form of a relation:

$$\sigma_{zp} = \sigma_{zg}\left[OCR^{\frac{\lambda^*-k^*}{\lambda^*}}\left(\frac{\tau}{t}\right)^{\frac{\mu^*}{\lambda^*}} - 1\right] = \sigma_{zg}\left[OCR^{\Lambda}\left(\frac{\tau}{t}\right)^{\frac{\mu^*}{\lambda^*}} - 1\right]; \quad (18)$$

$$\Lambda = \frac{\lambda^* - k^*}{\lambda^*}. \quad (19)$$

The two-component equation (15) for calculating settlement is determined by the accepted relation (1), which expresses the volumetric deformation as the sum of elastic and creep deformation. In Fig. 1 the trajectory for calculating the settlement under the two-component expression (15) is shown in blue color.

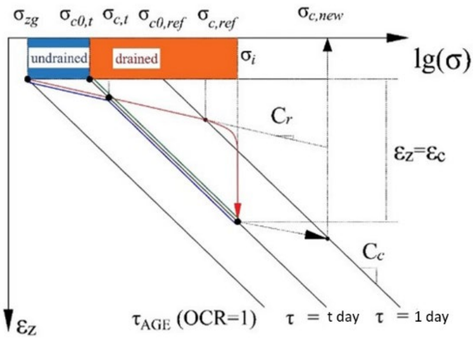


Figure 1. Scheme for determining settlement under one-dimensional conditions.

On the other hand, the total amount of strain can be expressed as a relation in which the equivalent amount of creep strain determines the total amount of strain:

$$\varepsilon_{ij} = \varepsilon_{ij}^{vp}. \quad (20)$$

In this case, the equation for calculating the settlement will take the form (the trajectory is shown in Fig. 1 in green):

$$S(t > teop) = \int_0^H \frac{C_c}{1+e_0} \lg\left(\frac{\sigma_{zp,i} + \sigma_{zg,i}}{\sigma_{c0,t}}\right) = \int_0^H \frac{C_c}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{c0,t}}\right). \quad (21)$$

In equation (21), the symbol σ_i denotes the sum of household and additional effective stresses in the skeleton of the soil massive (Fig. 1).

In equation (21) the concept of the initial value of over-consolidation pressure is introduced, corresponding to the intersection of the household stress level (from its weight) with the line of standard compaction by a horizontal line:

$$\sigma_{c0,t} = \sigma_{c0,ref}\left(\frac{\tau}{t}\right)^{\frac{C_a}{C_c}} = \sigma_{zg} OCR^{\Lambda}\left(\frac{\tau}{t}\right)^{\frac{C_a}{C_c}}. \quad (22)$$

Considering relation (21), a one-component expression for calculating settlement is obtained:

$$S(t > teop) = \int_0^H \frac{C_c}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{c0,ref}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c}}\right) = \int_0^H \frac{C_c}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{zg} OCR^{\Lambda}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c}}\right), \quad (23)$$

where H is the depth of the compressible thickness, assigned in accordance with equation (18).

To perform verification procedures, determining the degree of plasticity (19) is possible based on the ratio of the logarithms of the initial value of the over-consolidation pressure $(OCR_{0,ref})$ and the over-consolidation pressure, calculated, for example, by Terzaghi's method (OCR_{ref}):

$$\Lambda = \frac{\lg\left(OCR_{0,ref}\right)}{\lg\left(OCR_{ref}\right)}. \quad (24)$$

Based on the processing of several experiments for the parameter of normalized undrained strength [21,22], the following ratio was suggested:

$$\left[\frac{s_u}{\sigma'_v}\right]_{oc} = \left[\frac{s_u}{\sigma'_v}\right]_{nc} OCR^{\Lambda} = mOCR^{\Lambda}. \quad (25)$$

