# CONSOLIDATION ANALYSIS OF CLAYEY SOILS USING ANALYTICAL TOOLS

# ANALIZA KONSOLIDACIJE GLINASTIH TAL S POMOČJO ANALITIČNIH ORODIJ

**Bartłomiej Szczepan Olek**
Tadeusz Kościuszko University of Technology,
Warszawska 24, 31-155 Kraków, Poland
E-mail: bartlomolek@gmail.com

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## Keywords

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## Ključne besede

konsolidacija; glina; filtracija; koeficient konsolidacije; optimizacija

## Abstract

*The uncoupled Terzaghi consolidation equation (excess pore pressure only) is widely used to predict the rate and magnitude of settlements in clayey soils. The theoretical solution is based on the approach of considering the soil permeability and compressibility as one parameter obtained by experimental methods – the coefficient of consolidation cv. This article presents two analytical tools that allow us to determine the consolidation coefficient, which is independent of a single measurement point and represents the consolidation behavior for the significant progress of settlements. The presented methods were based on the process of optimizing the coefficient of consolidation value and the quasi-constant approach, which assumes the identification of a quasi-filtration consolidation phase using the log $c_v$ - U relationship. To assess the validity of each method, the experimental results were compared to the theoretical solution and quantified using a new statistical parameter $d_n$.*

## Izvleček

*Nevezana Terzaghija enačba vertikalne konsolidacije (upoštevan samo porni nadtlak) se pogosto uporablja za napovedovanje hitrosti in velikosti pomikov v glinastih tleh. Teoretična rešitev temelji na pristopu upoštevanja prepustnosti tal in stisljivosti v enem parametru, pridobljenem z eksperimentalnimi metodami - koeficient vertikalne konsolidacijske cv. V članku sta prikazani analitični orodji, ki omogočata določitev koeficienta vertikalne konsolidacije, ki je neodvisen od posamezne merilne točke in opisuje konsolidacijsko vedenje za značilen časovni razvoj posedkov. Predstavljene metode temeljijo na procesu optimizacije vrednosti koeficienta vertikalne konsolidacijske in kvazi - konstantnega pristopa, ki predpostavlja identifikacijo faze kvazifiltracijske konsolidacije z uporabo relacije log $c_v$ - U. Veljavnost obeh metod smo ocenili s primerjavo eksperimentalnih rezultatov in teoretičnih rešitev ter kvantitativno ovrednoteli z uporabo novega statističnega parametra $d_n$.*

# 1 INTRODUCTION

Studying the properties of geomaterials is one of the basic aspects involved in predicting the soil-structure interaction and planning any soil-strengthening modifications. Geomaterials include all the natural, processed or produced and improved materials used in geotechnical applications. Natural geomaterials are mainly soils and rocks, as well as mixed material behaving as a transient between soil and rock. Natural soils, especially soft clays, muds and expansive soils, can be problematic and may cause a potential threat to a construction. During the design of foundations and embankments on clayey soil, it is crucial to predict the magnitude and rate of settlements. The accuracy of predictions in the design stage depends on the input value of the coefficient of consolidation cv. A correct assessment of the real values of this parameter and the impact of the factors influencing it is a difficult problem. It has been a serious challenge for researchers and has not yet been fully resolved.

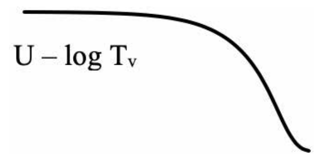
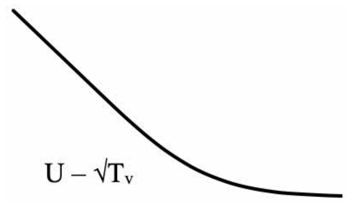
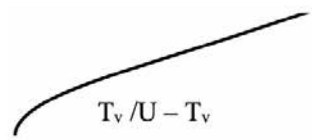
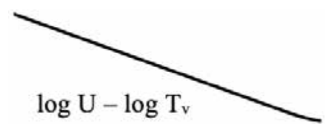
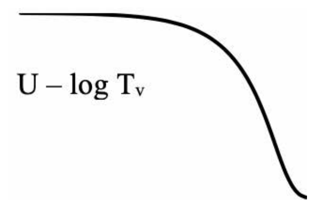
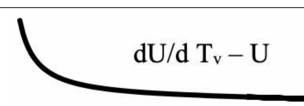
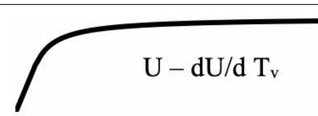
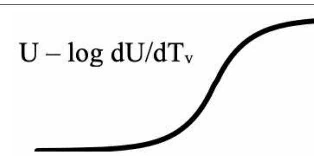
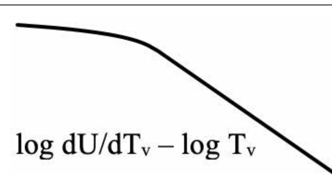
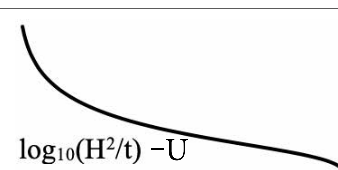
The consolidation process is a combination of two phenomena: permeability, which controls the rate at which water is removed from the pore space (and thus the rate of the settlement at any time) and compressibility, which controls the evolution of the distribution of excess pore-water pressure (and thus the duration of the consolidation process). The widely used Terzaghi theory is based on a linear stress-strain relationship and constant permeability. Theoretical solutions were based on a consideration of all the soil properties as one parameter – the coefficient of consolidation cv, obtained with experimental methods ([1]). Over the past 50 years, difficult and time-consuming attempts have been made to develop appropriate methodologies and interpretations of consolidation tests. The valuable material refers to the studies on the standardization of time-compression data analysis and can be found in ([2], [3], [4], [5], [6], [7], [8], [9], [10], [11], [12]). The achievements of the above-mentioned researchers relate to the commonly accepted Terzaghi theory and could be used for a uniform, initial, pore-pressure distribution. Solutions for a non-uniform and sinusoidal initial pore-pressure distribution can be found in Lovisa et al. [13] and Lovisa and Sivakugan [14], respectively. The existing methods for calculating the consolidation coefficient were collected in Table 1, where the experimental and theoretical relations considered during the analysis were included as well as the individual expressions for the coefficient of consolidation.

The realistic application of Terzaghi's theory for determining the consolidation coefficient assumes the identification of a primary consolidation range. This recognition can be conducted by the fitting procedure of the theoretical relationship between different variations of the degree of consolidation and the dimensionless time factor $U – T_v$ to the measured deformation with time or the pore-water pressure dissipation. This kind of procedure is carried out on the basis of the similarity between the observed and theoretical curves, which can be presented and interpreted in various ways. Consolidation coefficients determined on the basis of fitting procedures are characterized by a large dispersion, which results from choosing different reference points on the experimental curve and a different way of determining the start and the end of the primary consolidation. Cohesive soils are variable due to the nature of their formation (genesis) and the impact of environmental processes. Recognizing the coefficient of consolidation as a constant parameter is the main disadvantage of Terzaghi's conventional theory. It is known that the consolidation properties of the soil should be treated in an independent manner, and considering them as one coefficient makes it difficult to relate the experimental course of the process with the theoretical solution.

The main goal of the work was to develop a reliable interpretation tool for consolidation studies based on the optimization procedure. Special attention was paid to the secondary consolidation effect on the filtration nature of the process and on the relative duration of the quasi-filtration consolidation phase.

During the analysis, three basic assumptions resulting from Terzaghi's theory were examined: (i) the quasi-constant consolidation coefficient; (ii) the convergence between the theoretical and the experimental course of the consolidation curves; and (iii) parallelism in the course of the curves of the pore-pressure dissipation and deformation. This paper examines those aspects based on an analysis of the consolidation data with settlement and pore-water pressure measurements during the consolidation using a Barden-Rowe hydraulic consolidometer. Tests conducted on various soils with different liquid and plastic limits have been evaluated and the coefficient of consolidation has been determined. Two methods for computing the coefficient of consolidation were presented in the study.

