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Research paper

Some remarks on the pore water pressure dissipation patterns from the one-dimensional consolidation test

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Abstract: The pattern of pore water pressure dissipation from the one-dimensional consolidation test significantly affects the calculated value of the coefficient of consolidation. This paper discusses the interpretation methodology for laboratory dissipation data from the oedometer test with the pore water pressure measurements or Rowe cell test. In the analysis, the gradient-based algorithm for finding the optimal value of the coefficient of consolidation is used against experimental results, obtained for various fine-grained soils. The appropriate value of coefficient of consolidation is considered as one with the lowest associated error function, which evaluates fitness between the experimental and theoretical dissipation curves. Based on the experimental results, two different patterns of the pore water pressure dissipation are identified, and the saturation of the specimen was found to be the key factor in describing the change in the patterns. For the monotonically decreasing dissipation curve, an inflection point is identified. The values of degree of dissipation at the inflection point are close to the theoretical value of 53.4%.

Keywords: clay, consolidation, dissipation, excess pore water pressure, saturation

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1. Introduction

It has long been known that any construction of civil engineering structures founded on soft, fine-grained soils required consideration of the consolidation behaviour of the deposit. The process of gradual compression of saturated cohesive soils under applied loads, which must entail an expulsion of the water in the pore space and compaction of the soil skeleton, is termed consolidation. The consolidation process is a combination of two phenomena. The first one is permeability, which governs the rate at which water is removed from the pore space (and thus the rate of the settlement at any time). The second one is compressibility, which controls the evolution of the distribution of excess pore-water pressure (and thus the duration of the consolidation process). As stated by Leroueil [1], soil behaviour in the one-dimensional condition is predominantly influenced by strain rate, temperature, soil disturbance, the stress path followed, time-dependency, destructuration and bonding. Terzaghi's theory of consolidation is an essential and broadly used model in the evaluation of consolidation properties of fine-grained soils (such as clay, silt and loam, organic soil and dredged sludge). The coefficient of consolidation (c_v) is a key parameter that must be determined to accurately predict the settlement rate of a structure founded on the soil. The c_v values are commonly calculated from a one-dimensional consolidation test using compression or excess pore water pressure data. Shukla et al. [2] reviewed the state-of-the-art to determine c_v and discussed the available methods to evaluate the consolidation test results. Most methods referred to either as "conventional" or "alternative" methods utilize the characteristic features of average degree of consolidation (U_v) versus time factor (T_v) relationship from the Terzaghi consolidation theory to the observed compression versus time data. U_v is directly proportional to the percentage of consolidation and commonly is defined by axial strain or compression. In turn, T_v is a non-dimensional parameter that relates c_v, time and the drainage length (depth of soil). In terms of deformation value of U_v is the amount of consolidation at a given time within a soil mass to the total amount of consolidation obtainable under a given stress condition. The time required to reach any percentage of consolidation for any thickness of a particular soil layer can be evaluated from the laboratory consolidation curve [3]. This time for any consolidation percentage is a function of the square of the thickness of a particular soil layer and its permeability under specific consolidation pressure. The dimensionless time factor is frequently used to generalise the one-dimensional consolidation equation in the consolidation analysis. The variation of U_v with T_v is represented in the graphical solution [4] and may also be approximately computed using the simple relations [5]. $U_v - T_v$ relationship constitutes the basis of most conventional methods for determining c_v. For example, the Taylor's root-time method uses t_{90} time corresponding to 90% of consolidation to calculate c_v, while Mesri's inflection point method uses t_{70} time corresponding to 70% consolidation. The biased judgement in identifying the points on the consolidation curve causes discrepancies in estimating c_v. From the practical point of view, each 'conventional method' does not give unambiguous and reliable results and is even difficult to apply due to the presence of initial and secondary compression and/or data scattering [6]. Several full-matching approaches have been proposed and verified based on the experimental studies [7–11] to overcome reported issues. In the full-matching approaches all the recorded measurements are utilised and reflects the whole consolidation process. The advantage of this approach is obtaining more representative c_v values and minimizing the degree of subjectivity. Apart from the methods based on the measurement of the vertical deformation of the sample, there are several methods based on the analysis of the pore water pressure dissipation conditions. Conventionally pore water pressure (u_b) is measured at the base of the consolidation cell. Determination of the c_v value in this way can be done using a modified oedometer with the ability to measure pore water pressure or Rowe cell [12]. Robinson [13] proposed a method that utilises the uniqueness of excess pore water pressure and compression and the linearity concept of consolidation characteristics for determining c_v. Furthermore, Robinson and Soundara [14] estimated the c_v using the relation between the degree of dissipation (U_{ub}) and the time factor. The time factor corresponding to U_{ub} = 50% was obtained as 0.379. The time corresponding to U_{ub} = 50% from an experimental plot of U_{ub} versus time was used to calculate the c_v. A similar method based on the mid-plane dissipation point can be found in the Head's textbook [15]. Vinod and Sridharan [16] extended the conventional Asaoka method [17] to evaluate c_v and the end-of-primary (EOP) consolidation from the pore water pressure data. In turn, full-matching approaches for laboratory measured pore water pressure were developed by Dobak and Gaszyński [18], Olek [19] and Chow et al. [20].