Considering relation (25), the amount of settlement represented by (23) takes the form of an expression depending on the value of the undrained strength of the soil and the normalized coefficient of undrained strength *m* in a normally compacted state:

$$S\left(t > teop\right) = \int_0^H \frac{C_c}{1+e_0} \lg\left[\frac{\sigma_i}{\sigma_{zg,i}} \frac{s_{u,nc}}{s_{u,oc}} \left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c}}\right] = \int_0^H \frac{C_c}{1+e_0} \lg\left[\frac{\sigma_i m}{s_{u,oc}} \left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c}}\right]. \quad (26)$$

Accordingly, the criterion for limiting the compressible thickness can be expressed in the form of an equation depending on the ratio of the reference values of undrained strength in overconsolidated and normally compacted states (or their normalized values for a SHANSEP type structure, equation (25)):

$$\sigma_{zp} = \sigma_{zg}\left[\left[\frac{\left[\frac{s_u}{\sigma'_v}\right]_{oc}}{\left[\frac{s_u}{\sigma'_v}\right]_{nc}}\right]\left(\frac{\tau}{t}\right)^{\frac{C_a}{C_c}} - 1\right] = \sigma_{zg}\left[\frac{s_{u,oc}}{s_{u,nc}}\left(\frac{\tau}{t}\right)^{\frac{C_a}{C_c}} - 1\right] = \sigma_{zg} k_{u,i}\left(t\right), \quad (27)$$

where $k_{u,i}\left(t\right)$ – the coefficient of limitation of compressible thickness (undrained work) is variable in time.

Considering (27), it is possible to obtain a general one-component expression for calculating settlement over time using two criteria for limiting the compressible thickness:

$$S\left(t > t_{eop}\right) = \left| \begin{array}{ll} \int_0^H \frac{C_r}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{zg,i}}\right) & \text{at} \quad \frac{\sigma_i}{\sigma_{zg,i}\left(1+k_i\left(t\right)\right)} < 1 \\ \int_0^H \frac{C_r}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{zg,i}\left(1+k_{u,i}\left(t\right)\right)}\right) & \text{at} \quad \frac{\sigma_i}{\sigma_{zg,i}\left(1+k_i\left(t\right)\right)} \geq 1 \end{array} \right. . \quad (28)$$

Undrained shear strength relationship used in (26) can often be approximated from static-sounding results.

Mayne [23], based on a correlation analysis of over-consolidation pressure and resistance to penetration of the probe cone for 49 types of clays, proposed a simple relation:

$$OCR = k\frac{q_c}{\sigma'_v}. \quad (29)$$

Here k is the proportionality coefficient, which, according to the results of statistical processing, was 0.29.

By grouping the ratios (28) and (29), the equation (30) is obtained:

$$S\left(t > t_{eop}\right) = \begin{cases} \int_0^H \frac{C_r}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{zg,i}}\right) & \text{at} \quad \frac{\sigma_i}{kq_c}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c - C_r}} < 1 \\ \int_0^H \frac{C_c}{1+e_0} \lg\left(\frac{\sigma_i}{\sigma_{zg,i}\left[k\frac{q_c}{\sigma'_v}\right]^{\Lambda}}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c}}\right) & \text{at} \quad \frac{\sigma_i}{kq_c}\left(\frac{t}{\tau}\right)^{\frac{C_a}{C_c - C_r}} \geq 1 \end{cases} \tag{30}$$

Equation (30) can be modified based on more complex relations between overconsolidation coefficients or undrained shear strength based on static-sounding results (considering the developing pore pressure in the massif).

3. Results and Discussion

3.1. Example of Settlement Calculation using Undrained Strength Parameters

An example of calculating the settlement over time is the administrative building in the central part of St. Petersburg, erected on a slab foundation on a weak foundation (dimensions 30×25 m in plan). Fig. 2 and 3 show the physical and compression properties of the clay soils composing the site.