**Table 1**. Comparison of existing methods for determining the coefficient of consolidation.

| Method | Experimental relation | Expression | Form of the theoretical curve | Reference |
|---|---|---|---|---|
| Metoda log *t* | *δ – log t* | $c_v = \frac{0{,}196H^2}{t_{50}}$ |   | Casagrande & Fadum [15] |
| Metoda √*t* | *δ* – √*t* | $c_v = \frac{0{,}848H^2}{t_{90}}$ |   | Taylor [16] |
| Slope method | *δ* – √*t* | $c_v = \frac{\pi}{4}\left(\frac{m_1}{\delta_{EOP}}\right)^2 H^2$ | | Al.-Zoubi ([17], [18]) |
| Rectangular hyperbola method | *t/δ – t* | $c_v = 0.24\frac{M}{c}H^2$ |   | Sridharan et al. [4] |
| Logarithmic method | *log δ – log t* | $c_v = \frac{(\pi/4)H^2}{t_{88.3}}$ |   | Sridharan & Prakash [6] |
| Inflection point method | *δ – log t* | $c_v = \frac{0.405H^2}{t_{70}}$ |   | Mesri & Feng [19] |
| Early stage method | *δ – log t* | $c_v = \frac{0.0385H^2}{t_{22.14}}$ | | Robinson & Allam [20] |
| SRS method | *dδ/dt – δ* | $c_v = \frac{m_1H^2}{2{,}468}$ |   | Al-Zoubi [21] |
| *δ–dδ/dt* method | *δ – dδ/dt* | $c_v = -\frac{4H^2}{\pi^2 m_2}$ |   | Tewatia et al. [10] |
| *δ–log dδ/dt* method | *δ – log dδ/dt* | $c_v = -\frac{0.2566v_{16.19}H^2}{s_{50}}$ |   | Tewatia et al. [10] |
| Velocity method / Improved velocity | *log dδ/dt – log t* | $c_v = \frac{0.793H^2}{t_{88.5}}$ |   | Parkin [22] Pandian et al. [23] |
| One point method | *log₁₀(H²/t) – U* | $c_v = \frac{T_iH^2}{t_i}$ |   | Sridharan et al. [24] |

## 2 PRINCIPLES OF TERZAGHI'S CONSOLIDATION THEORY

The one-dimensional differential equation that governs the consolidation and pore-water-pressure dissipation process is expressed as follows:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \qquad (1)$$

where $t$ is the time variable, $u$ is the pore-water pressure, and $z$ is the depth below the top of the soil layer.

By introducing the dimensionless variables:

$$Z = \frac{z}{H} \qquad (2)$$

and

$$T_v = \frac{c_v t}{H^2} \qquad (3)$$

equation (1) is as follows:

$$\frac{\partial u}{\partial T_v} = c_v \frac{\partial^2 u}{\partial Z^2} \qquad (4)$$

The dimensionless time factor $T_V$ defined by equation (3) is related to the average degree of consolidation $U$, which determines the progress of the process. The solution to equation (4) for the initial uniform excess pore-water pressure inside the soil layer is given by:

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v} \qquad (5)$$

The theoretical velocity of consolidation $U=dU/dt$ is a product of the differentiation of the relationship between $U$ and $T_V$ with respect to $T_V$. Depending on the degree of consolidation, the following approximations can be used:

$$\dot{U} = \frac{1}{\sqrt{\pi T_v}} \quad \text{for} \quad T_v \approx 0.197 (U \le 50\%) \qquad (6)$$

$$\dot{U} = 2e^{-\left(\pi^2 T_v / 4\right)} \text{for} \quad T_v > 0.197 (U > 50\%) \qquad (7)$$

## 3 COURSE OF CONSOLIDATION

Considering one-dimensional strain, volume changes are caused by the initial or immediate compression, the primary consolidation, and the secondary (rheological) consolidation. It should be noted that rheological conditions depend on the soil skeleton's susceptibility to plastic deformations. The progress of the consolidation process is assessed on the basis of pore-pressure dissipation or the relative settlement of the consolidated layer (Fig. 1).

The initial compression occurs almost immediately after the load application due to the expulsion and compression of air in the voids. Primary consolidation is a time-dependent deformation caused by the excess of pore water pressure. Tewatia et al. [21] separated three phases of this deformation using the relationship between the compression and the compression rate. The first primary phase is characterized by the smallest impact of secondary consolidation effects and the calculated values of the coefficient of consolidation are the highest. After that the transition from first primary to second primary phase occurs. The second primary phase in many soils is characterized by a constant coefficient of consolidation value for a considerable percentage of the total settlement. Olek and Woźniak [22] separated this phase using the criterion of a quasi-constant value of the coefficient of consolidation and the relationship between the degree of consolidation and the coefficient of consolidation. As the

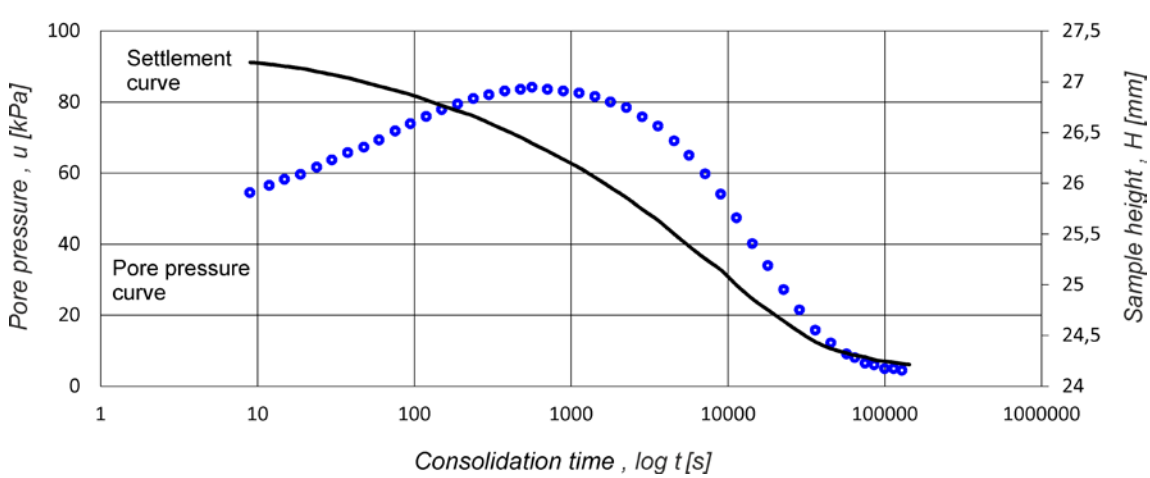



**Figure 1**. Typical experimental course of consolidation for clayey soil.

consolidation progresses, the impact of the soil's secondary consolidation increases. The transient behaviour is characterized by obtaining different temporary values of cv. The deformations in this phase result from both the pore pressure dissipation and the elasto-plastic nature of the soil skeleton's compression. The last phase is a pure creep, time-dependent deformation under a virtually constant effective stress. It should be noted that not all the phases are observed in all the soils.

The pore pressure dissipation curve is characteristic. In research practice, the mobilization delay characterized by an increase in the pore pressure is usually observed. Dobak and Pająk [23] indicated some soil properties (particle size distribution, nature of micro-pore connections, content of minerals prone to swelling) that determine the delay of the load transfer on the liquid phase. The character of the pore pressure increases and reaches its stabilized maximum value $u_{b,max}$. As the pore pressure is mobilized, the larger but not fully developed influence of the limited permeability of soil causes a delay in the deformation. It can also be seen that the volume of the soil is temporarily reduced after loading due to compression or releasing gases from the sample.