This paper elaborates on different pore water pressure dissipation patterns of the various fine-grained soils based on the experimental observations. For this purpose, tests on intact and reconstituted soils were conducted to better understand the soil behaviour. Furthermore, special attention was paid to the shape of the dissipation curves and their consequences on the interpretation of the test. Particular attention during laboratory investigation was paid to the sufficient saturation of the sample, using increases in back pressure. The main objective of the present investigation is to relate c_v to the permeability of the soil rather than to its compressibility. Considering the limitations and potential difficulties in the correct interpretation of the laboratory dissipation data, a simple non-graphical method to determine c_v based on the measured pore pressures has been incorporated in this work. In this way, consolidation curves were numerically modelled using a gradient-based algorithm to find the optimal value of c_v.

2. Theoretical considerations

A one-dimensional differential equation that governs the consolidation and pore water pressure dissipation process is expressed as follows [4]:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \tag{2.1}$$

where: u – excess pore water pressure, t – time, z – distance from the top of the sample subjected to consolidation.

According to Terzaghi's theory, the relationship between degree of dissipation U_{ub} and time factor T_v, can be derived as:

$$U_{ub} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M} \sin M \exp\left(-M^2 T_v\right) \tag{2.2}$$

where: M – $(2m + 1)\pi/2$, m – integer, T_v – time factor $T_v = c_v t/H^2$, H – length of the drainage path.

Note that, Eq. (2.2) is valid as long as c_v is assumed constant throughout the whole dissipation process. Figure 1 presents the theoretical description of the dissipation process in terms of dimensionless parameters. The relationship between U_{ub} and T_v as well as relationship between theoretical slope (M) and T_v are depicted. Terzaghi's U_{ub} versus $\log T_v$ curve describes the consolidation progress and shows that the actual time required for any given portion of the consolidation process to occur at any point within the sample is inversely dependent on the c_v and directly dependent on the stratum depth squared. This curve has an inflection point at which the sense of concavity of the curve changes. When the slope of the $U_{ub}/\log T_v$ curve, $M = dU_{ub}/d \log T_v$ is plotted against $\log T_v$, the inflection point of maximum slope, M_i, is found at $U_{ub} = 53.4\%$ and $T_v = T_{vi} = 0.408$ with $M_i = 1.077$.

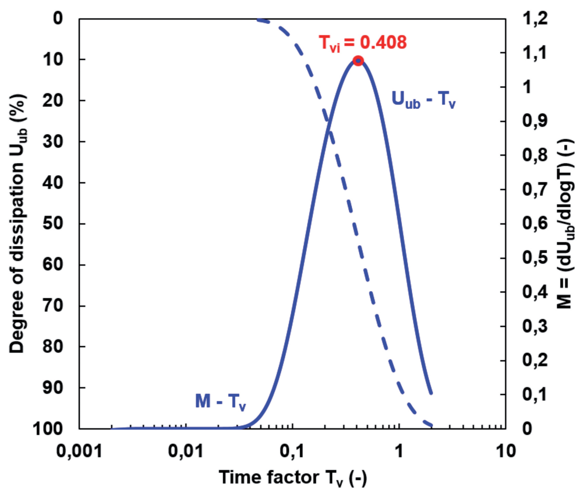

Fig. 1. Theoretical relationships according Terzaghi theory of consolidation (Note: the dashed line indicates relationship between U_{ub} and T_v and the solid line indicates relationship between M and T_v)

These findings can be used to assess experimental results against the theory. When u_b is measured at the base of the soil sample under consolidation with drainage only on top of the sample, the expression for an experimental degree of dissipation is as follows:

$$U_{ub,i} = \frac{u_0 - u_i}{u_0} \tag{2.3}$$

where: u_0 – excess pore water pressure at the initial stage of consolidation, u_i – excess pore water pressure at time t.