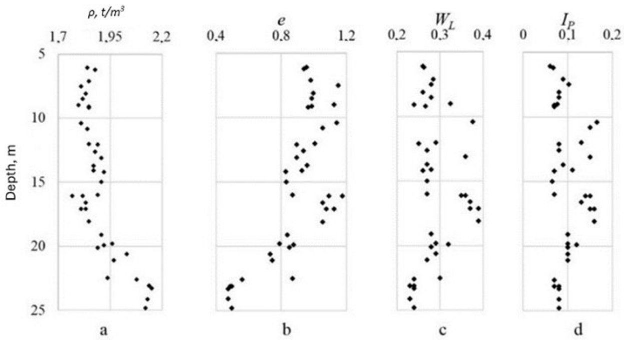


Figure 2. Physical properties of clay soils composing the site: a) density; b) porosity coefficient; c) is the moisture content at the yield boundary; d) plasticity index.

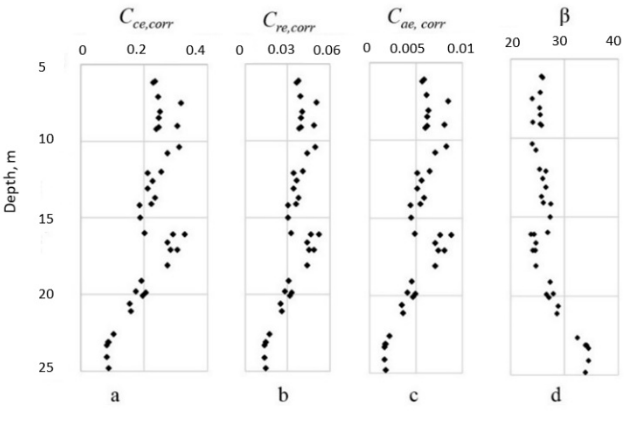


Figure 3. Properties of clay deposits according to empirical dependencies: a) compression index, b) unloading/ recompression index, c) secondary consolidation index, d) coefficient β (equation (3)).

The results of static sounding are shown in Fig. 4a.

Equations (27) and (28) for calculating settlement include the normalized parameter of undrained shear strength and the value of undrained strength in the over-consolidated state (in situ).

For approximate estimates of non-consolidated-undrained shear strength in St. Petersburg, a well-known relation can be used that uses the resistance value of the probe cone during static sounding with a constant coefficient $N_k \approx 19$:

$$C_{UUC} = \frac{q_c}{19}, \tag{31}$$

where q_c – resistance to penetration of the probe cone, kPa.

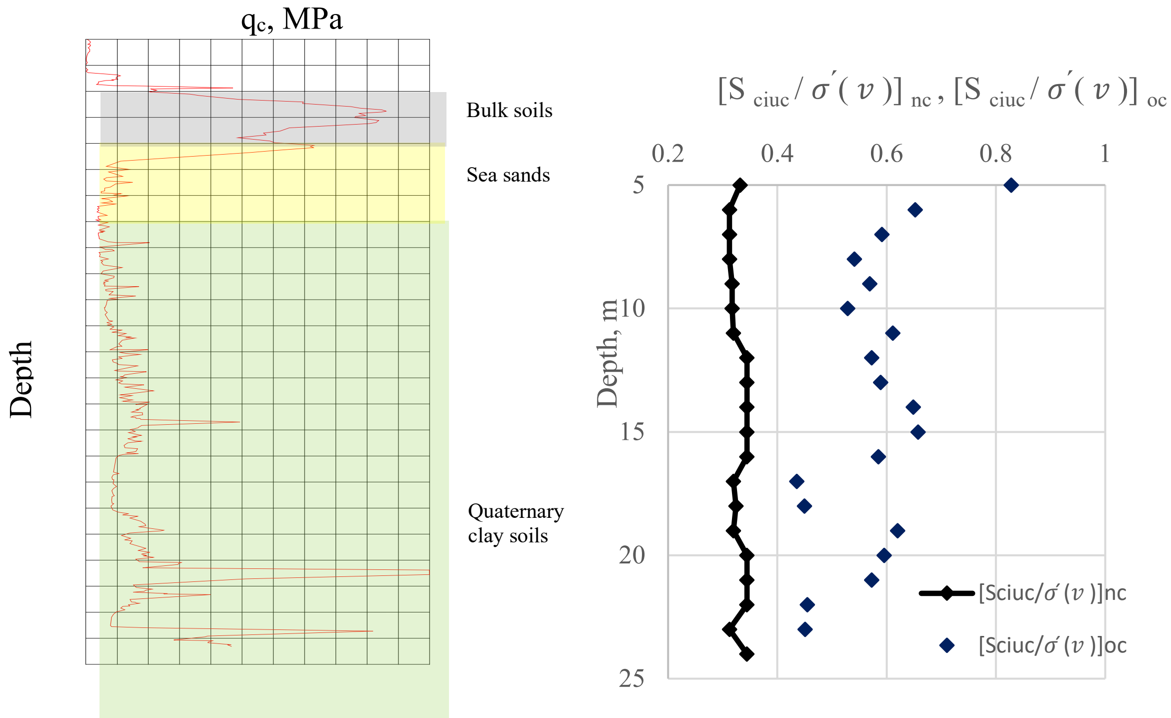
Relations (26) and (27) are valid for consolidated-undrained tests. Accordingly, it is necessary to convert non-consolidated-undrained characteristics into consolidated-undrained ones to use these equations. It can be done, for example, based on Chen's research [24], connecting isotopically unconsolidated-undrained and isotopically consolidated-undrained strengths by the ratio:

