Experimental insights into consolidation rates during one-dimensional loading with special reference to excess pore water pressure

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Abstract

Consolidation rate has significant influence on the settlement of structures founded on soft fine-grained soil. This paper presents the results of a series of small-scale and large-scale Rowe cell consolidation tests with pore water pressure measurements to investigate the factors affecting the consolidation process. Permeability and creep/resistance structure factors were considered as the governing factors. Intact and reconstituted marine clay from the Polish Carpathian Foredeep basin as well as clay–sand mixtures was examined in the present study. The fundamental relationship correlating consolidation degrees based on compression and pore water pressure was assessed to indicate the nonlinear soil behaviour. It was observed that the instantaneous consolidation parameters vary as the process progresses. The instantaneous coefficient of consolidation first drastically increases or decreases with increase in the degree of consolidation and stabilises in the middle stage of the consolidation; it then decreases significantly due to viscoplastic effects occurring in the soil structure. Based on the characteristics of the relationship between coefficient of consolidation and degree of dissipation at the base, the consolidation range that complies with theoretical assumptions was established. Furthermore, the influence of coarser fraction in clay–sand mixtures in controlling the consolidation rates is discussed.

Keywords Clays · Consolidation · Data optimisation · Excess pore water dissipation · Gradient-based algorithm · Permeability

List of symbols

CA	Casagrande method	PTIB	Porous top impermeable boundary
c_v	Coefficient of consolidation	R	Reconstituted sample
c_v^e	Instantaneous coefficient of consolidation based on compression	t	Time variable
c_v^u	Instantaneous coefficient of consolidation based on pore water pressure	T_v	Non-dimensional time factor
e_0	Initial void ratio	u	Pore water pressure
e_i	Void ratio at time t	u_0	Pore water pressure at the initial stage of consolidation
ESCS	European Soil Classification System	u_i	Pore water pressure at time t
GR	Gradient-based method	u_f	Final pore water pressure
I	Intact sample	U_e	Average degree of consolidation
I_p	Plasticity index	$U_{n,i}$	Experimental degree of consolidation
LIR	Load increment ratio	$U_{n,i}^*$	Theoretical degree of consolidation
M	$(2m + 1)\pi/2$	U_{ub}	Degree of dissipation at base
m	Integer	w_L	Liquid limit
MSL	Multiple-stage loading test	$w_{n,i}$	Range around each theoretical point $U_{n,i}^*$ characterising the divergence
		w_p	Plastic limit
		z	Distance from the top of the sample subjected to consolidation
		η	Factor of dominance
		ε_i	Strain at time t

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ε_f	Final strain at the end of excess pore water pressure dissipation under the given load increment
χ	Size-ratio between fine and coarse particles

1 Introduction

Compression, as far as is usually recorded in consolidation tests, measurement of pore water pressure is still not a routine practice. In engineering literature, there are fewer methods for analysing pore water pressure behaviour during consolidation than for observed deformation. Shortly after the formulation of the Classic Theory of Consolidation [62], Taylor [60] stated that conventional testing methods (based on strains) do not give sufficient data for a true interpretation of consolidation behaviour and for reliable comparison between test results and theory. To overcome the unsatisfactory results, much more attention was paid to pore water pressure measurements during consolidation testing. In the second half of the twentieth century, research was focused around the dominant influence of the pore water pressure measuring system on early stages of consolidation and the duration of the previous load increment [17, 43, 70]. In the studies by these above-mentioned authors, results of consolidation tests indicated the occurrence of delay in pore water pressure mobilisation, e.g. time lags in pore water measurements, characterised by an increase in the pore water pressure, until its stabilised maximum value was reached.

Perloff et al. [50] presented an analytical solution describing the effect of flexibility of the pore water pressure measuring system on the distribution of pore water pressure in a specimen during consolidation. Christie [9] clarified the effect of flexibility of the pore water pressure measuring system using the theoretical solutions of an extension of Terzaghi's theory. There have also been some reports on contradiction in relation to the classical assumptions of instantaneous and complete transmission of an external load to the liquid phase of the sample [3, 21, 32, 56, 79, 80].

In other respects, research on laboratory measured pore water pressure highlighted that the derived vertical coefficient of consolidation, c_v , differs significantly compared to that determined from compression data [14, 44, 48, 49, 53, 66]. Robinson [51] proposed a linear relationship between pore water pressure and compression for determining c_v . This method is based on the uniqueness of excess pore water pressure and compression using the

linearity concept of consolidation characteristics. A subsequent study conducted by Robinson [52] revealed that the secondary compression actually starts during the dissipation of excess pore water pressure, and its beginning strongly depends on the load increment ratio, LIR. Thus, the observed settlement is due to a combination of pore water pressure dissipation during primary consolidation and secondary compression. However, the evolution of compression during the dissipation process has only been investigated marginally and has not been sufficiently documented. The main objective of this study was to investigate the conditions of compression development during pore water pressure dissipation. Therefore, the uniqueness of theoretical relationship correlating the degrees of consolidation was evaluated and discussed. In addition, attention was paid to the methodological aspects in determination of c_v based on compression and pore water pressure. The key issue for predicting the rate of settlement is the reliable determination of c_v . Considering that Terzaghi's theory of consolidation is based on the principle of time scaling of the consolidation process, according to which the settlement time is proportional to the length of the drainage path, the conditions of pore water pressure dissipation become crucial. The determination of c_v based on pore water pressure data relates this parameter to the filtration aspect of the process and permeability of the medium. One definite advantage of this approach is that it facilitates the comparison of laboratory dissipation data with the piezocone dissipation test results. To the author's knowledge, there is no full matching method for the calculation of c_v based on pore water pressure. Thus, a novel method is proposed in this study in which a gradient-based algorithm for finding optimal value of c_v is incorporated. The principle of the presented method is similar to the piezocone dissipation test method [2, 7, 10, 22, 31, 55, 58] and relates c_v to the permeability of the soil rather than to its compressibility. Furthermore, the concept of instantaneous c_v was used to illustrate the nonlinear consolidation behaviour and to study the difference in consolidation time at the end of primary consolidation determined by using two types of laboratory data.

In the experiments presented in this paper, compression and dissipation rates during one-dimensional consolidation were examined. For this purpose, tests on intact, reconstituted marine clay and artificially prepared mixtures of clay and fine sand were conducted to better understand the soil behaviour. Furthermore, special attention was paid to the fundamental factors that govern the consolidation process. In this way, two types of consolidation curves were numerically modelled using a gradient-based algorithm to find the optimal value of c_v . All the simulations in this study were carried out on multiple-stage loading (MSL) Rowe cell tests.