The value of c_v can be directly obtained based on the inverse modelling procedure in accordance with Fig. 2. As stated by Jin et al. [21], the aim of the inverse modelling is to find values for the model parameters that provide the best attainable fit between model predictions and corresponding observations. Using the inverse analysis, a given model is calibrated by iteratively changing input values until the simulated output values match the observed data [22]. Given that this is a mono-objective problem, c_v with the lowest error function was selected and was considered the optimal value for experimental results. In this study, a fitness (error) function under mono-objective framework with one criterion was considered:

$$Error\,(x) = \frac{\sum \frac{\left|U_{n,i} - U^*_{n,i}\right|}{U_{n,i}}}{\sum w_{n,i}} \tag{2.4}$$

where: $U_{n,i}$ – experimental degree of consolidation, $U^*_{n,i}$ – theoretical degree of consolidation, $w_{n,i}$ – range around each theoretical point $U^*_{n,i}$ characterising the divergence.

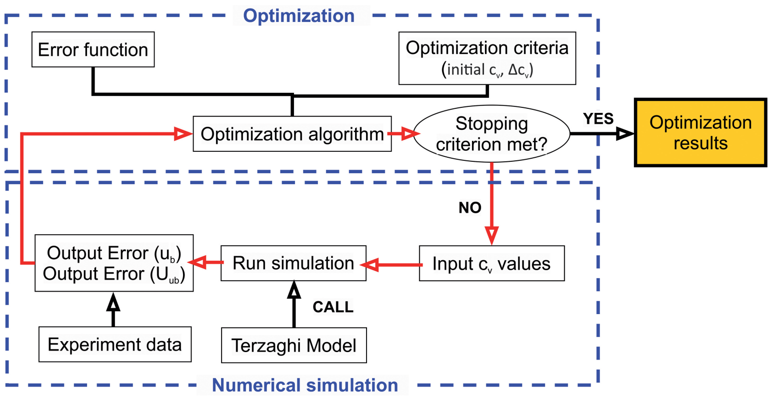

Fig. 2. Flowchart of optimization procedure

To reduce the influence of factors that affect the error function, such as the shape of the experimental dissipation curve, a number of measurement points and the scale effects on the fitness between the experimental and the simulated results, weighted to each calculation point, were adopted:

$$w_{n,i} = \frac{U^*_{n,i} - U^*_{n,i-1}}{2} + \frac{U^*_{n,i+1} - U^*_{n,i}}{2} \tag{2.5}$$

In the analysis, the average difference between the measured and the simulated results is expressed in the form of the least square method. In the formulation of an error function, an expression for the individual norm (e.g., the pore water pressure at the base u_{ub} or degree of dissipation U_{ub}) has been established. The individual norm is generally based on Euclidean measures between discrete points, composed of the experimental and numerical results.

3. Materials and methods

3.1. Soils used in the present study

The present study investigates the different pore water pressure dissipation patterns concerning the potential problems in interpreting test data. In the light of the scientific literature and conducted experiments, representative examples were selected and discussed. The soil samples for this study were collected from several regions located in Poland: Krakowiec clays from the northern part of the Carpathian Foredeep and the organic soils from a Vistula deltaic plain. The choice of the materials was dictated by the intention of capturing various consolidation behaviour of soils that cover a relatively wide range of Atterberg limits and creep susceptibility. In the study presented herein, both intact (I) samples and reconstituted (R) samples were tested. The reconstituted samples of Krakowiec clays were prepared from slurry at water content equal to the liquid limit (LL). The properties of the selected soils used in the study are summarized in Table 1. The liquid limits were measured using the cone penetration method. The plastic limit tests were performed following the standard procedure (rolling thread).

Table 1. Physical properties of the soils utilised in the present study

Soil type	G_s (–)	Sand* (%)	Silt* (%)	Clay* (%)	w (%)	LL (%)	PL (%)	PI (%)	Reference
Krakowiec clay A (I)	2.72	5	38	57	83.8	65.0	24.6	40.4	This study
Krakowiec clay C (I)	2.70	–	33	67	24.0	87.9	36.5	51.4	This study
Huaian clay (R)	2.70	–	–	–	–	100.0	38.8	61.2	[33]
Lianyungang clay (R)	2.70	–	–	–	–	55.6	28.8	26.8	[33]
Nanjing clay (I)	2.72	–	–	–	46.8	52.0	25.9	26.1	[33]
Illite (R)	2.45	0	36	46	–	131.0	78.0	53.0	[13]
Bentonite (R)	2.70	0	12	88	–	115.0	38.0	77.0	[13]
Red Earth (R)	2.64	44	47	9	–	33.0	19.0	14.0	[13]
Organic soil A (I)	2.61	13	68	19	60.3	82.9	33.3	49.6	This study
Organic soil B (I)	2.64	19	67	14	50.4	51.8	29.4	22.4	This study
Organic soil C (I)	2.62	2	56	42	83.8	109.8	54.2	55.5	This study

Note that: G_s – specific gravity, sand*, silt*, clay* – fraction percentage, w – water content, LL – liquid limit, PL – plastic limit, PI – plasticity index, (I) – intact sample, (R) – reconstituted sample