$$\frac{s_{UUC}}{s_{CIUC}} = 0.911 + 0.499\left[\frac{s_{UUC}}{\sigma'_v}\right]_{oc}. \tag{31}$$

The theoretical solution for the normalized parameter of isotropically consolidated-undrained strength in a normally compacted state will take the form:

$$m_{CIUC} = \left[\frac{s_{CIUC}}{\sigma'_v}\right]_{nc} = \frac{M}{2}\left[\frac{1}{2}\right]^{\Lambda} = \frac{3\sin(\varphi)}{3-\sin(\varphi)}\left[\frac{1}{2}\right]^{\Lambda}. \tag{32}$$

At angles of internal friction of the compressible mass of 28^0-32^0, the normalized parameter of the isotropically consolidated undrained strength will be 0.29–0.32 (Fig. 4b). Based on equation (31), the value of isotropically consolidated undrained strength can be calculated. The value of its normalized parameter is also shown in Fig. 4b.



a. b.

Figure 4. Engineering-geological conditions of the site: a) graph of penetration resistance to the probe cone during static sounding; b) determination of the normalized parameters of the undrained shear strength in a normally compacted and over-consolidated state (31), (32).

Using relation (25), the value of the soil mass over-consolidation coefficient can be approximated based on the ratios of the calculated normalized parameters of undrained shear strength. In general, the calculated value of the overconsolidation coefficient for the engineering-geological conditions of St. Petersburg is in reasonably good agreement with the determination based on the empirical relationship

(29) with a coefficient k = 0.28–0.29. Accordingly, the settlement of the soil mass can be calculated using equations (27) and (28), based on relation (30).

Fig. 5a and 5b show the results of calculating the total stresses in the soil mass, the reference value of the overconsolidation pressure and the value of vertical relative deformations in time (under the center of the loaded area of the slab – mainly under compression conditions).