The courses of the uniaxial strains and the pore-pressure dissipation do not usually overlap. Regarding the theoretical assumptions, changes in the voids ratio e are not proportional to the changes in the effective stress, and the compressibility and permeability parameters for a relatively high stress applied, decrease during the consolidation process. The explanation of the above can be made on the basis of three definitions of the degree of consolidation, referring to excess pore water pressure, changes in the effective stress and changes in the strain. Comparing them with each other, some irregularities can be encountered. Terzaghi's theory assumes that the change in the effective stress is almost linearly dependent on the deformation or change in the voids ratio. However, this is not correct, because this change is proportional to the change in the logarithm of the effective stress. During the consolidation process, the thickness of the loaded soil layer decreases due to the decrease in the voids ratio. The corresponding settlement of the layer at any time is expressed as a percentage of the total settlement and is called the average degree of consolidation $U_{avg}$. The average degree of consolidation can be expressed as follows:

$$U_{avg} = 1 - \frac{\int_{z=0}^{z=2H} u dz}{2H \times u_0} = \frac{\int_{z=0}^{z=2H} (\Delta\sigma' - u) dz}{2H \times \Delta\sigma'} \qquad (8)$$

where $H$ refers to the layer thickness, $u_0$ is the initial excess pore water pressure caused by the load application. The consolidation process can be considered as completed when the total excess pore water pressure is dispersed due to the load increase. However, because of the absence of a linear relationship between the changes in the pore pressure and the voids ratio, the average degree of consolidation over time calculated on the basis of the pore water pressure measurements $U^{u}_{avg}$ is not equal to the average degree of consolidation determined on the basis of the registration of settlements $U^{\varepsilon}_{avg}$. This can be expressed as follows:

$$U^{u}_{avg} \neq U^{\varepsilon}_{avg} \qquad (9)$$

and

$$1 - \frac{\int_0^{2H} u dz}{2H \times (\Delta\sigma')} \neq 1 - \frac{\int_0^{2H} \varepsilon_t dz}{\int_0^{2H} \varepsilon_{t=\infty} dz} \qquad (10)$$

# 4 RELIABILITY OF THE CONSOLIDATION ANALYSIS

In this section the two methods for determining the coefficient of consolidation are briefly described together with preliminary studies of the usefulness of the considered solutions.

## 4.1 Optimisation method for the coefficient of consolidation and the convergence criteria

Using Terzaghi's model to describe the consolidation process has certain consequences. The course of the consolidation caused by the flow of water through the soil is determined by a set of curves. A fixed value of the consolidation coefficient is assigned to each curve. The compatibility between the experimental data and the theoretical solution can be the criterion for compliance with Terzaghi's model. In this study, the theoretical characteristics of the consolidation progress with the smallest possible discrepancy were assessed using the statistical parameter $d_n$:

$$d_n = \frac{\sum \frac{\left| U_{n,i} - U^*_{n,i} \right|}{U_{n,i}} \times w_{n,i}}{\sum w_{n,i}} \qquad (11)$$

$$w_{n,i} = \frac{U^*_{n,i} - U^*_{n,i-1}}{2} + \frac{U^*_{n,i+1} - U^*_{n,i}}{2} \qquad (12)$$

where $U_{n,i}$ is an experimental consolidation degree, $U^*_{n,i}$ is a consolidation degree calculated for a theoretical solution on the basis of the modified dimensionless time factor $T_v^{\mathrm{mod}}$ and $w_{n,i}$ is a range around each theoretical point $U^*_{n,i}$ characterizing the dispersion. In Figure 2 a graphical presentation of this approach is shown, where the dashed line refers to the experimental course and the continuous line to the theoretical one. The best-fitted model curve with the corresponding consolidation coefficient is the one for which the $d_n$ parameter is the smallest. The use of a particular type of weighted average allowed us to determine accurately the representation of individual measurements under changing axial deformation or the speed of the pore water pressure dissipation conditions, taking into consideration the real environment of each point. A similar comparison of the consolidation was conducted by Mikasa and Takada [24] based on the curve-rule method, Lovisa, Sivakugan & Read [25] using the variance method and Sebai & Belkacemi [26] using a probabilistic method and a minimization of the sum of the squared residual (SSR). In the second and third approaches, the authors applied ranges of probable values for $d_0$, $d_{100}$ and $c_v$.

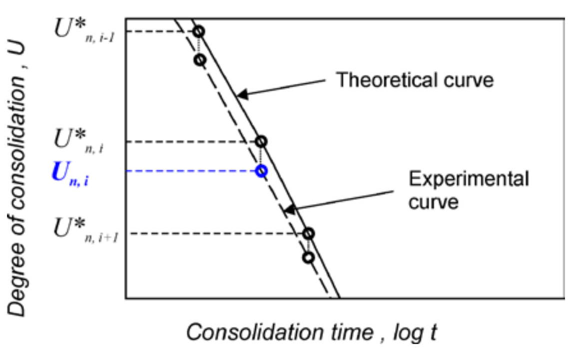



**Figure 2**. Graphical illustration for the estimation of the dispersion between the theoretical and experimental consolidation progress.

## 4.2 Quasi-constant $c_v$ method

For the initial, uniform distribution of excess pore water pressure $u_0$ and double drainage, the average degree of consolidation can be recorded as:

$$U_{avg} = 1 - \frac{\int_0^H u_e dz}{u_i} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \exp^{-M^2 T_v} \qquad (13)$$

where $u_i$ is the initial excess pore pressure distribution as a result of an applied load, $u_e$ is an excess pore water pressure at any time during the consolidation process, $M=(\pi/2)(2m+1)$ and $m$ is an integer. The experimental data obtained from the test can be converted into a dimensionless time factor $T_v$ using the following expression:

$$T_v = \frac{c_v t}{H^2} \qquad (14)$$

Most graphical methods based on the curve-fitting procedure assume obtaining a consolidation factor for one selected point. In the case of the log ($t$) method of Casagrande and Fadum [11] the point refers to 50% of the consolidation progress. In Taylor's $t^{1/2}$ method [12], this point corresponds to 90% of the consolidation progress. The remaining extensions of the above methods mainly concern the choice of a different reference point ([15], [16], [27], [28]). Through the functional dependence $c_v$ – $U$ plotted on a semi-logarithmic chart, the variability of the coefficient of consolidation can be examined in relation to the entire experimental course of consolidation. Using the quasi-determination criterion, such a consolidation phase can be determined, where constant or quasi-constant values of the consolidation coefficient for a significant part of the deformation course are observed. The values of the consolidation coefficient for individual reference points are calculated on the basis of two solutions for the dependence $T_v$ – $U$, assuming a parameter $T_v$ for a rectangular or parabolic distribution of the pore-water pressure, using the following formula:

$$c_{v,i} = \frac{T_{v,i} H_i^2}{t_i} \qquad (15)$$

where $c_{v,i}$ is the coefficient of consolidation for the considered time $t_i$ , and $H_i$ refers to the height of the sample at the analysed time $t_i$.

The parametric rectangular distribution is based on the solution of the rectangular distribution of the excess pore water pressure in the axis of the sample. In this case, the degree of consolidation was determined on the basis of the uniaxial deformation of the sample. The parabolic distribution was based on the values obtained from the solution of the series, where the consolidation coefficient was determined on the basis of the distribution of the pore water pressure. Distinguishing from the entire course of the consolidation process, the duration of the quasi-filtration consolidation phase can be performed by analyzing the variability of the relation $c_v$ – $U$. Based on many results of consolidation studies in the course of the consolidation described by the function $c_v$ – $U$, three ranges with a different nature of changes in the consolidation coefficient can be identified. In Figure 3a an example of a characteristic course of the semi-logarithmic relation $c_v$ – $U$ was shown. The variability in the initial phase of consolidation is determined by the moment of applying a load to the sample of saturated soil (in terms of deformations, this is the initial compression). At this stage, the consolidation coefficient demonstrates the highest values. Then the values of the consolidation coefficient decrease together with the increase in the consolidation degree $U$ and stabilize to a quasi-linear character (slight fluctuations in the course of the $f = c_v$ – $U$ function are observed in this phase). Before the stabilization is recorded, there is a bound which is noted both for the course of the pore pressure dissipation and for the uniaxial strain. Stabilized values can be defined as quasi-constant values of the consolidation factor $c^q_{v,i}$. The stabilization confirms that the assumptions (i) and (ii) are fulfilled, and the limit value $c^q_{v,i}$ determines the end of the quasi-filtration phase, e.g., $U_{EOP}$. At a later stage, the consolidation coefficient value is characterized by lower values, depending on the size of the impact of the rheological mechanisms. If the limit values $c^q_{v,i}$ for the interval are known, the geometric mean of these values can be calculated as follows:

$$\prod c^q_v = \sqrt[n]{a_1 \cdot a_2 \cdot \dots \cdot a_n} \qquad (16)$$

where $a_1$, $a_2$, $a_n$ are the following values of the consolidation coefficient from the set of $n$ values for the quasi-filtration phase. The value of the consolidation coefficient calculated this way is independent of a single measurement point and represents the consolidation behaviour for a significant settlement progress.