3.2. Rowe cell consolidation test

Laboratory experiments were performed using the small-scale Rowe cell (internal diameter 75 mm) to collect the dissipation data during consolidation. The pore water pressure u_b was measured at a small ceramic flush-mounted in the base plate and connected to the pressure transducers in the cell. An appropriate testing procedure was adopted to gather high-quality data from the test by the performance of saturation (with B-check) and consolidation stages. The ramp method was used to ensure the saturation of the samples. In this technique, the cell and back pressure at the specimen's faces are ramped, and the B-check is performed at regular intervals to examine whether the required Skempton's pore pressure Coefficient B (B-parameter) has been reached. The B-parameter was calculated as the ratio of the increase in the excess pore water pressure to the applied stress increment. Lade and Hernandez [23] indicated that the B-parameter is affected by the soil's porosity, compressibility of the soil structure, compressibility of the pore water, absolute pressure existing in the pore fluid, and degree of saturation. Note that if back pressures are applied to specimens too rapidly, the specimens may be overconsolidated by the temporary application of effective stresses that are higher than intended [24]. The course of the consolidation process, particularly its initiation, depends significantly on transferring the load to the soil sample. In this study, initiation of the consolidation was done by opening the external pressure valve with the back-pressure pressure valve closed. In this case, the pore pressure increases and settles at a certain level, and the sample does not settle. After the pore pressure has stabilized, the back-pressure valve is opened. The pore pressure drops to the equalizing pressure, with the sample settling simultaneously. For the experiments in the Rowe cell, a saturation of the soil samples with water was led in stages with appropriate diaphragm loads (cell pressures). Back pressures between 25 kPa and 250 kPa were utilized. On average, the duration of the saturation phase was between 10 min and 60 min. A soil sample was assumed to be saturated if the B-parameter was sufficiently high (in the analysis a criterion of $B > 0.98$ was expected).

4. Results and discussion

4.1. Experimental evidence on the laboratory-measured pore water pressure during consolidation testing

A major issue for consolidation analysis with pore water pressure measurements is time-lag. The delay or the time-lag in the pore water pressure mobilisation occurs in the early stage of consolidation and is characterised by an increase in the pore water pressure until its stabilised maximum value was reached [25]. Following the terminology given by Burns and Mayne [26], this kind of dissipation response can be classified as the 'non-standard dilatory dissipation curve' with the pore water pressure decay. Northey and Thomas [27] and Perloff et al. [28] indicated the possible reason for the time-lags and associated it with the low volumetric compliance of the pore water pressure measuring system. However, this is not the only problem when one analysis pore water pressure data. Therefore, based on the results of consolidation studies with pore pressures records published in the world literature, the shape of the dissipation curves was analysed, and potential problems were identified. The time-pore pressure parameter (C_{IL}) curves obtained from the chosen tests on intact and reconstituted clays as well as organic soils are shown in Fig. 3.

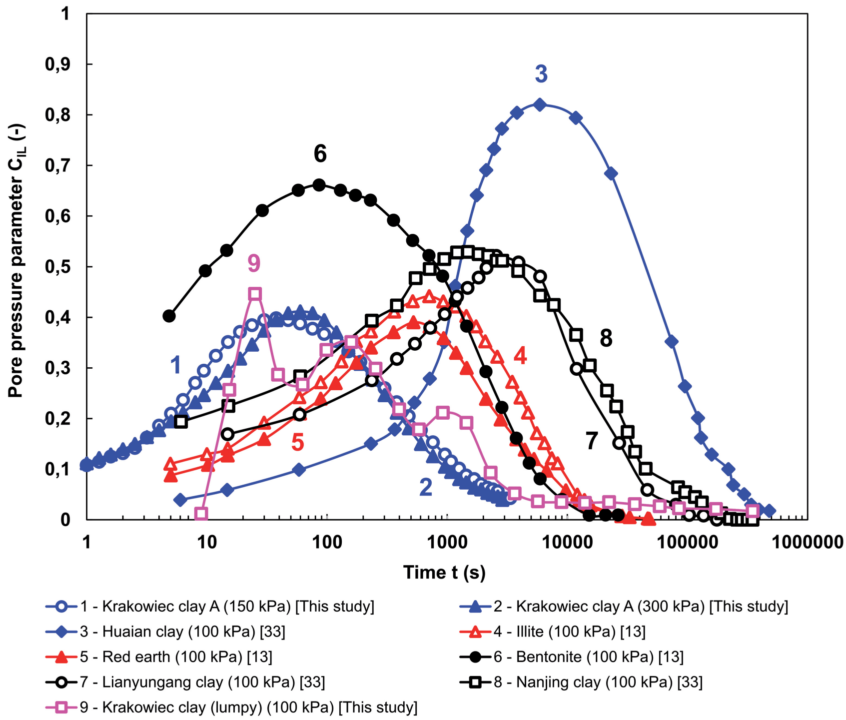

Fig. 3. Excess pore water pressure distributions during the consolidation tests (Note: the conditions of the samples are summarised in Table 1)