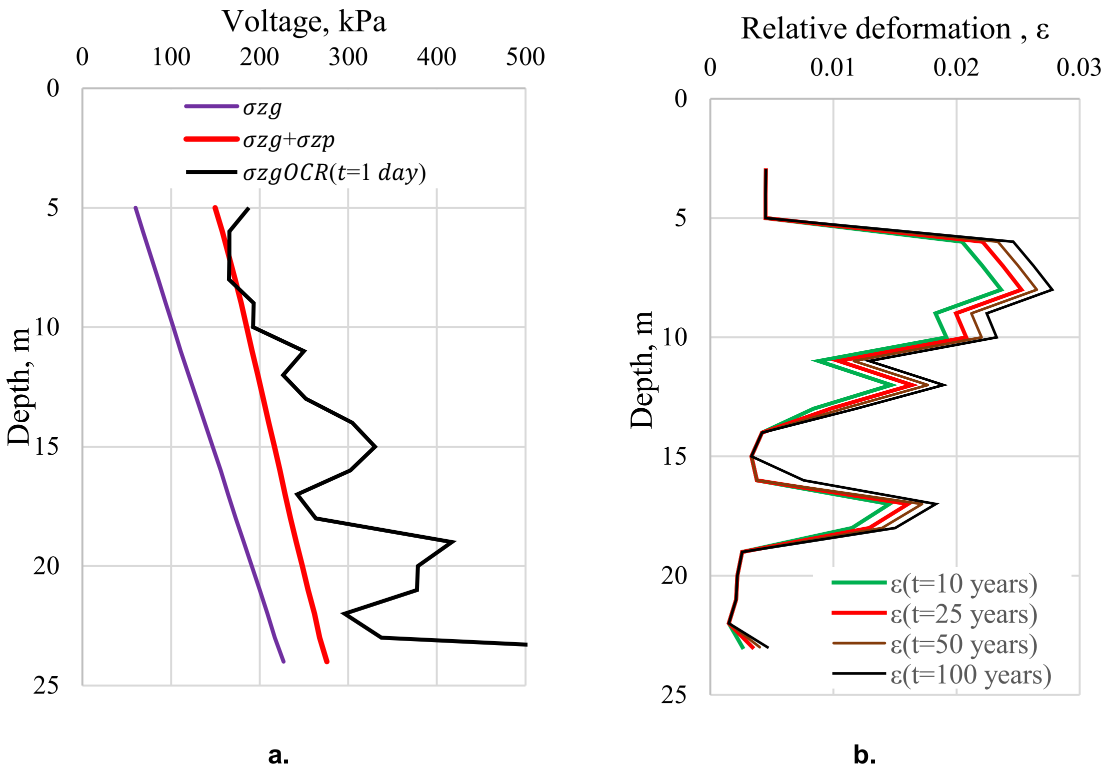


Figure 5. Results of engineering calculations: a. distribution of stresses in the massif and the initial/reference value of over-consolidation pressure; b. relative vertical deformations of the massif at different times.

Fig. 6 compares observed and calculated settlements over time based on numerical solutions (soft soil creep, soft soil models) and the proposed engineering method using undrained shear strength parameters. According to the calculation results, the predicted precipitation values based on the engineering and numerical methods (according to the SSC model) are nearly 25–100 years. In the remaining (mainly initial) periods, the engineering calculation overestimates the development of settlement, which is explained by the failure to consider the phenomena of filtration consolidation (the solution was obtained at constant maximum effective stresses in the soil mass), as well as the failure to consider the construction time of the building.