2 Consolidation rates during one-dimensional loading

The rational theoretical description of the consolidation process refers to the idealised, two-phase mineral skeleton-water system, whose laboratory equivalent is a soil slurry devoid of structural bonds. Ensuring a fully saturated state of the tested material and restricted control of the conditions of the experiment allowed us to carefully investigate the evolution of compression (time-dependent volumetric strain defined as $(e_0 - e_i)/(1 + e_0) \times 100\%$) along the excess pore water pressure dissipation. Two simultaneous observations of this phenomenon can be very useful in the prediction of real material behaviour undergoing consolidation due to loading. Permeability controls the rate at which water is expelled out of the soil and compressibility controls the evolution of excess pore water pressures [57]. Then, a combination of permeability and compressibility can be used to establish the rate of strain at any time and the duration of consolidation. Based on a simple case of Terzaghi theory, three assumptions about consolidation behaviour of fine-grained soil can be made:

- (i) There is a uniqueness of excess pore water pressure and the course of compression,
- (ii) there exists a convergence between theoretical and experimental consolidation curves, and
- (iii) the constancy of c_v is valid throughout entire dissipation process.

2.1 Two definitions of degree of consolidation

For orderliness, the partial differential equation that governs the consolidation and pore water pressure, the dissipation process is expressed as follows:

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

A solution of Eq. (1) for a set of adequate boundary conditions describes how the excess pore water pressure, u , dissipates with time, t , and location, z . To assess assumption (i), it is convenient to qualitatively describe the relationship between two otherwise defined global, non-dimensional measures of the process [60]. In the study presented herein, deformation of the soil and pore water pressure was simultaneously measured and experimental data were converted into the degree of consolidation, U , which is directly proportional to the percentage consolidation [61]. Thus, two definitions of this parameter can be derived. The first is defined by axial strain or compression, and it is referred to as the average degree of consolidation, U_ε ; the second is defined by an excess pore water pressure, and it is referred to as the degree of dissipation at the base

of the sample, U_{ub} . At a given moment, the experimental degree of consolidation calculated on the basis of strain can be expressed as:

$$U_{\varepsilon,i} = \frac{\varepsilon_i}{\varepsilon_f} \quad (2)$$

The relationship between the degree of consolidation and the dimensionless time factor, T_v , can be derived using the Terzaghi theory as follows:

$$U_\varepsilon = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \exp(-M^2 T_v) \quad (3)$$

On the other hand, when excess pore pressure is measured at the base of soil sample under consolidation with drainage only on the top of the sample, we can use following expression:

$$U_{ub,i} = \frac{u_o - u_i}{u_o} \quad (4)$$

It is to be noted that the measurement of u_i at the base of the sample pertains to the threshold distribution of the pore water pressure.

The theoretical degree of consolidation in this case can be derived as:

$$U_{ub} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M} \sin M \exp(-M^2 T_v) \quad (5)$$

Equations (3) and (5) represent both traditional and exact solutions for Terzaghi's consolidation theory, which was proved by Lovisa using a method where the mass flux per unit area at each drainage boundary was determined [38]. The consolidation process can be considered as actually completed when the excess pore water pressure is dissipated as a result of the increase in load. However, for the nonlinear dependence between changes in the pore water pressure and void ratio (strain) [11], the degree of dissipation at any time t calculated on the basis of the pore pressures is not equal to the average degree of consolidation, which can be expressed as follows:

$$1 - \frac{\int_0^{2H} u dz}{2H \cdot \Delta \sigma'} \neq 1 - \frac{\int_0^{2H} \varepsilon_i dz}{\int_0^{2H} \varepsilon_{t=\infty} dz} \quad (6)$$

2.2 Relationship between U_ε and U_{ub}

Figure 1 presents the theoretical unique relationships between degrees of consolidation U_ε , U_{ub} and the non-dimensional time factor, T_v , and the relationship correlating those degrees. As illustrated in this figure, a linear relationship exists between U_ε and U_{ub} , when $U_{ub} > 35\%$

Fig. 1 Consolidation relationships: a) variation of theoretical average degree of consolidation and the degree of dissipation at base, b) theoretical consolidation rates

(unique linear increase in compression with the progress of pore water pressure dissipation).

The rate of this linear relationship specified by $\Delta U_e / \Delta U_{ub}$ was established as 0.64. This feature can be used to assess the experimental results theoretically. For a given fine-grained soil, any observed discrepancy from a theoretical line can be used as a diagnostic tool to highlight nonlinear soil behaviour. Some Rowe cell consolidation test results on various worldwide soft soils with different mineralogy and physical properties such as Atterberg limits were selected for the first sight appraisal. Table 1 summarises basic physical properties of the chosen soils. Figure 2 depicts a plasticity chart for classification purposes as per European Soil Classification System (ESCS) [30]. Most of these soils were found to plot above an A-line, defined by $I_p = 0.73 (w_L - 20)$ in the chart and represent a wide spectrum of plasticity. From Fig. 3, it is evident that the experimental $U_e - U_{ub}$ relationships for various soils are not unique and vary as consolidation progresses for particular soil. The vast majority of experimental curves are located below the theoretical line. Zeng et al. [80] related the location of an experimental $U_e - U_{ub}$ curve to the amount of consolidation degree happening within the period from the end of primary consolidation based on the deformation–time curve (EOP_e) to the end of primary consolidation determined by the excess pore water pressure observations (EOP_u). For this purpose, the degree of consolidation due to the excess pore pressure dissipation at EOP_u was used. It is to be noted that Zeng *et al.* [80] obtained EOP_e by the Casagrande procedure [5]. The authors concluded that the experimental $U_e - U_{ub}$ curve with a larger amount of consolidation degree difference between EOP_e and EOP_u is located above the theoretical

line. For Huaian clay, Lianyungang clay and Nanjing clay, a decreasing trend in the amount of consolidation degree with the decrease in liquid limit was also observed. The accuracy of estimation of EOP strain from the consolidation curve will be discussed further together with the analysis of compression development during excess pore water pressure dissipation and instantaneous consolidation parameters.