For convenience each consolidation curve was presented in form of the changes in the pore pressure parameter, C_{IL}. C_{IL}, is a dimensionless and normalises the excess pore water pressure through load increments. In other words, it is a ratio of the excess pore water pressure to the stress increase, i.e., $C_{\mathrm{IL}} = \Delta u_b/\Delta\sigma'$ (or $\Delta u_b/\Delta u_0$; in this work the load increment equals initial excess pore water pressure). The physical meaning of the C_{IL} and B-parameter is the same, and the distinction comes from the considered steps in the consolidation test: consolidation or saturation. For the purposes of the following considerations, the various results of own research were also included in the analysis. Inspection of the plotted distributions of the C_{IL} parameter with time for various soils reveals six features that should be taken into account during interpretation of the dissipation data. In general, all curves show the time-lag in the early phase of the test. In general very high pore water pressure mobilization times significantly affect the determination of c_v. This should be of particular importance when one is analysing curves 3, 7 and 8.

One of the advantages of using the C_{IL} parameter in assessing the pore pressure distribution is the possibility to directly check, whether the increase in pore pressure reaches the value of the applied stress and thus the theoretical condition is fulfilled, that the pore water transmits 100% of the applied load. As can be seen from Fig. 3, this criterion is not fulfilled in any case. One of the possible causes is the incomplete saturation of the sample, leading to the insufficient or incorrectly performed saturation procedure. Another issue worth discussing is the incomplete pore pressure dissipation at the end of the test, which may be due to the initiation of the next loading stage too early or the inability of further dissipation due to insufficient hydraulic gradients generated in the sample (see curves 1, 2 and 3). The latter case often occurs with very stiff intact soils of low-permeability, such as hard-plastic or compacted clays. Furthermore, in some cases, the pore water pressure does not increase from close to zero but from a much higher value. A possible explanation is, for example, the lack of complete pore pressure dissipation in the previous load increment or sudden spike in pore pressure in the build-up stage due to soil response to the increase in the load (e.g., in high-plastic reconstituted or remoulded clays). When considering the experimental data, sometimes one comes across significant fluctuations in the records, causing difficulties in the interpretation of the dissipation stage (see curves 3, 8 and 9). A noteworthy case is curve no. 9 relating to lumpy clay soil. In this type of soil, the internal structure and large spatially irregular pore spaces influence soil behaviour under load. As detailed in Fig. 3, the pore pressure distribution pattern is not a monotonic decreasing curve but an undulating curve with distinct pressure jumps. Such behavior could be attributed to the collapse of the soil structure due to the loading and wetting of the lumps, as a result of rapid reduction in the volume of the soil and the generation of step changes in the pore water pressure. Based on the observations, it is believed that the course of saturation of the specimen, primarily affects the interpretation of test results.

4.2. Normalization of the excess pore water pressure

During all the Rowe cell tests, excess pore water pressure was measured at the bottom of the sample, and then u_b versus time curves were generated. For practical purposes, it is convenient to normalize the dissipation data to examine the changes in the dissipation response with respect to the changes in the soil parameters or testing conditions [29]. Normalization can be done by the use of initial (u_0) or maximum excess pore water pressure ($u_{b,\max}$). For the purposes of this work, the first approach was formally adopted according to Eq. (2.3). It should be emphasized that the recorded initial value of the excess pore water pressure was always the same as its maximum value.

4.3. Optimization of the dissipation data

Seven individual tests on various fine-grained soils were first selected as an objective for the optimisation process. Afterwards, each dissipation data were converted into the degree of dissipation vs time curves to present the results. The theoretical degree of dissipation calculated from Eq. (2.2), adopting the optimised coefficient of consolidation and the experimental degree of dissipation calculated from Eq. (2.3) are shown in Fig. 4. Considering that c_v is the only one parameter required for the application of the Terzaghi dissipation model, the optimisation problem can be solved with the help of a function that can evaluate the error between the experimental and numerical results. For the purpose of automation of the process of optimization, two criteria were adopted in this study. First, the initial value of the coefficient of consolidation (c_{v0}) is determined on the basis of the approximate fit of the experimental and theoretical curves. Then, the value of the iterative step length expressed by the change in the coefficient of consolidation (Δc_v) was determined. For a given iteration step, this value is subtracted from c_{v0} or the value of c_v generated in the previous iteration step to produce a new parameter set. In this way, a family of theoretical solutions were obtained. From the set of possible solutions, a target solution is chosen as the one associated with the smallest error (i.e., higher fitness). The principle of the method used is analogous to the piezocone dissipation test methods (e.g., [30–32]).