For example, a sample of a natural clay from Krakowiec with an initial height of 24.3 mm was consolidated at a load of 400 kPa. In the first step of the analysis, a quasi-linear part of the log $c_v$ – $U$ plot was identified and the average value of the quasi-filtration coefficient of the consolidation was calculated according to the formula (16). In order to check the compatibility of the theoretical solution, a curve was constructed and compared to the curve obtained from the test (Fig. 3b). Using the optimization method, the lowest value of the $d_n$ parameter was determined, together with the corresponding coefficient of consolidation value (Fig. 3c).

These values were used to determine the compliance between the best model solution and the quasi-constant approach. The best-fitting model curve was obtained using the coefficient of consolidation $c_v$ = 2.60 x10$^{-8}$ m$^2$/s, with the conformity assessed using the parameter $d_n$ = 0.000889. The average value of the quasi-filtration consolidation ratio was $c^q_v$ = 2.59 x 10$^{-8}$ m$^2$/s and the $d_n$ parameter was 0.000893. Note that the values of the $d_n$ parameter ranged from 0.0009 to 0.0001, indicating a very good “fit” between the measured and the theoretical data. As the results show, both approaches are in very good agreement.

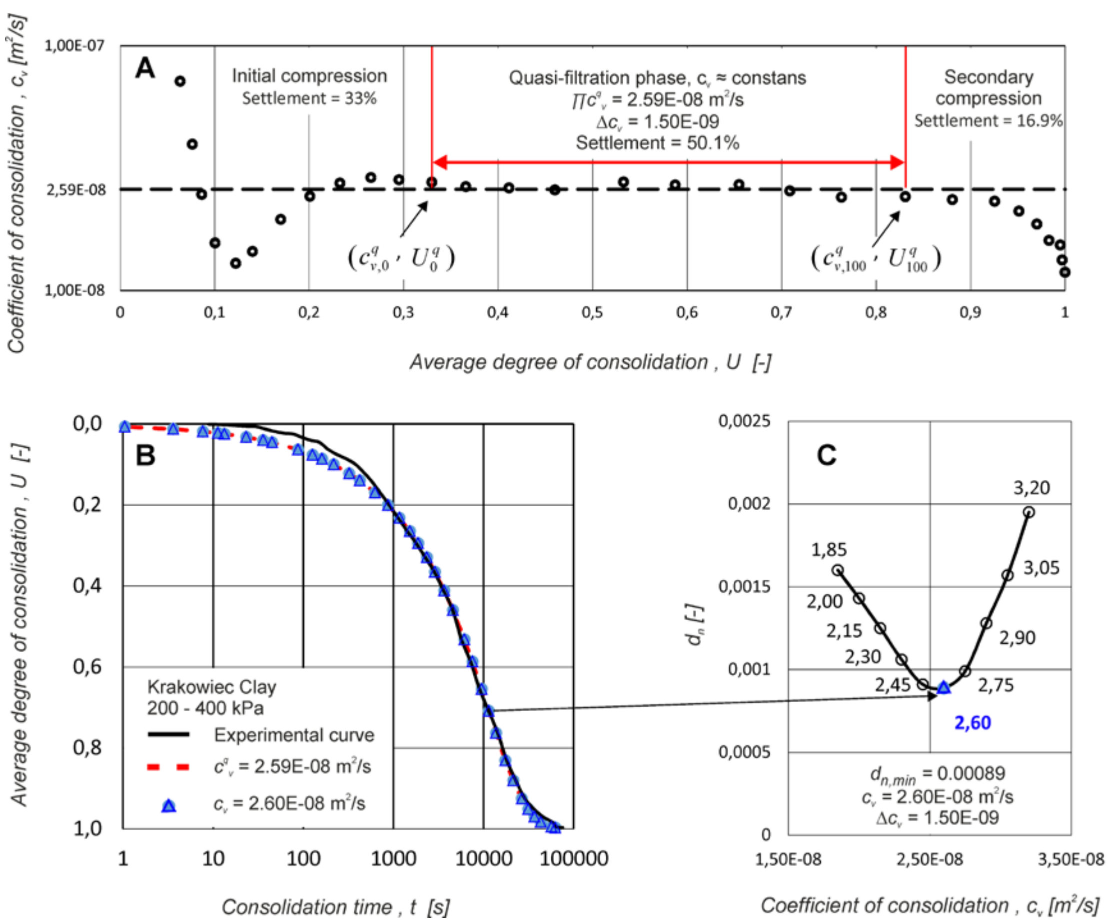



**Figure 3**. Experimental results showing the variation of the coefficient of consolidation with the degree of consolidation referring to the optimization procedure: A – coefficient of consolidation versus degree of consolidation, B – comparison of the experimental data and theoretical best-fit solution; C – relationship between $d_n$ parameter and coefficient of consolidation $c_v$.

## 5 SOILS AND TESTING PROCEDURES

To illustrate the importance of assessing the validity of the optimization method and the quasi-constant approach and to determine the consolidation parameters, an extensive testing programme was established. In this study the behaviour of the reconstituted clay as well as the behaviour of clay–sand mixture and the natural organic mud soil were the focus of interest. Three soils, namely, Krakowiec clay (designated as C), 50 % clay – 50 % sand mixture (designated as S), and organic silty clay as per ISO Soil Classification (PN-EN ISO 14688-2), [29] were used for the study. Washed fine river sand was used for the mixture (passing a 0.2 mm sieve and retained on a 0.063 sieve).The physical properties of these soils are listed in Table. 1. The clay suspension was made of Krakowiec clay from a deposit near the village of Chmielów, in the Podkarpackie voivodeship, Poland. Based on a chemical analysis for aluminosilicate refractory products and raw materials in accordance with ISO 12677-2: 2007, [30], which was an alternative method to the X-ray fluorescence (XRF), the chemical composition of the clay used in this study was determined. Table 3 shows the tested chemical compounds. Additionally, a qualitative phase analysis was carried out using the X-ray diffraction method and quartz, calcite, dolomite, albite, siderite, illite, muscovite, kaolinite and microcline were found.

After drying the Krakowiec clay at a temperature of 106 degrees, it was separated into lumps and wiped through a sieve with a sieve diameter of 0.0063 with the use of distilled water. The suspension was left for one week until the clay fraction sedimented. Then, the clarified water was removed from the surface, and the clay fraction was dried. The pastes with plasticity approaching the liquid limit ($w_n \approx LL$) were made from dried and mortar-ground soil mass combined with distilled water to form a uniform texture. The first four IL-type tests with the measurements of the pore-water pressure were made according to the following path of load increments: 50, 100, 200, 400 kPa. The fifth and sixth studies were carried out for a different path of load increment: 25, 50, 75, 100, 125 kPa. The pore pressure was measured centrally on the lower surface of the specimen along the

**Table 2**. Physical parameters of soils utilized in the present study.

| Soil | Particle size | | | Atterberg limits | | Plasticity index | Organic matter | Specific gravity |
|---|---|---|---|---|---|---|---|---|
| | Sand [%] | Silt [%] | Clay [%] | Liquid limit [%] | Plastic limit [%] | $I_p$ [%] | OM [%] | $G_s$ [-] |
| Krakowiec clay | 14 | 48 | 38 | 65.02 | 24.60 | 40.42 | - | 2.72 |
| Organic silty clay | 2 | 56 | 42 | 109.78 | 54.22 | 55.56 | 11.33 | 2.62 |

**Table 3**. Chemical composition of Krakowiec Clay.

| | Chemical compound | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | $SiO_2$ | $Al_2O_3$ | $Fe_2O_3$ | CaO | MgO | $K_2O$ | $Na_2O$ | $TiO_2$ |
| % content | 10.31 | 59.34 | 14.17 | 5.39 | 5.40 | 2.13 | 2.47 | 0.79 |

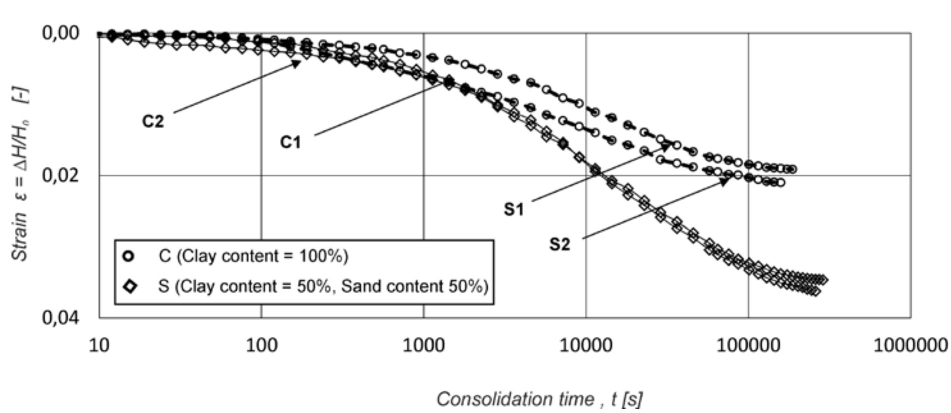



**Figure 4**. Strain–time relationships for samples with different clay contents.

impermeable base of the cell. The consolidation cell used a fixed-type ring setup with single drainage on the top end of the specimen. The paste samples were prepared in a consolidation ring with a diameter of 75 mm and a height of 30 mm. The strain-time curves obtained in the studies for the first four tests are shown in Figure 4.