3 Experimental methods

3.1 Experimental setup

In the study presented herein, to investigate compression development during excess pore water pressure dissipation, two hydraulic consolidation set-ups of different sizes were used to carry out multiple-stage loading tests (MSL). Multiple-stage loading with reloading at EOP consolidation tests (MSL_p) and 10 days long-term multiple-stage loading consolidation tests (MSL_{10}) were used for the investigation (Table 2). The research schedule included (MSL_p) and (MSL_{10}) experiments in a small-scale Rowe cell and large-scale Rowe cell (internal diameter of 75 mm and 151.4 mm, respectively). The sketch of the Rowe cell apparatus is shown in Fig. 4. Possible disadvantages of the Rowe cell were highlighted by Khan and Garga [28] and Blewett et al. [4]. (MSL_p) tests were performed on low hydraulic conductivity reconstituted and intact clay, which enables the careful registration of effects related to filtration and creep. Conducting tests on reconstituted samples treated as debonded material was intended to eliminate the influence of soil structure. To study the effect of grain size

Table 1 Physical parameters of intact and reconstituted soils utilised in the present study

Soil	Type of deposit	State of specimen	Specific gravity G_s	Clay fraction (%)	Atterberg limits			Data source
					Liquid limit w_L (%)	Plastic limit w_p (%)	Plasticity index I_p (%)	
Boston Blue clay	Glaciomarine	I	2.81	60	33	18	15	[42]
Batiscan clay	Marine	I	2.80	81	49	22	27	[42]
Ottawa clay	Glaciomarine	I	2.82	74–76	68	29	39	[40]
Krakowiec clay	Marine	I	2.65	57	65	24.8	40.2	This study
Huaian clay A	Dredged	R	2.70	nm	100	38.8	61.2	[80]
Huaian clay B	Dredged	R	2.70	nm	70.8	33.5	37.3	[80]
Lianyungang clay	Dredged	R	2.70	nm	55.6	28.8	26.8	[80]
Nanjing clay A	Dredged	I	2.70	nm	43.8	22.8	21	[80]
Nanjing clay B	Dredged	I	2.72	nm	52	25.9	26.1	[80]
Red Earth	Lateritic	R	2.64	9	33	19	14	[51]
Bentonite	–	R	2.70	88	115	38	77	[51]
Kaolinite	–	R	2.63	29	50	30	20	[51]
Illite	–	R	2.45	64	80	35	45	[51]

* nm not mentioned

Fig. 2 Classification of soil utilised in the present study by liquid limit and plasticity index

composition and permeability on the consolidation behaviour, the $(MSL)_{10}$ tests in large-scale Rowe cell were performed on artificially prepared samples consisting of mixtures of fine and coarse fractions in various proportions.

3.2 Testing procedure

Appropriate testing procedure is required to gather high-quality data from MSL test in the Rowe cell. The way to achieve this is through the following three test stages: (i) saturation (B-check), (ii) pore pressure mobilisation and (iii) consolidation. For experiments in small-scale Rowe cell, saturation of soil sample with water was led in stages

Fig. 3 Variation of experimental average degree of consolidation and the degree of dissipation at base for worldwide fine-grained soils

Table 2 List of Rowe cell tests on samples of Krakowiec clay and clay–sand mixtures

Test/Sample	Soil type	Sample conditions	Rowe cell size	Effective pressure range (kPa)	Loading conditions
IK1	Krakowiec clay	Reconstituted	Small	300–900	EOP criterion
IK2	Krakowiec clay	Reconstituted	Small	300–900	EOP criterion
IK3	Krakowiec clay	Reconstituted	Small	300–900	EOP criterion
IN1	Krakowiec clay	Intact	Small	300–900	EOP criterion
IN2	Krakowiec clay	Intact	Small	300–900	EOP criterion
RM1	Clay–sand mixture	Reconstituted	Large	25–200	10 days
RM2	Clay–sand mixture	Reconstituted	Large	25–200	10 days
RM3	Clay–sand mixture	Reconstituted	Large	25–200	10 days

Fig. 4 Schematic diagram of the Rowe cell

with subsequent diaphragm loads (cell pressures) of 100, 150, 200, 250 and 300 kPa, at respective back pressures of 90, 140, 190, 240 and 290 kPa. The values explained above were chosen by reference to the loading scheme. The B-value for each step was calculated as the ratio of the increase in the excess pore water pressure to the applied stress increment according to procedure given by Head [20]. For all tests, the B-value was between 0.97 and 0.99. The tests with a uniform stress distribution and single-sided drainage (PTIB) were conducted, and 300 kPa back pressure was maintained during testing. Diaphragm loads of 600, 900 and 1200 kPa were used for the tests, giving effective pressures of 300 kPa, 600 kPa and 900 kPa, so that load increment ratios (LIR) of 1.0, 0.5 and 0.33, respectively, were achieved. Figure 5 shows typical excess pore water pressure normalised by the load increment, $C_{IL} = \Delta u / \Delta \sigma'_v$ for a low hydraulic conductivity

Fig. 5 Distribution of normalised excess pore water pressure for small-scale Rowe cell tests on reconstituted marine clay

reconstituted marine clay. The high values of C_{IL} parameters and practically only a few seconds of build-up time indicated fully saturated state of soil and immediate load transfer to the samples.

The loads were doubled at each increment ($LIR = 1.0$) for the large-scale Rowe cell experiments, and the values of effective pressure ranges were 25–50 kPa, 50–100 kPa, 100–200 kPa and 200–400 kPa. The B-check procedure was the same as above, with appropriate subsequent diaphragm and back pressures to ensure sufficient B-value. A permeability test was carried out after the completion of consolidation at each load step. The samples were subjected to vertical filtration in the bottom-up direction. The measurement of hydraulic conductivity (discharge method under constant pressure) was carried out by inducing a flow through the soil specimen by adjusting the influent pressure to 20 kPa for a load of 50 kPa. However, after the end of consolidation at loads greater than 50 kPa, the influent pressure was raised to 50 kPa. The amount of expelled water was also measured with the help of a measuring cup. Such a procedure is preferred to compare the values of discharge readings from the cup with the values of discharge measured in the Rowe cell.

3.3 Preparation of samples

The basic soil material used in the present study was a Middle Miocene marine deposit of Krakowicz clay, collected from a site near Tarnobrzeg in southeastern Poland, on the east bank of the river Vistula (northeastern part of the Carpathian Foredeep). The selected clay generally consists of illite, quartz, montmorillonite and calcite, with

illite as the main clay mineral. The sensitivity, S_t , of the Krakowicz clay from the investigated depth of 6 m is 4.57 [46]. In this study, the behaviour of reconstituted and intact kaolinite clay as well as a mixture of clay with coarse fractions was the focus of interest. Based on the results shown in Table 1 and Fig. 2, the soil was found to plot above an A-line in the plasticity chart and was classified as high plasticity CLAY (CIH) as per ESCS in accordance with the European Standard EN ISO 14688-2 [15].