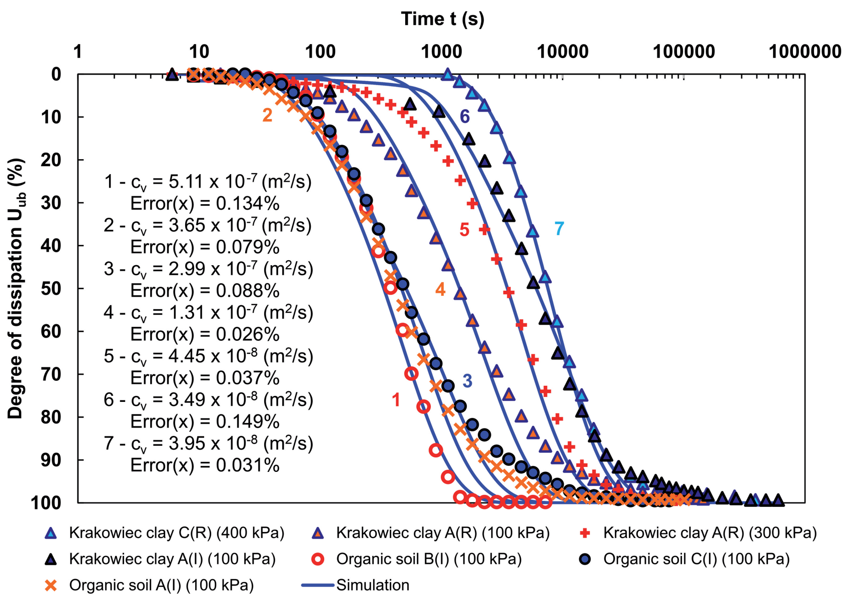

Fig. 4. Modeling of the dissipation data based on the optimization procedure (Note: values of pressure in the brackets are the same as initial excess pore water pressure u_0 values for each test)

As indicated earlier, the main issue in consolidation studies is the theoretical assumption of immediate load transfer to soil skeleton from the pore water after the load's application. In world literature, achieved times to complete pore water pressure build-up reach up to 200 minutes with considerable compression (see Fig. 3). This stands in opposition to the classic assumptions for the consolidation process. Comparing the present results with the laboratory dissipation data known from various sources [13, 33], confirmed that applied load was immediately transferred into pore pressure for all tested soils.

This study's rigorous experimental procedure contributes to collecting high-quality data with no build-up in the pore water pressure in the early stage of the consolidation. Thus, a monotonically decreasing response with time until equilibrium conditions are achieved is observed for each soil. This behaviour is mainly attributed to the normally or lightly overconsolidated fine-grained soils. The obtained results confronted with the data presented in Fig. 3 indicate two basic types of pore pressure dissipation curves:

– Type I – non-standard dilatory dissipation curve (see Fig. 3);
– Type II – monotonically decreasing dissipation curve (see Fig. 4).

For all the tested specimens, type II appears; thus, analysis is carried out on the characteristic features of these curves. Primarily, the occurrence of the inflection point on the curve was taken into account. As can be seen from Fig. 4, a good agreement between the simulations and experiments are observed in all the cases considered. For the predictions, the $Error(x)$ of the employed loading stages are lower than 0.15%, indicating the good fits achieved. In general, the predictions exhibit a similar time-dependent variation pattern as the experiments; however, there are discrepancies between the prediction and the experiment, particularly during the early and late stages of the dissipation process. Organic soil A, B and C noted a visible slowdown of the observed pore pressure dissipation in the later stages of the consolidation process ($U_{ub} > 50\%$) compared to the theoretical solution. This can be attributed to the high susceptibility to the creep deformation of this kind of soil [34, 35]. Dissipation delay due to creep was also observed for the reconstituted Krakowiec clay A and B. In the case of reconstituted samples with high plasticity, low initial void ratio, low permeability and under the high vertical effective stresses, the latter part of dissipation curves were successfully reproduced by the model. On the other hand, the initial part of the experimental curve indicates a faster dissipation rate than the theoretical curve. This may result from the creation of privileged pore water migration paths and a relatively easy flow of pore water through the soil. Recognised phenomenon depends on a rate of changes in the soil's solid structure due to the moment of the application of the load. Depending on the type of soil and the adopted loading scheme, these changes affect the pore water migration paths, allowing water to drain with the simultaneous dissipation of the pore water pressure excess.