In general, a similar compression behaviour was found for the two types of samples. Samples with sand content revealed greater stiffness compared to the clay samples, which is reflected in the smaller recorded settlement. It is interesting to note that despite the preparation of a homogeneous clay paste with the addition of sand, the same curves were not obtained. The biggest differences were visible in the advanced stage of consolidation, where for sample S1 the deformations proceeded slightly more slowly than for sample S2.

## 6 VALIDATION ANALYSIS

In this section the previously developed solutions were applied to the interpretation of the consolidation tests. Then the results of the analyses were discussed, and the importance of optimisation was highlighted with an example. The comparison of the two classic curve-fitting methods for the coefficient of consolidation with those developed in this study were also presented.

### 6.1 Interpretation of the consolidation test

Based on the $U$ – $T_v$ relationship for the initial constant, positive pore-water pressure, Sridharan et al. [20] noted that it was possible to generate theoretical curves $\log_{10}(H^2/t_{theor})$ – $U$. This approach assumes applying the experimental data on one graph, represented by measuring coordinates relative to the model curves $\log_{10}(H^2/t_{theor})$ – $U$, plotted on the basis of known values of the consolidation coefficient and the corresponding theoretical time. The consolidation coefficients used to construct the theoretical curves $\log_{10}(H^2/t_{theor})$ – $U$ were obtained on the basis of the procedure described in Section 3. The combination of the experimental curves $\log_{10}(H^2/t_{texp})$ – $U$ versus the background of the model curves $\log_{10}(H^2/t_{theor})$ – $U$, allows a direct comparison of the obtained values of the consolidation factor (Fig 5).

The quasi-filtration phase was established on the basis of the log $c_v$ – $U$ relationship. Two interpretation assumptions were made when interpreting the results of the research. The first one assumes the criterion of convergence of the uniaxial strains and the pore-pressure dissipation. The second assumes a coincidence between the experimental data and the corresponding theoretical solution. In the case of a paste made of clay without any admixture of sand, the consolidation behaviours do vary. Despite a similar course of uniaxial deformation curves, the filtration nature of the consolidation process was ambiguous. For sample C1, there was a clear delay in the pore pressure dissipation. The consolidation factor calculated on the basis of the pore pressure dissipation was significantly lower than that determined on the basis of the deformation. In turn, for sample C2, the opposite behaviour was observed: the excess pore water pressure dissipation proceeded faster than the deformation. It should be noted that we are dealing here with two different factors that drive the consolidation process. One can be described as filtration and the other as a rheological factor. A qualitative assessment of the dominance of one factor over another can be performed using the param-

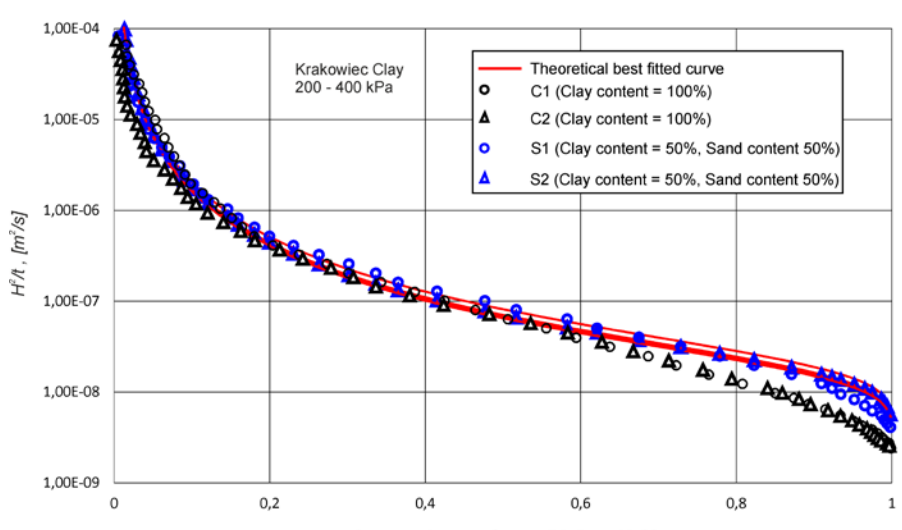



**Figure 5**. Family of the $\log_{10}(H^2/t_{texp})$ – $U$ curves collocated with the $\log_{10}(H^2/t_{theor})$ – $U$ curves.

eter $\eta$ developed by Dobak and Gaszyński [31]. This parameter is determined using two values of the coefficient of consolidation calculated on the basis of excess pore water pressure dissipation and strain in accordance with the formula:

$$\eta = \frac{c_{v,\varepsilon} - c_{v,u}}{c_{v,u}} \qquad (17)$$

where $c_{v,\varepsilon}$ , $c_{v,u}$ are consolidation coefficients calculated on the basis of deformation and pore water pressure dissipation, respectively.

On the basis of the $U - t$ diagrams and the $d_n$ parameter, the range of consolidation was determined and expressed as a percentage of the quasi-filtration phase in relation to the entire process, for which the compliance of the applied theoretical model with respect to the obtained experimental data was the largest. Figures 6a and 6b show the consolidation behaviour for the reconstituted clay paste. For samples C1 and C2, the percentage of compliance was 37% and 34%, respectively. The 3% difference was based on the greater extent of the initial compression after applying the load for sample C2. However, the effect of the initial compression caused a very slight deviation in relation to the theoretical solution. Both samples showed very similar susceptibility to the rheological effects, with the visible part of the secondary consolidation, which starts close to $U$ = 55%. The adopted scheme for the implementation of the experiments assumed the possibility of extending the relative scope of the quasi-filtration consolidation through the application of a sand fraction. The results of the tests carried out on samples of sand and clay pastes showed that the content of the sand fraction caused a significant reduction in the differentiation between the courses of the pore pressure dissipation and the uniaxial strain curves (Fig. 6c). On the basis of the presented graphs, a very slight predominance of settlement progress was observed in comparison to the dissipation excess