The tests in the small-scale Rowe cell were carried out on the sediment obtained from the clay slurry. To obtain the clay fraction, kaolinite slumps were crushed and rubbed with distilled water by a mesh diameter of 0.0063 mm, and the thicker fractions remaining on the sieve were collected for the determination of particle-size distribution. The clay slurry was left for 2 days for sedimentation, and then, the clarified water was removed from the top surface, and the material was dried at 105 °C. The homogenous samples were obtained by mixing a certain quantity of water with the powder to prepare a slurry at a moisture content corresponding to the liquid limit.

To evaluate the effect of particle-size distribution on the value of determined consolidation parameters, the tests were carried out on samples consisting of clay–sand mixtures in various proportions, thus obtaining mixtures of a different permeability. The modelling of the sample grain composition consisted of two stages: pulping the proper weight of the dried clay in the mortar and preparing fractions of specific weights ($0.1 \text{ mm} < \varphi < 0.5 \text{ mm}$). The reconstituted samples were prepared in the laboratory as three batches of different size-ratios between fine and coarse particles, that is, χ values of 0.25, 0.50 and 0.75. Figure 6 shows the effect of particle-size distribution on

Fig. 6 Effect of particle-size distribution on permeability for clay–sand mixtures

permeability of the testing materials. The first sample (RM1) consisted of 25% fines and 75% mixture of coarse grains ($\chi = 0.25$). In the second sample (RM2), the fines content constituted 50% ($\chi = 0.50$). In the third sample (RM3), the proportions of the mixture components were reversed: 75% fines and 25% mixture of the remaining fractions ($\chi = 0.75$). The weights obtained in different proportions were mixed with an amount of distilled water to create a homogeneous slurry, which was poured into the consolidation cell. Two different sizes of samples were used in this study. The thin sample of 30 mm height and 75 mm diameter was tested in a small-scale Rowe cell. The thick sample of 40 mm height and 151.4 mm diameter was tested in a large-scale Rowe cell.

4 Dissipation pore water pressure induced compression

Small-scale Rowe cell experiments on both reconstituted and intact Krakowiec clay were conducted according to the restricted EOP criterion. Mesri et al. [42] observed that during the secondary consolidation phase (practically constant effective vertical stress), there is a small amount of excess pore water pressure left that tends to be dissipated from pore space due to the act with creep deformation. The choice of an appropriate EOP criterion based on the pore water pressure was suggested by Aboshi [1], Choi [8], Mesri and Choi [42], Feng [16], Kabbaj et al. [27], Kim and Leroueil [29], Mesri et al. [42] and Watabe et al. [68]. In this work, the EOP criterion was established based on initial and final excess pore water pressure, $u_f = 1\% u_0$. Hence, the duration of each load increment depends on the dissipation of the excess pore water pressure. The achieved time for the completion of dissipation does not exceed four days for all tests provided in the small-scale Rowe cell.

The consolidation process is traditionally divided into two successive phases: a primary consolidation phase and a secondary consolidation phase. During the primary consolidation phase, soil compression is controlled by the dissipation of excess pore water pressure and time-dependent (creep) deformations. It should be noted that in most fine-grained soils, the dissipation process is delayed by the viscous-plastic effects [18, 69]. The excess pore water pressure is dissipated at the end of primary consolidation (EOP), and the initial total applied stress is fully effective. Thereafter, the soil continues to deform, but at a rate controlled by soil viscosity. This is referred to as the secondary consolidation phase, and a point marking the transition between the two phases is the EOP state. However, one could assume that creep may occur during the primary consolidation phase along with the dissipation of excess pore pressure and the total deformation of consolidation

may show dependency on the duration of consolidation, and hence the thickness of the clay layer or the length of the drainage path [1, 3]. In this work, it is assumed that creep is a process in which the deformation of soil will occur as a function of time and that the creep rate is controlled by viscous resistance [19]. Joseph [25] studied the viscous and secondary consolidation phenomena to understand their physical mechanisms using the Dynamical Systems Theory [26]. Suggestion was made that viscous behaviour occurs both during and after primary consolidation in accordance with Creep Hypothesis B and is due to the strain rate dependence of the coefficients of friction at interparticle contacts. This view is referred to as the isotache theory in which the strain at EOP consolidation increases with the thickness of the clay and leads to unique secondary compression behaviour [12, 13, 18, 19, 35, 37, 72, 81, 82]. According to Joseph [25], secondary compression is the continued deformation (after completion of dissipation of the excess pore water pressure) of the soil structure after consolidation due to the small numbers of particles moving at random shear strains, in a Poisson process, to the new final positions. Based on Hypothesis B, many different elastic viscoplastic (EVP) constitutive models have been used to calculate consolidation settlements of soft soil ground [71, 73, 76–78, 83]. In contrast, there is also evidence which suggests that similar strain levels at EOP along any stage of consolidation can occur independently of the thickness of consolidated layer and duration of primary consolidation [36, 40]. This independence is often referred to as Creep Hypothesis A, which implies the same mobilised preconsolidation pressure in the field as the preconsolidation pressure determined from laboratory tests on thin samples.

4.1 Predominant factors controlling the rate of volume change

There are two different factors that drive the consolidation process. One can be associated with permeability and is called the hydrodynamic factor. The second can be established on the basis of viscous/time-dependent behaviour and is referred to simply as the resistance of soil structure/creep factor [11]. Figure 7 presents the typical consolidation rates during one-dimensional loading resulting from $(MSL)_p$ tests on reconstituted Krakowiec clay (sample IK2 and IK3). The strain and pore water pressure data were converted in this way into the degree of consolidation and then compared. It is seen that the superimposed $U_e - t$ and $U_{ub} - t$ curves indicate stress-dependent soil behaviour and clearly show the dominance of one factor over another when single load increment is considered. Given that Terzaghi's theory assumes uncoupled consolidation equations, excess pore water pressure and strain are determined

Fig. 7 Consolidation rates resulting from one-dimensional loading for reconstituted marine clay: **a** sample IK2, **b** IK3

separately (no hydro-mechanical coupling). Separate analysis of compression data collected from three successive load increments reveals an increasing trend in the delay of consolidation rate together with an increase in the applied effective vertical stress. In contrast to the strains developed in the samples, the consolidation behaviour determined by pore water pressure records is the opposite. Consolidation rates increase with an increase in the applied effective stress, which show a faster rate of pore water pressure dissipation with increasing load. As can be seen in Fig. 7, the gap between $U_\varepsilon - t$ and $U_{ub} - t$ curves decreases with an increasing stress. Similar characteristics of the compression and dissipation rates at high effective stresses arise from pore structural changes and possible breakdown of the solid particles. The pore structural changes are mainly influenced by the movement and rearrangement of soil aggregates, shearing of the soil particles and the resulting changes in the soil hydraulic

properties. However, of much more interest is the comparison of the two rates in a single load increment.