To better illustrate the interpretation of the results detailed example is discussed. Figure 5 depicts an example of the interpretation of dissipation data according to the gradient-based algorithm incorporated into the optimisation procedure. It is seen that the simulation with $c_v = 1.18 \times 10^{-7}$ m^2/s and $Error\,(x) = 0.031\%$ shows the best match with the experimental results. In the discussed example, the following optimisation criteria have been used: $c_{v0} = 1.96 \times 10^{-7}$ m^2/s and $\Delta c_v = 1.11 \times 10^{-8}$ m^2/s. It is observed that in the later stages of consolidation, pore water pressure dissipation is delayed by viscous time-dependent effects such as creep. The influence of creep on the dissipation rate is that it keeps changing the instantaneous values of c_v. It is a different "Terzaghian Soil" at every instant having different (lower) c_v values. It is illustrated in the magnified part of the dissipation curve in Fig. 5. Following the assumption of a constant length of the iteration step, the optimal value of c_v is increased by 9.32%, 18.56% and 27.96%, respectively, to obtain a theoretical curve that fits the experimental curve in the later stages of consolidation. Based on previous findings on the occurrence of creep during pore water pressure dissipation (e.g., [36–40]), one would be led to the conclusion that creep and dissipation processes act concurrently during consolidation. However, in the early stages of consolidation, the effects of creep are imperceptible and can be visibly noted in the late stages. In the early stages of consolidation, the permeability of the soil medium predominantly governs the consolidation process. Throughout the test, the pore spaces are reduced, and the structural changes in the soil skeleton affect the dissipation of the pore water pressure. It leads to the delay in the pore water pressure dissipation and deterioration of the sample drainage conditions.

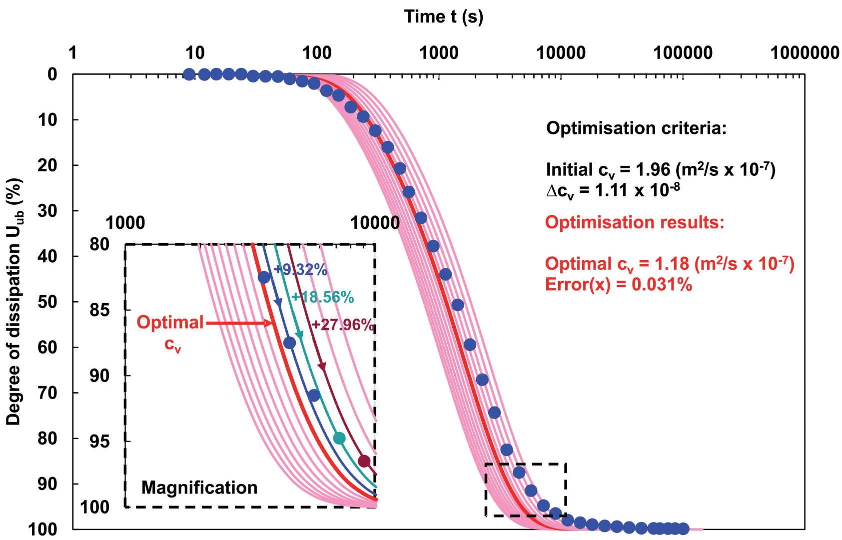

Fig. 5. Interpretation of the dissipation data according to the gradient-based algorithm incorporated into the optimization procedure

4.4. Gradient of the $u_B - \log t$ curve

The inflection point in the $u_b - \log t$ curve can be identified using the finite difference technique combined with a visual inspection. The slope of $u_b - \log t$ curve, $m = \Delta u_b / \Delta \log t$ is plotted against time in Fig. 6, from which the time at the inflection point ($t_{\text{inflection point}}$) can be pointed out easily. As can be seen, each curve is concave down. For the organic soils, inflection point appears much faster than for the reconstituted and intact clays. In addition, the soils under higher loads, i.e. Krakowiec clay C(R) under 400 kPa and Krakowiec clay A(R) under 300 kPa, indicate higher times at the inflection point. It is interesting to note that for each analysed soils, the values of degree of dissipation at inflection point are close to the theoretical value of 53.4% (Fig. 6). The most significant deviation from these theoretical values showed Krakowiec clay A(R) under 300 kPa, Krakowiec clay A(I) under 100 kPa and Krakowiec clay C(R) under 400 kPa.