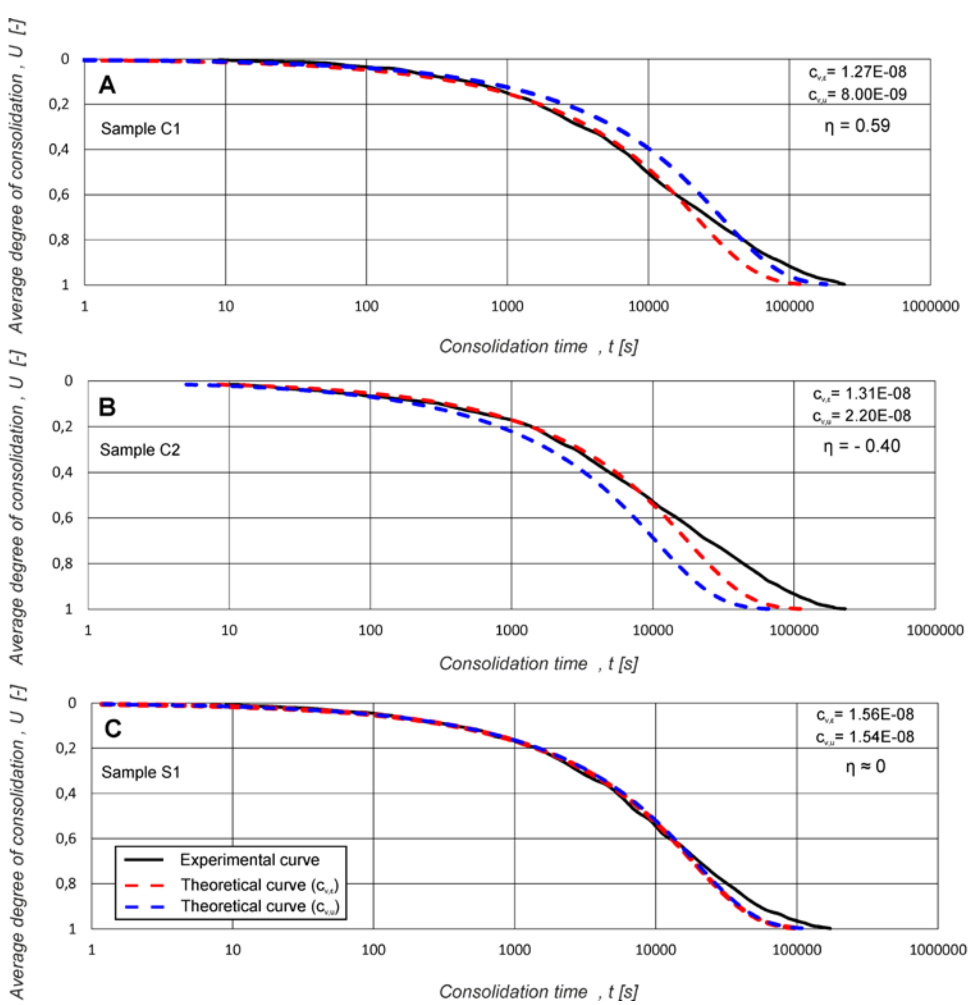



**Figure 6**. Consolidation behaviour of reconstituted clay paste with and without additional sand content: A) predominance of the rheological factor over the filtration factor; B) predominance of the filtration factor over the rheological factor; and C) similar course of consolidation in terms of filtration and creep factors.

pore water pressure. The assumed presence of the sand fraction influenced the extension of the quasi-filtration phase and the reduction of the secondary consolidation phase, which can be seen in the $U-t$, $\log_{10}(H^2/t)-U$ and $\log c_v - U$ diagrams. The range of the separated phase is in the highest compliance with the theoretical model in the case of the S1 and S2 samples, i.e., 70% and 69%, respectively. Table 3 presents the consolidation parameters obtained from the current analysis.

**Table 3**. Results of consolidation parameters' interpretation obtained with the Quasi- constant, Optimisation and Casagrande methods.

| | $c_v$ [m$^2$/s] | | | | Course parameters | |
|---|---|---|---|---|---|---|
| Sample | Strain *q-c* | Pore pressure *q-c* | Optimisation | Log *t* | $\eta$ | $d_{n,min}$ |
| S1 | $1.56^{-08}$ | $1.54^{-08}$ | $1.56^{-08}$ | $1.82^{-08}$ | 0.033 | 0.0032 |
| S2 | $1.60^{-08}$ | $1.49^{-08}$ | $1.61^{-08}$ | $1.85^{-08}$ | 0.073 | 0.0030 |
| C1 | $1.27^{-08}$ | $8.00^{-09}$ | $1.28^{-08}$ | $1.46^{-08}$ | 0.59 | 0.0038 |
| C2 | $1.31^{-08}$ | $2.20^{-08}$ | $1.31^{-08}$ | $1.36^{-08}$ | -0.40 | 0.0034 |

## 6.2 Comparison of the Optimisation, Quasi-constant, Taylor, and Casagrande methods in terms of $c_v$ and $d_n$

The selected experimental consolidation courses together with the best-fitting model curve obtained for a reconstituted clay (study no 5) and for an organic soil (study no. 6) are shown in Figures 7 and 8, respectively. For each of the load increments the experimental $U-T_v$ data was plotted against the theoretical curve from which the $d_n$ parameter was calculated. Both example sets of $U-T_v$ curves demonstrate the high quality of the fit associated with the $d_n$ parameter, irrespective of the physical properties of the tested soils. It can be observed that the secondary consolidation essentially starts around $U = 60\%$ for the clay samples and $U = 40–60$ % for the organic soil samples. It indicates that the inves-

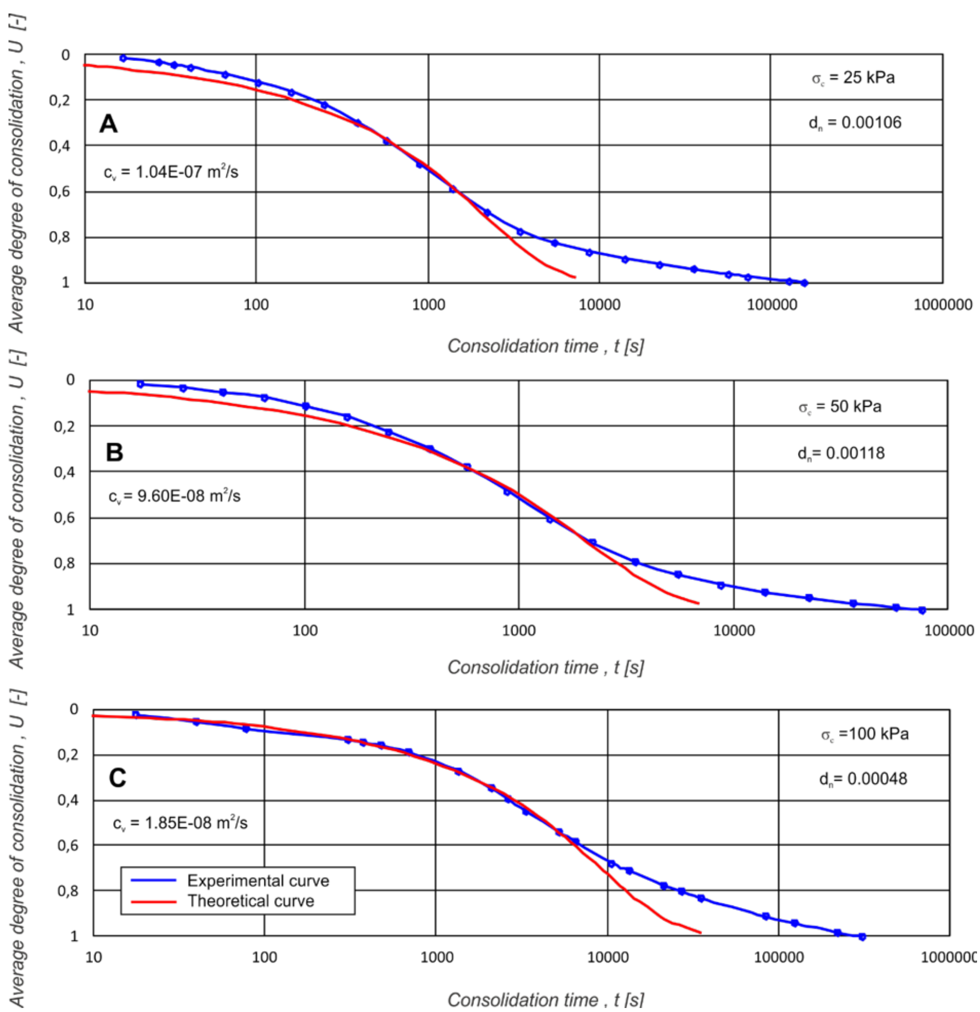



**Figure 7**. Experimental consolidation courses versus the best-fitting model curves for reconstituted clay.

tigated organic soil is prone to significant secondary deformations. It was found by the optimization method that Terzaghi's consolidation model is able to capture a slight range of the total deformation. This is mainly due to the postulation that the consolidation process is regarded as purely filtration [32]. Figs 7–8 show that the greater is the discrepancy between the experimental and theoretical curves, the greater is the presence of secondary consolidation.