Consolidation rate is commonly described by the coefficient of consolidation, c_v , which depends on soil mineralogy, loading time, load application method, overburden load history, drainage conditions and thickness of the consolidated layer [36]. It is well documented that laboratory determination of this parameter strongly depends on the method used for its computation [33, 39]. Particular methods use a different matching of a single experimental point to the related consolidation progress for deriving c_v [5, 60, 63–65]. The main differences between these methods are based on the distinct graphical procedures used to find the primary consolidation range, e.g. 0% and 100% percentage of consolidation. Moreover, the application of some methods strongly depends on the shape of consolidation curves. Essentially, for soils that do not exhibit theoretical S-shaped settlement–time curves (type I according [33, 67]), it is unreasonable to use graphical methods in which the linear part of secondary consolidation should be distinguished from the curve to determine the consolidation parameters [39, 44, 45]. This inconvenience is associated with the Casagrande method (CA). When recorded data from the consolidation test produce ‘flat-shaped’ curve or when the curves exhibit no inflection point, it is impossible to determine the consolidation parameters by most of these methods. To overcome the drawbacks and limitations of existing graphical procedures, the optimisation approach was proposed for the determination of c_v and simulation of all tests.

4.2 Optimisation MSL consolidation tests

In mathematical optimisation, the gradient (GR) method is a first-order iterative algorithm for finding the minimum or maximum of a function, which plays an important role in solving many inverse problems [23, 24, 34, 74, 75, 79]. Using the inverse analysis, a given model is calibrated by iteratively changing input values until the simulated output values match the observed data. An inverse problem in consolidation is defined as the process of calculating from a set of observations the accurate value of c_v that produced them and could be resolved with the help of gradient-based algorithm. Given that this is a mono-objective problem, c_v with the lowest error was selected and was considered as the optimal value for experimental results. Such approaches have been adopted and validated for various natural and artificial geomaterials by Dobak and Gaszyński [14], Olek and Pilecka [47] and Olek [45]. To carry out the GR method at a suitable level of accuracy, a function that could evaluate the error between the experimental and theoretical solutions and then minimise this function should be defined. In the work described herein, convergence with

the smallest possible discrepancy was assessed by the scalar error function expressed as follows:

$$Error(x) = \frac{\sum \frac{|U_{n,i} - U_{n,i}^*|}{U_{n,i}}}{\sum w_{n,i}} \quad (7)$$

To reduce the influence of factors which affect the error function such as the shape of the experimental consolidation 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 as follows:

$$w_{n,i} = \frac{U_{n,i}^* - U_{n,i-1}^*}{2} + \frac{U_{n,i+1}^* - U_{n,i}^*}{2} \quad (8)$$

The proposed approach for determining the c_v is done through the combination of the complete range of theoretical and experimental consolidation courses. The determination of c_v by an existing methods is attributed to the use of specific points on consolidation curve, while in the GR method, a representative average consolidation rate is considered. Other advantages of the optimisation approach have been pointed out by Olek [48]. After using the compression and pore water pressure data to determine the values of c_v , a qualitative assessment regarding which governing factor of consolidation is dominant can be done with the use of the η parameter. This estimate can be called the factor of dominance, η . Therefore:

$$\eta = \frac{c_{v,e} - c_{v,u}}{c_{v,u}} \quad (9)$$

If $\eta = 0$, there is full agreement between the excess pore water pressure dissipation and the course of strain, and therefore, the consolidation process is governed by the two factors equally. However, if $\eta > 0$, there is a delay in the rate of pore water pressure dissipation with respect to soil compression rate. In this case, creep is the dominant factor. When $\eta < 0$, the hydrodynamic factor drives the consolidation process and causes a rate of dissipation faster than the compression rate. Thus, the study of the changes in the η gives a description of permeability evolution with the consolidation progress and the subsequent load increments. Moreover, it is useful in prediction of the consolidation governing factors contribution in the formation of the settlement.

4.2.1 Multiple-stage loading with reloading at EOP consolidation tests (MSL)_p

Figure 8 illustrates selected successive target values for c_v to be optimised together with corresponding objective errors for (MSL)_p tests. In addition, the η parameter was marked for each set of experimental data from a single load increment. As shown in the figure, the η parameter takes

positive, negative or close to zero values that depends on the susceptibility of soil body to the viscous/time-dependent deformation. Despite the change in soil porosity (decrease) during loading, the pore pressure dissipation rate increased with increasing stress, and the η parameter was used to explain this observation. In general, the η parameter decreases with increasing stress for all three tests. Positive values of the η parameter, usually at the first two load increments, indicate a significant delay in pore pressure dissipation due to creep. This behaviour can be associated with heterogeneous distribution of pores in samples and difficulties in water flow through the medium. As the stress increases, privileged water migration paths in the soil structure are formed through which water is able to flow. Hence, η parameter is a good indicator of the changes in permeability during the consolidation process.

Figure 9 presents an example of simulated results of two successive load increments for the IK2 sample. Apparently, the general consolidation behaviour of the tested reconstituted soil, under two successive load increments, indicates mixed conditions in terms of domination of the governing factor during the process. These results confirm previous observations made by Robinson [51] and Olek [48] which revealed that both factors have come to be recognised as playing interacting roles in consolidation. However, consolidation behaviour of soil is more complex and cannot be directly illustrated only by the single η parameter. This is evident mainly for large-scale Rowe cell experiments, where there are much longer drainage paths and a much greater impact of nonlinear compressibility of soil affecting the consolidation behaviour.