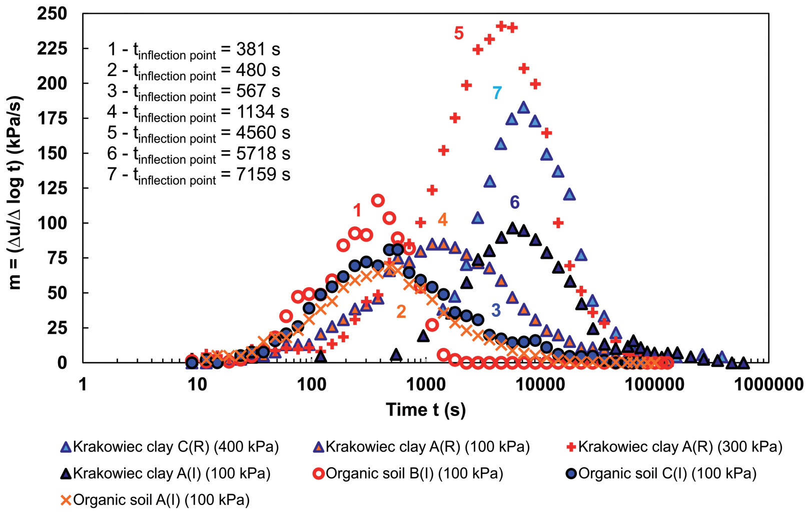

Fig. 6. Diagnostic gradient curves for the identification of the inflection point and corresponding time at the inflection point

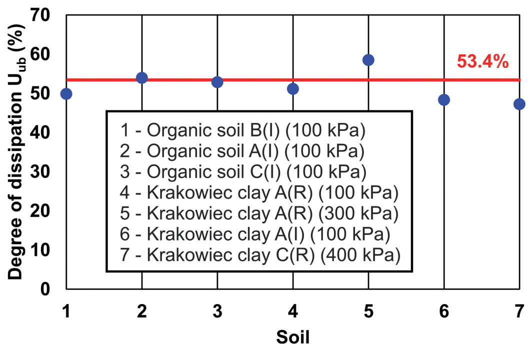

Fig. 7. Degree of dissipation at the inflection point for the soils utilised in the present study

5. Final remarks

This paper elaborates on pore water pressure dissipation patterns obtained in the laboratory consolidation test to study the consolidation rate. Basically, there are two types of laboratory pore water pressure at the base – time curves in the semi-logarithmic plot. Type I is characterised by the dilatory dissipation response with the pore water pressure build-up stage. This stage has significant build-up time, and the produced value of c_v from the dissipation stage may not be a real value. Type II curve shows monotonically decreasing dissipation response and is characterised by a well-defined inverse "S" shape with the presence of an inflection point. For the monotonically decreasing dissipation curve, the values of degree of dissipation at the inflection point are close to the theoretical value of $U_{ub} = 53.4\%$. In this work a method to estimate c_v and interpret the dissipation data using an optimization approach was used. A series of theoretically possible solutions were produced using numerically generated curves and their corresponding errors to find the optimal value of c_v. Based on the experimental evidence, the late parts of the dissipation curve are affected by the viscosity of soil structure, resulting in the delayed dissipation rate. Further work will be done on advanced models which capture anisotropy, rate-dependence (creep), and the effect of structure (destructuration + bonding).

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Kilka uwag na temat wzorców rozpraszania ciśnienia wody w porach z jednoosiowego badania konsolidacji

Słowa kluczowe: ił, konsolidacja, rozpraszanie, nadciśnienie wody w porach, nasycenie wodą

Streszczenie:

W artykule omówiono wzorce rozpraszania ciśnienia wody w porach, uzyskane w laboratoryjnym badaniu jednoosiowej konsolidacji. Wzorzec rozpraszenia ciśnienia wody w porach istotnie wpływa na obliczoną wartość współczynnika konsolidacji. Zasadniczo istnieją dwa typy krzywych rozpraszania ciśnienia wody w porach w przestrzeni półlogarytmicznej. Typ I charakteryzuje się dylatacyjną odpowiedzią rozpraszania na etapie wzrostu ciśnienia. Czas narastania ciśnienia może być znaczny, a obliczona wartość cv z etapu rozpraszania może nie być wartością rzeczywistą. Krzywa typu II wykazuje monotonicznie zmniejszającą się odpowiedź rozpraszania i charakteryzuje się dobrze zdefiniowanym odwróconym kształtem „S" z obecnością punktu przegięcia. Dla monotonicznie malejącej krzywej dyssypacji wartości stopnia dyssypacji w punkcie przegięcia są zbliżone do wartości teoretycznej $U_{ub} = 53.4\%$. W pracy omówiono metodologię interpretacji laboratoryjnych danych rozpraszania ciśnienia, pochodzących z badania komorze Rowe'a. W analizie wykorzystano gradientowy algorytm wyznaczania optymalnej wartości współczynnika konsolidacji w celu porównania rozwiązania teoretycznego z wynikami eksperymentalnymi, uzyskanymi dla różnych gruntów drobnoziarnistych. Optymalną wartość współczynnika konsolidacji powiązano z najniższą wartością funkcji błędu, która ocenia dopasowanie między eksperymentalną i teoretyczną krzywą rozpraszania.

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