The coefficient of consolidation for the clay sample computed using Eq. (6) and (7) and those obtained using the Taylor ($t^{1/2}$ ) and Casagrande (log $t$) methods with reference to the $d_n$ parameter are shown in Figure 9. The optimal $c_v$ value of each curve was determined based on the lowest value of the $d_n$ parameter. This value represented the best agreement between the experimental and theoretical curves. The results of the analysis for clay and organic soil in Tables 4–5 showed that the accuracy of the determined $c_v$ with the quasi-constant method in relation to the best analytical solution increases together with the rise of the consolidation load. The $c_v^q$ values were slightly higher than those determined on the basis of the optimization. However, the largest discrepancies were observed for loads of 25 and 50 kPa.

Nevertheless, both methods are characterized by good compliance and the $c_v$ values correspond with each other. Using the optimization method, the value of $c_v$ changed, which refers to the distance from the theoretical curves imposed on the experimental curve, which should be chosen very carefully. In turn, in the quasi-constant method, a very precise distinction of the quasi-filtration phase for which the value of $c_v$ will be calculated is crucial. Making mistakes at this stage of the analysis could result either in an inadequate shape of the $d_n$ – $c_v$ curve as well as in a lack of assumed linearity of part of the $c_v$ – $U$ curve. The lowest values of the $d_n$ parameter were obtained for the optimization method and the quasi-constant method. The highest values were

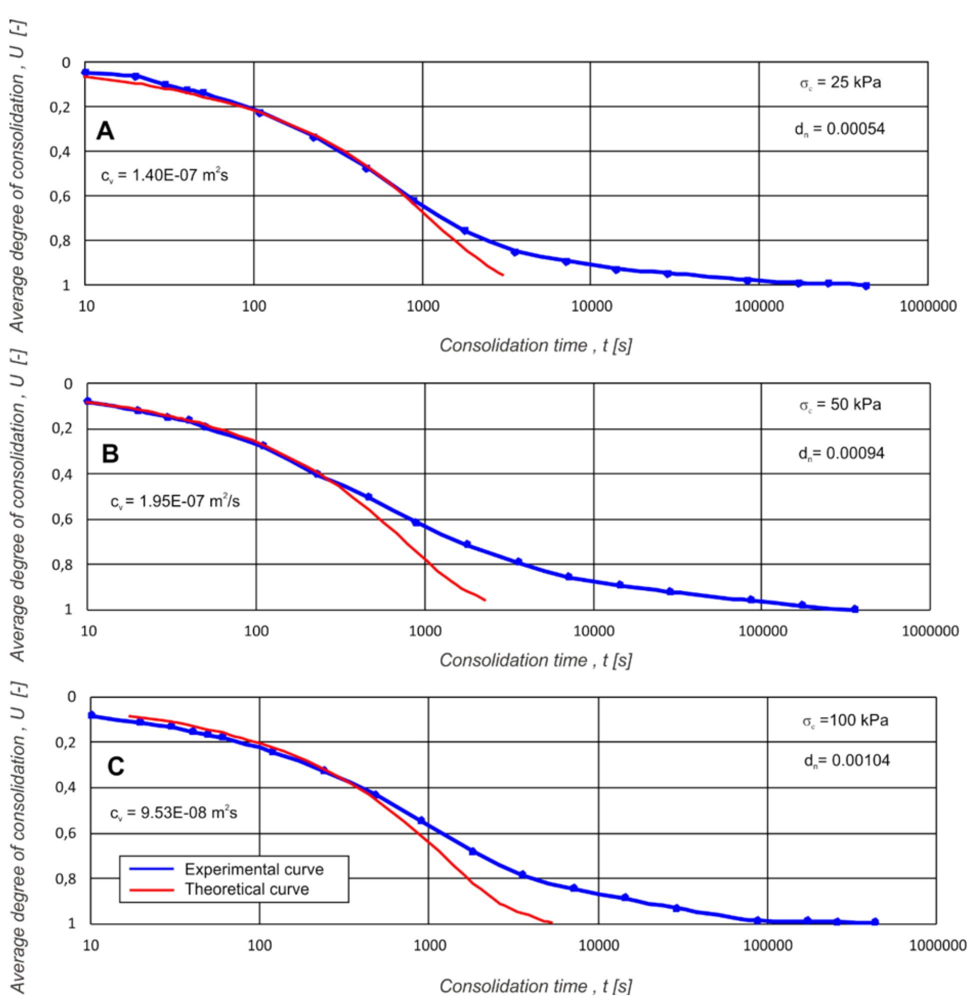



**Figure 8**. Experimental consolidation courses versus the best-fitting model curves for organic soil.

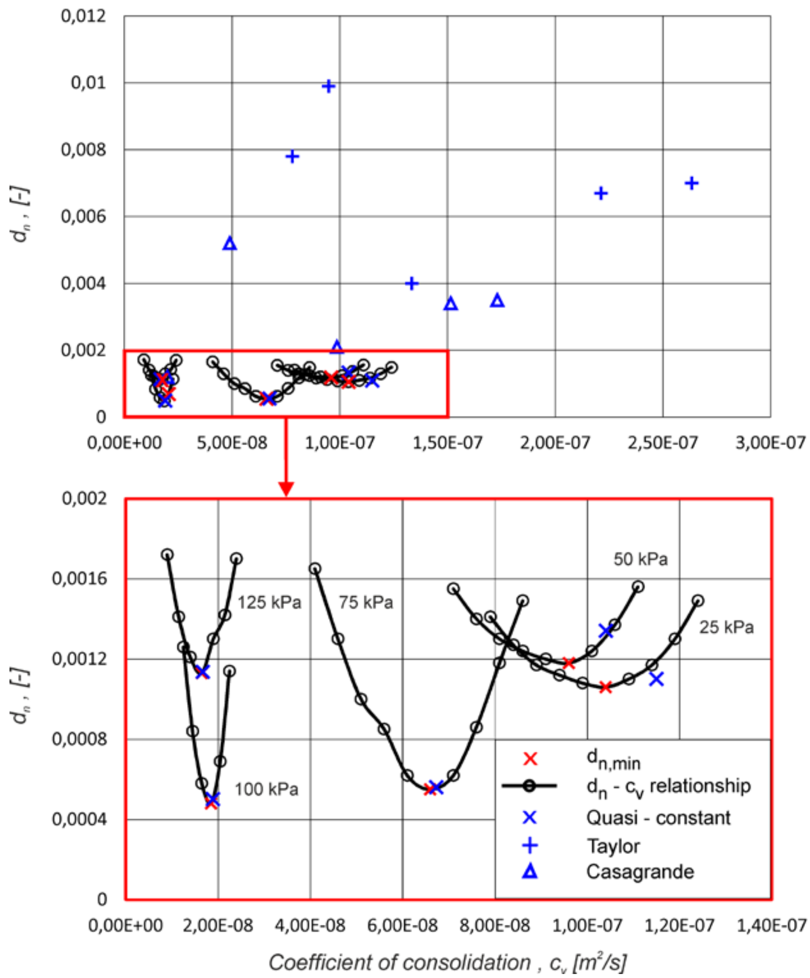



**Figure 9**. Coefficient of consolidation for all increments by various methods with reference to the changes of the dn parameter. The optimal values of the coefficient of consolidation related to the lowest $d_n$ parameter are marked with red crosses.