4.2.2 Long-term multiple-stage loading consolidation tests (MSL)₁₀

The comparisons between simulations and experimental data for pore pressure response during long-term consolidation tests in large-scale Rowe cell (MSL)₁₀ on mixtures with different size-ratios between fine and coarse particles, χ , are presented in Fig. 10. Following the optimisation approach described in this section, c_v values related to the rate of dissipation were established. At first sight, the corresponding values of the objective error, which lie between 0.031% and 0.2076%, indicate a good agreement between the experiments and simulations. As the tests in large-scale Rowe cell were conducted up to ten days, a large amount of secondary consolidation governed by creep was obtained. Therefore, the simulated curve relates to idealised conditions of the Terzaghi model and can be an indicator of the impact of viscous-plastic mechanisms, delaying the dissipation of pore water pressure at later stages of consolidation. Hence, the simulation produces reliable results for early and/or middle consolidation stages

Fig. 8 Results of optimisation for single load step in small-scale Rowe cell tests: **a** faster dissipation rate than compression/strain rate, **b** concurrent consolidation rates, **c** faster compression/strain rate than dissipation

Fig. 9 Comparison of consolidation rates for two successive load increments in small-scale Rowe cell test: **a** case when delayed dissipation rate occurs in relation to compression/strain rate, **b** case when faster dissipation rate occurs in relation to the compression/strain rate

when the dissipation process is less affected by viscous/time dependency. The observed delay in dissipation rate can be discussed with the help of functional relationship between instantaneous c_v and degree of dissipation at the base, U_{ub} . This will be further investigated in the next section.

4.3 Compression development during excess pore water pressure dissipation

To recognise the development of compression during excess pore water pressure dissipation, the experimental relationships between U_ϵ and U_{ub} were studied. The linear portion of $U_\epsilon - U_{ub}$ curve — that is, the primary consolidation range — was characterised by instantaneous values of c_v , which remained constant, while U and T_v varied or increased linearly with compression and time. Instantaneous values of c_v were calculated for each experimental point on consolidation curve according to the procedure developed by Olek [48]. This procedure facilitates the verification of assumption (iii) and the assessment of the optimal value of c_v determined by the GR method.

4.3.1 Experiments in small-scale Rowe cell

Figure 11 shows the relationships between U_ϵ and U_{ub} for reconstituted and intact Krakowicz clay. It can be seen that both the experimental $U_\epsilon - U_{ub}$ and $\delta - U_{ub}$ curves have equivalent shapes and can be used to evaluate the test results. The experimental observations indicate that the relationship between U_ϵ and U_{ub} is divergent from the theoretical assumption in most cases. As shown in Fig. 11, the consolidation behaviour of both reconstituted and intact Krakowicz clay is described by the set of nonlinear $U_\epsilon - U_{ub}$ curves. The location of these curves depends greatly on the applied vertical effective stress. In case of reconstituted samples under lower vertical effective stress, the $U_\epsilon - U_{ub}$ curve is located below the one with higher vertical effective stress. Figure 11 also shows the significant differences in consolidation behaviour of intact clay. The initial states of the two intact samples were significantly different. The initial water content of sample IN1 before being placed inside the consolidation cell was 41%; thus, the sample consistency was close to plastic. On the other hand, initial water content of sample IN2 was 24%,

Fig. 10 Comparisons between simulated and experimental results for: **a–d** clay–sand mixture with $\chi = 0.25$, **e–h** clay–sand mixture with $\chi = 0.50$, **i–l** clay–sand mixture with $\chi = 0.75$. Note: d_n parameter denote error between the experimental and theoretical results

indicating stiff soil consistency. For the stiff sample (IN2), the pore water pressure dissipated very slowly with the consolidation time. An increasing trend in the delayed rate of dissipation in relation to the rate of compression influenced the location of the $U_e - U_{ub}$ curves. Behaviour of these curves for sample with plastic-like consistency (IN1) was similar to that of reconstituted clay. The only difference was that the $U_e - U_{ub}$ curves for sample IN1 were very close together.

The experimental $U_e - U_{ub}$ curves for intact Krakowicz clay indicated greater nonlinearity vis-à-vis reconstituted

samples. Typical experimental variations of $\Delta U_e / \Delta U_{ub}$ against U_{ub} for investigated soils are shown in Fig. 12. An analysis of the data revealed that the experimental ratios of $\Delta U_e / \Delta U_{ub}$ were not constant and greatly differed from each other and from the theoretical constant beyond $U_{ub} > 35\%$. A vast majority of those ratios were higher than the theoretical constant: $\Delta U_e / \Delta U_{ub} = 0.64$. It is also worth noting that $\Delta U_e / \Delta U_{ub}$ ratios increased with increase in applied pressure.

Instantaneous consolidation parameters will exploit this feature to precisely identify the end of primary

Fig. 11 Variation of experimental average degree of consolidation and the degree of dissipation at base for the reconstituted and intact marine clay

Fig. 12 Experimental ratios of $\Delta U_e / \Delta U_{ub}$ against U_{ub} for intact and reconstituted marine clay

consolidation (EOP); in particular, via the use of $\delta - U_{ub}$, $c_v^e - U_e$, $c_v^e - U_{ub}$ and $\eta - \text{time}$ relationships. Using the $\delta - U_{ub}$ relationship, it is possible to isolate the secondary compression from time–compression data, establishing the range of consolidation where the compression develops linearly with the progress of pore water pressure dissipation. At the same time, $c_v^e - U_e$ and $c_v^e - U_{ub}$ relationships are useful in studying the nonlinearity in consolidation behaviour due to creep effects. Experimental evidence has shown that the secondary compression actually starts during the dissipation of excess pore water pressure [42, 52]. Thus, the predicted settlement is due to a combination of pore water pressure dissipation during primary compression and secondary compression. The point where the $\delta - U_{ub}$ plot deviates from linearity after a certain degree

of consolidation was identified as the so-called beginning of secondary compression. It can be demonstrated either by using the $c_v^e - U_{ub}$ relationship, when both compression and pore pressure data are available, or by using the $c_v^e - U_e$ relationship, when only compression data are available. To carry out the analysis, a case when both consolidation courses were concurrent with each other was chosen. Therefore, the consolidation curves produced by the three samples under an effective stress of 900 kPa were used for the investigation. Figure 13 shows the consolidation range when compression linearly develops with the progress of pore water pressure dissipation. The relationships between two types of instantaneous c_v values are drawn with their mean value (dotted lines), determined on the basis of established consolidation range when these