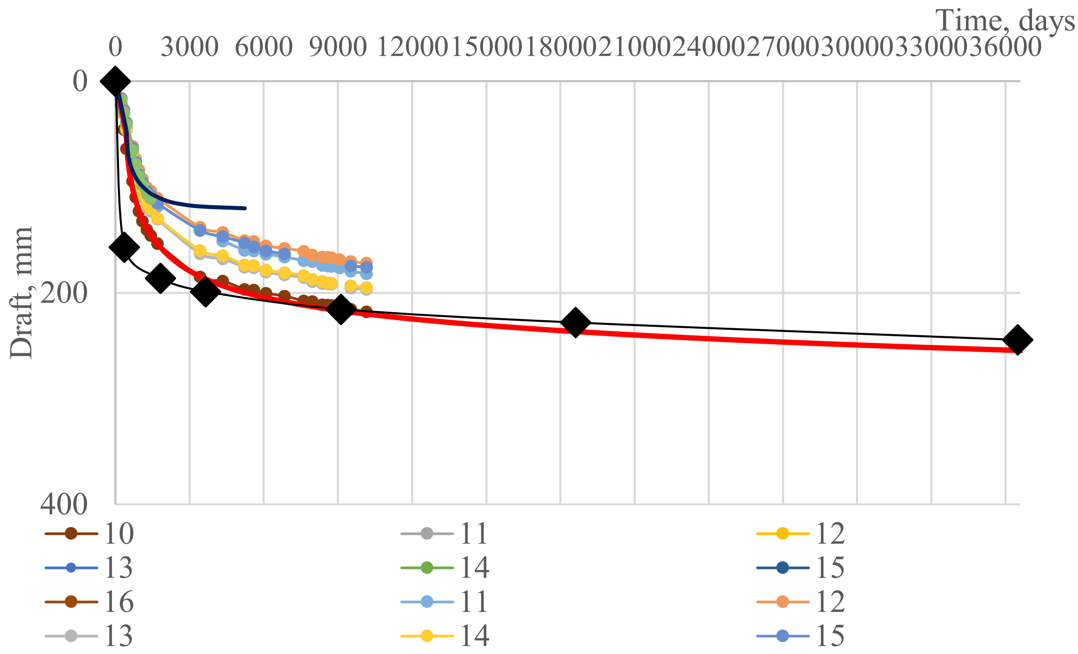


Figure 6. Comparison of calculation results based on numerical solutions (SSC, SS models) and the engineering method.

3.2. Discussion of Research Results

The presented work aims to develop an engineering method for calculating settlements of buildings and structures on weak soils. The obtained solutions make it possible to calculate long-term precipitation over time by modifying the layer-by-layer summation method.

For weak soils and additional loads comparable to the reference value of overconsolidation pressure, a one-component equation (23, 28) that already includes the elastic parts of deformations can be used to calculate settlement. The method's main advantages include the simplicity and speed of calculations and the ability to control the depth of the compressible soil mass at each calculated moment in time. The procedure for verifying the compressible layer limitation coefficients involves checking whether the calculated values are close to zero [19,20], which is especially important for thick layers of weak clayey deposits.

Considering the disturbance of natural structure of the samples, the calculated coefficients of limitation of the compressible thickness are often equal to zero or negative, leading to incorrect estimates of the degrees of deformation of the soil massif. The compressible layer limitation coefficients thus determined can be directly used to calculate settlement over time.

Values of consolidated undrained strength based on laboratory tests and indirect field methods, such as static probing, impeller testing, and dilatometric studies, can be used to perform calculations. It is possible to use a combination of these indirect field methods to calculate the settlement.

Also, an undoubted advantage of the one-component method is the possibility of using the concept of the ratio of secondary consolidation indices and compression indices in a power dependence (23, 26, 30) [25]. This approach for calculating settlement using field methods can be applied even at the soil classification level using appropriate regional correlations.

4. Conclusion

The paper provides method for calculating long-term settlements of buildings and structures on soft soils, considering their creep. Based on the results of this research, the derived conclusions are as follows:

1. A comprehensive framework (the results of which are expressed in Equations 28 - 30) has been established and validated for assessing the limit of compressible thickness in soft soils and for predicting long-term settlements through an elastoviscoplastic model (Equations 1-23), as detailed in this study.
2. The introduced elastoviscoplastic model provides a sophisticated understanding of soil mechanics under load, capturing both immediate and progressive deformations by incorporating viscoelastic and plastic characteristics of soil, thus offering an accurate representation of soil behavior under sustained loading conditions.

3. Criteria for determining the limiting compressible thickness have been developed. These criteria assist in identifying significant segments within the soil profile that are vital for settlement calculations, thereby enhancing the foundation design and evaluation processes on soft soils.
4. The precision of the model in predicting settlements has been illustrated through extensive numerical simulations. A conducted sensitivity analysis emphasizes the importance of accurate parameterization, highlighting the necessity of precise soil property determination for reliable results.
5. The practicality and reliability of the model and its criteria have been reinforced through empirical validation against real-world studies. The alignment of model predictions with observations from diverse engineering projects confirms the model's validity and potential to refine construction methodologies by enabling more informed foundational design decisions on soft soils.

This research marks a significant advancement in modelling soil behavior and predicting long-term settlements, enhancing the understanding of elastoviscoplastic soils.

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