**Table 4**. Consolidation parameters for the reconstituted clay obtained from the interpretation of the consolidation tests using the quasi-constant approach, the optimization method, the log($t$) method the and $t^{1/2}$method.

| Load | | Quasi-constant Method | Optimization Method | log($t$) Method | $t^{1/2}$ Method |
|---|---|---|---|---|---|
| 25 | $c_v \times 10^{-8}$ m²/s | 11.5 | 10.4 | 17.3 | 26.3 |
| | $d_{n,min}$ | 0.0011 | 0.0010 | 0.0035 | 0.0070 |
| | $U_{EOP}$ % | 69.1 | 69.3 | - | - |
| 50 | $c_v \times 10^{-8}$ m²/s | 10.4 | 9.60 | 15.1 | 22.1 |
| | $d_{n,min}$ | 0.0013 | 0.0011 | 0.0034 | 0.0067 |
| | $U_{EOP}$ % | 70.7 | 72.1 | - | - |
| 75 | $c_v \times 10^{-8}$ m²/s | 6.73 | 6.60 | 9.86 | 13.3 |
| | $d_{n,min}$ | 0.0005 | 0.0005 | 0.0021 | 0.0040 |
| | $U_{EOP}$ % | 61.0 | 62.0 | - | - |
| 100 | $c_v \times 10^{-8}$ m²/s | 1.89 | 1.85 | 4.88 | 9.47 |
| | $d_{n,min}$ | 0.0005 | 0.0004 | 0.0052 | 0.0099 |
| | $U_{EOP}$ % | 58.0 | 59.8 | - | - |
| 125 | $c_v \times 10^{-8}$ m²/s | 1.67 | 1.65 | 2.0 | 7.8 |
| | $d_{n,min}$ | 0.0011 | 0.0011 | 0.0012 | 0.0078 |
| | $U_{EOP}$ % | 51.0 | 51.0 | - | - |

**Table 5**. Consolidation parameters for the organic soil obtained from an interpretation of the consolidation tests using the quasi-constant approach, the optimization method, the log($t$) method the and $t^{1/2}$method.

| Load | | Quasi-constant Method | Optimization Method | log($t$) Method | $t^{1/2}$ Method |
|---|---|---|---|---|---|
| 25 | $c_v$ ×10$^{-8}$ m$^2$/s | 11.39 | 11.40 | 11.60 | 8.40 |
| | $d_{n,min}$ | 0.0011 | 0.00054 | 0.0035 | 0.0070 |
| | $U_{EOP}$ % | 61.0 | 61.0 | - | - |
| 50 | $c_v$ ×10$^{-8}$ m$^2$/s | 18.90 | 19.50 | 24.00 | 17.00 |
| | $d_{n,min}$ | 0.0013 | 0.00093 | 0.0034 | 0.0067 |
| | $U_{EOP}$ % | 70.7 | 40.0 | - | - |
| 75 | $c_v$ ×10$^{-8}$ m$^2$/s | 6.73 | 13.00 | 15.50 | 10.50 |
| | $d_{n,min}$ | 0.0005 | 0.0005 | 0.0021 | 0.0040 |
| | $U_{EOP}$ % | 61.0 | 62.0 | - | - |
| 100 | $c_v$ ×10$^{-8}$ m$^2$/s | 9.66 | 9.53 | 9.89 | 8.00 |
| | $d_{n,min}$ | 0.0005 | 0.0010 | 0.0052 | 0.0099 |
| | $U_{EOP}$ % | 58.0 | 43.0 | - | - |
| 125 | $c_v$ ×10$^{-8}$ m$^2$/s | 8.48 | 9.50 | 9.21 | 7.50 |
| | $d_{n,min}$ | 0.0011 | 0.0011 | 0.0012 | 0.0078 |
| | $U_{EOP}$ % | 51.0 | 51.0 | - | - |

obtained with the $t^{1/2}$ method, indicating a significant discrepancy between the laboratory measurements and the theoretical fitting. For individual load levels, the conformity of the calculated $c_v$ for the three methods was obtained only in one case. For the load of 125 kPa on the basis of the optimization method, the quasi-constant and log($t$) methods, the $d_{n,min}$ values were calculated as 0.0011, 0.0011 and 0.0012, respectively. Similar results were obtained in the case of the organic soil.

The $c_v$ values of the reconstituted clay obtained using the log $t$ method and the $t^{1/2}$ method were significantly higher than those determined on the basis of the optimization and the quasi-constant approaches. The differences between those two methods and the optimisation method are discussed using the obtained ratios of the $c_v$ values. This method is often adopted in geotechnical practice and was used, among others, in the works of Sridharan and Prakash [6], Robinson [8], Al - Zoubi [17] and Cortellazzo [33]. The first and second ratios compare the $c_v$ values determined using log $t$ and $t^{1/2}$ methods with those determined using the optimisation method. The $c_v$ values determined using the log $t$ and $t^{1/2}$ methods were approximately 1.5 to 2.7 and 2 to 5 times higher than those obtained with the optimization method, respectively. The third relation compared the log $t$ and $t^{1/2}$ methods and was calculated as from 1.5 to 4. This regularity is confirmed by previous analyses carried out for various clay soils by Sridharan et al. [20], Feng and Lee [27], Chan [34] and Shukla et al. [35]. In the case of the organic soil the $c_v$ values obtained using the log $t$ method were significantly higher and using the $t^{1/2}$ method were significantly lower than those determined on the basis of the optimization and the quasi-constant approaches. The $c_v$ values determined using the log $t$ method were approximately 1.0 to 1.3 times higher than those obtained from the optimization method. The $c_v$ values determined by the $t^{1/2}$ method were approximately 0.6 to 0.9 times lower than those obtained from the optimization method. The ratio compares the $c_v$ values determined using the log $t$ method with those determined using the $t^{1/2}$ method, which were always higher than 1.0 and lower than 2.0.

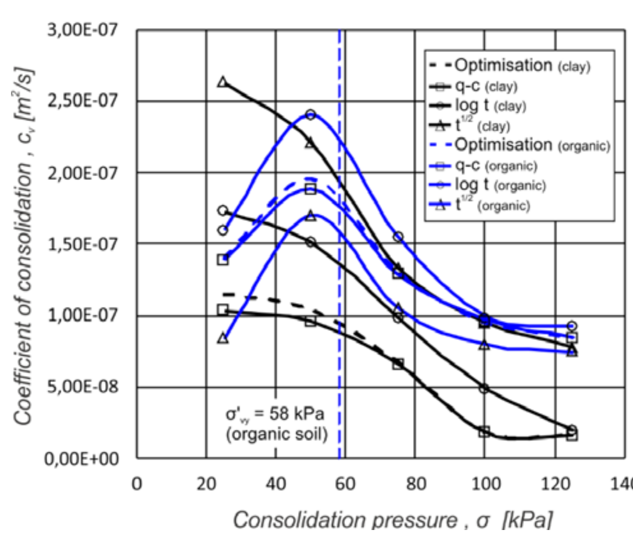



**Figure 10**. Comparison of $c_v$ – $\sigma_c$ curves obtained for different load steps.

Figure 10 illustrates the values of the coefficient of consolidation obtained for different load steps. A downward trend of the $c_v$–$\sigma'_c$ relationship was observed in the case of the reconstituted clay for all four methods and was the largest for the $t^{1/2}$ method. The shapes of the calculated $c_v$–$\sigma$ curves for the organic soil were generally similar. It is also evident from Fig. 10 that a drastic decrease in the $c_v$–$\sigma$ curve appeared near the vertical yield stress $\sigma'_{vy}$.

## 7 CONCLUSIONS

Mathematical modelling, including a comparison of the experimental data with the sets of theoretical solutions, is a promising interpretation approach in consolidation studies. Terzaghi's consolidation theory does not take into account both the initial and secondary effects, hence the $c_v$ values are dependent upon the theoretical solution and refer to the primary consolidation only. Analytical tools made it possible to determine the coefficient of consolidation $c_v$ with the smallest value of the statistical $d_n$ parameter that led to the best fitting of the laboratory data. In this study the $d_n$ parameter was identified as an error function between the experimental and theoretical solutions. The optimization method based on the process of minimizing this function can be implemented in computer spreadsheet programs that are commonly used in various geotechnical applications. Furthermore, the $d_n$ error calculated between the experimental and theoretical degree of consolidation was generally quite low, and always less than the error associated with the log $t$ and $t^{1/2}$ $c_v$ values. The optimization method was also used to assess the reliability of the results of the quasi-constant method. Using the log $c_v$ – $U$ relationship, the variability of the coefficient of consolidation in relation to the entire experimental course of consolidation was examined. The analysis of the relationship between the coefficient of consolidation and the degree of consolidation showed the presence of a region with semi-established $c_v$ values.

Based on the results of oedometer tests on various soils, the $c_v$ values estimated by the quasi-constant approach were in good agreement with those obtained from the optimization method. The coefficient of consolidation determined by the graphic methods, e.g., log $t$ and $t^{1/2}$, is highly variable, due to the assumption of different reference points on the experimental curve. In the case of reconstituted clay the $t^{1/2}$ method gave higher $c_v$ values and higher $d_n$ values than those obtained from both of the presented methods and the log $t$ method. In the case of the organic soil the $t^{1/2}$ method gave lower $c_v$ values than those obtained from both the presented methods and the log $t$ method. In general, the values of $c_v$ calculated using the log $t$ method were greater than those determined using other methods.

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