Fig. 13 Distributions of instantaneous consolidation parameters of reconstituted marine clay: **a–c** linear relationships between compression and degree of dissipation at base, **d–f** variation of coefficient of consolidation with degree of consolidation, **g–i** variation of coefficient of consolidation determined by compression with degree of dissipation at base for linear segment of δ - U_{ub} curve, **j–l** distribution of η parameter during consolidation

values are constant in Fig. 13d–f. Instantaneous values of c_v vary during consolidation. When c_v is plotted versus U on a semi-log plot, three different ranges of specific variability may be identified. For compression data, according to Tewatia and Venkatachalam [63], wherever the $c_v^e - U$ curve is horizontal, the soil follows theoretical behaviour, and wherever the curve exhibits any slope, the soil behaviour is influenced by the initial compression and viscous-plastic effects. The variability in the initial phase of consolidation is determined by the moment of applying load to the sample. (In terms of deformations, this is the primary

compression.) At this stage, the coefficient of consolidation demonstrates the threshold values. Then, the values of c_v decrease or increase together with the increase in the degree of consolidation and stabilise to a quasi-linear character. It should be noted that slight fluctuations in the course of $c_v^e - U$ or $c_v^u - U$ in this phase may be observed. The stabilisation confirms that assumption (iii) is fulfilled, and the point on the curve where the plot deviates from linearity in the middle/advanced stage of consolidation determines the EOP state. Considering that the range with quasi-constant values of c_v is identified, a geometric mean

of these values can be calculated and compared with those determined by the GR method.

The value of c_v calculated in this way is independent of a single measurement point and represents the consolidation behaviour for its significant progress. A comparison of the c_v values obtained from the linear part of instantaneous c_v versus degree of consolidation plot and those determined by optimisation technique is shown in Fig. 14. Considering the compression data, the ratio of the coefficient of consolidation calculated by the two methods, $c_v^{\text{quasi-constant}}/c_v^{\text{optimised}}$, was always between 1.0 and 0.5. On the other hand, for pore pressure data, the $c_v^{\text{quasi-constant}}/c_v^{\text{optimised}}$ ratio was between 0.5 and 1.5. The discrepancies obtained in the results between these methods relate to the first and second load increment, where both consolidation rates were significantly different. In case of sample IK1, the initial dissipation rate under effective stress of 900 kPa was much slower than the rate of compression. Drastic decrease in the initial values of the coefficient of consolidation explains the noticed delay in the dissipation rate. In contrast, both consolidation rates for samples IK2 and IK3 were convergent during this stage. As illustrated in Fig. 13e–f, instantaneous values of c_v increased up to 20% of the consolidation progress and then stabilised for a considerable period during the advancement of the process. The results of these two samples are quite similar showing the slight predominance of the dissipation rate over compression rate in the middle stage of consolidation. This phenomenon is clearly illustrated in Fig. 13j–l where the factor of dominance, η , assumes negative values with consolidation time. In the later stages of consolidation, instantaneous c_v shows a decreasing trend and the η parameter has positive values. As a consequence, the

instantaneous values of consolidation parameters clearly describe the extent to which the hydrodynamic and structure/creep factors govern the behaviour of consolidating soil.

4.3.2 Experiments in large-scale Rowe cell

Consolidation tests conducted in the large-scale Rowe cell were carried out to study the effect of coarser fractions on consolidation behaviour because the various mixtures with different size-ratios between fine and coarse particles, $\chi = D_{\text{fine}}/D_{\text{coarse}}$, represent materials with a different permeability (see Fig. 6). For all mixtures, the plots of void ratio versus hydraulic conductivity yielded a straight line, with the hydraulic conductivity change index, c_k , varying between 0.61 and 1.14. As can be seen in Fig. 6, the k values gradually decrease with an increase in the percentage of fine fractions. Similar results were obtained for clay–sand mixtures by Sällfors and Öberg-Högsta [54] and for bentonite–sand mixtures by Sivapullaiah et al. [59] and Castelbaum and Shackelford [6].

Figure 15 shows the relationships between U_e and U_{ub} for mixtures with different size-ratios, χ , between fine and coarse particles. It can be seen that the experimental curves diverge from the theoretical curves and cannot be expressed by a linear line similar to the other fine-grained soils mentioned in the engineering literature [41, 80]. As Fig. 15 shows, all the $U_e - U_{ub}$ curves are located in the lower part of the diagram below the theoretical line. In case of sample RM1 ($\chi = 0.25$), which had fewer but larger voids than the two other mixtures, the pore-size distribution was distinct and the $U_e - U_{ub}$ curves were farther apart from each other. Considering that the fine particles do not fully fill the voids created by coarse fractions, pore water was expelled easier and faster. At higher percentages of fine particles, many more of these particles fill the voids, which cause slower expelling of pore water from the sample and hence longer durations of dissipation. The experimental ratios $\Delta U_e/\Delta U_{ub}$ for the three mixtures were not constant and greatly differed from each other and from the theoretical constant of 0.64 (Fig. 16).

Comparison of consolidation rates reveals faster compression rates than dissipation rates in the initial and middle stages of consolidation and much lower rates in the later stages. In the most cases, both curves cross another in the middle or later stage of the process. The mixed behaviour mainly results from nonlinear compressibility of the soil, heterogeneous distribution of pores in the sample and viscous properties. Figure 17 shows the dependence of stress on the rates of consolidation. For clarity, two mixtures with extreme values of χ —the lowest and the highest—were presented. As may be seen from the figure, the $U_{ub} - t$ curves are moving down with the increase in load.

Fig. 14 Comparison of coefficient of consolidation by the two methods

Fig. 15 Variation of experimental average degree of consolidation and the degree of dissipation at the base for: **a** clay–sand mixture with $\chi = 0.25$, **b** clay–sand mixture with $\chi = 0.50$, **c** clay–sand mixture with $\chi = 0.75$

Fig. 16 Experimental ratios of $\Delta U_e/\Delta U_{ub}$ against U_{ub} for clay–sand mixtures

Furthermore, results show the shift to the right of the curves for the RM3 sample relative to the curves produced by the RM1 sample. Thus, the rate of compression and dissipation increases with the increase in load and the presence of coarse fraction in the sample.

Figure 18 compares the calculated EOP times for different vertical effective stresses along with the amount of consolidation degree ($100 - U_{EOP}$) happening within the period from the end of primary consolidation based on the compression–time curve (EOP_e) to the end of primary consolidation determined by the excess pore water pressure

observations (EOP_u). Meaning of this amount can also be expressed by the difference between the time at the end of primary consolidation based on the compression–time curve and the time when dissipation is fully completed. As can be seen from Fig. 18, there is a clear decreasing trend in the value of EOP time with the increase in vertical effective stress. To identify EOP parameters from a single compression curve, the GR method combined with quasi-constant criterion for c_v [44, 48] and the CA method were used. The lowest values of EOP times come from the GR method and the highest from the observed completion of

Fig. 17 Effective vertical stress dependence on average degree of consolidation and degree of dissipation at base for clay–sand mixtures

dissipation. The very short EOP times determined by the GR method result from very short periods of consolidation when the instantaneous c_v based on compression takes constant values. The EOP times determined on the basis of CA method were on average 5.5, 3.8 and 3.9 times lower than those determined by the completion of the dissipation, for sample RM1, RM2 and RM3, respectively. In all considered cases, the remaining pore water pressure at the EOP state determined by compression–time curve induces the additional compression of mixtures due to uncompleted dissipation. The latter observation very much affects the $(100 - U_{EOP})$ values. The $(100 - U_{EOP})$ values depend greatly on the method used for determining EOP state based on the compression–time curve. These values based on the CA method do not exceed 25%. In comparison, the optimisation approach gave values mostly within a range of 25% to 95%. The high percentage of remaining pore water pressure associated with the GR method may indicate a large amount of creep contribution to the total consolidation settlement. The $(100 - U_{EOP})$ values determined on the basis of CA method confirm the experimental evidence given by Zeng et al. [69] for the four kinds of clays with liquid limit within a spectrum ranging from 43.8% to 70.8%. Nevertheless, the EOP state determined by graphical matching procedure, like the CA method, is not a true end of primary consolidation theoretically [39, 44, 64]. Creep acting during the primary consolidation phase significantly shortens EOP times, both for clay–sand mixtures and fine-grained soils. In general, the $(100 - U_{EOP})$ values increase with an increase in vertical effective stress. As shown in Fig. 18b, the highest values of $(100 - U_{EOP})$ were obtained for sample RM1 ($\chi = 0.25$) and varied between 94.2% and 94.7%, excluding the vertical effective

stress of 25 kPa. The behaviour of sample RM2 ($\chi = 0.50$) was very similar, except that the $(100 - U_{EOP})$ values were lower. For sample RM3 ($\chi = 0.75$), a clear increasing trend in the value of $(100 - U_{EOP})$ was noted. On the contrary, scattered data were produced through the Casagrande procedure. No regularity was found in terms of the location of the $U_e - U_{ub}$ curves on the plot with the value of $100 - U_{ubo}$, where U_{ubo} is the amount of consolidation degree happening within the period from the end of primary consolidation determined by the optimisation of the compression–time curve to the end of primary consolidation determined by the excess pore water pressure.

Figure 19 shows distributions of instantaneous c_v determined from a compression–time curve with the degree of dissipation at the base, U_{ub} . For all mixtures, instantaneous c_v linearly decreases in the middle stage of dissipation, excluding the first load increment (effective vertical pressure of 25 kPa). As shown in Fig. 19, the sample with the highest content of sand fraction ($\chi = 0.25$) has higher values of c_v , and the highest rate of change during dissipation compared to the remaining two samples with $\chi = 0.50$ and $\chi = 0.75$. For all cases, the values of instantaneous c_v decrease with the increase in clay content in the mixture. Thus, the higher the value of χ , the higher the values of instantaneous c_v . Moreover, the behaviour of $c_v - U_{ub}$ curves in later stages of consolidation, when $U_{ub} > 85\%$, indicates significant creep effect, illustrated by the drastic decrease in c_v values. Similar trend was also observed for intact and reconstituted Krakowiec clay and other fine-grained soils investigated in the present work. A possible explanation for this observation is a substantial increase in heterogeneity due to the presence of coarse fraction, spatial distribution patterns of the soil pores,

Fig. 18 Conditions at end of primary consolidation state: a) effective vertical stress dependence on time at EOP state determined by three methods, b) comparison of amount of consolidation degree ($100 - U_{EOP}$) happening within the period from end of primary consolidation determined by the compression to the end of primary consolidation determined by the excess pore water pressure

length of the drainage paths, nonlinear compressibility and duration of the loading time. The η parameter changed in all the analysed samples. In the initial and middle stages of consolidation, the η values were positive and in the later stages were close to 0 or negative. This demonstrates improvement in the conditions for the pore pressure dissipation during consolidation.

5 Conclusions

Phenomenon of compression development during dissipation of the excess pore water pressure was investigated based on available world test data as well as main

Fig. 19 Distributions of instantaneous coefficient of consolidation determined on the basis of the compression data with the dissipation progress for clay-sand mixtures

experiments on various types of fine-grained soils. Based on the experimental results and discussion presented in the previous sections, main conclusions are drawn as follows:

1. Theoretical unique relationship between average degree of consolidation and degree of dissipation is not valid for considered soils. For a given soil, above $U_{ub} > 35\%$, the $U_e - U_{ub}$ relationship is formed by a set of curves corresponding to different load increments.
2. The nonlinearity between compression and pore water pressure results from the impact of two major governing factors in consolidation. All the experiments demonstrate that the resistance of soil structure/creep factor and permeability has come to be recognised as playing interacting roles in consolidation.
3. Prior to the GR method, the c_v can be derived based on the compression and pore water pressure data. The proposed approach utilises the combination of the complete range of theoretical and experimental consolidation courses. The determination of c_v by an existing methods is attributed to the use of specific points on consolidation curve, while in the GR method, a representative average consolidation rate is considered. The optimal values of c_v in significant cases are consistent with those determined on the basis of constancy criterion for c_v . Moreover, GR method facilitates the comparison of the laboratory dissipation data with the piezocone dissipation test results.
4. Instantaneous consolidation parameters vary during the process and may describe the extent to which the

hydrodynamic and structure/creep factors govern the consolidation of soil. The size of the coarser fraction plays an important role in controlling the consolidation rates. For the investigated clay–sand mixtures, values of instantaneous c_v decrease with the increase in clay content in the mixture.

5. EOP times determined based on the compression and dissipation of pore water pressure have a clear decreasing trend with the increase in vertical effective stress. The EOP time strongly depends on the method used with the following decreasing order: excess pore water pressure observations > CA method > GR method.
6. The amount of consolidation degree ($100 - U_{EOP}$) happening within the period from the end of primary consolidation based on the compression–time curve to the end of primary consolidation determined by the excess pore water pressure ambiguously determine the location of the $U_e - U_{ub}$ curve on the plot. However, the $U_e - U_{ub}$ curves for low-permeability clay–sand mixture lie above that for the high-permeability mixture. The $(100 - U_{ubc})$ values determined according to the CA method were significantly lower than the $(100 - U_{ubo})$ values determined by the optimisation approach. Moreover, the amount of $(100 - U_{ubo})$ for the mixtures show decreasing trend with the decrease in permeability.

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