An experimental investigation of the influence of plasticity on creep degradation rate

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Abstract

Intrinsic soil properties, such as the Atterberg limits, are essential factors influencing the mechanical behaviour of the fine-grained soils. In this study, a series of long-term multiple-stage loading oedometer tests were performed on alluvial organic soils to investigate the creep behaviour. The plasticity ratios ranged from 0.4 to 0.63. The smaller value of the plasticity ratio R_p indicated higher soil plasticity. The results showed that the coefficient of secondary compression $C_{\alpha e}$ of alluvial organic soils was stress- and strain-rate-dependent. The coefficient of secondary compression change index m was derived using a double-logarithmic approach for a creep degradation and was related to the plasticity and clay percentage to fines. Based on the results, it was found that high plasticity soils exhibit slow creep degradation rate during one-dimensional straining under normally consolidated state. The results show that the higher soil plasticity expressed by the plasticity index, plasticity ratio and clay percentage to fines, smaller the coefficient of secondary compression change index. Moreover, the correlations among a soil plasticity properties and creep parameters for the alluvial soils have also been proposed.

Keywords Consolidation · Creep · Plasticity index · Plasticity ratio · Secondary compression

Abbreviations

$C_{\alpha e}$	Coefficient of secondary compression	PC	Clay percentage to fines
$C_{\alpha e}^*$	Modified coefficient of secondary compression	PI	Plasticity index
$C_{\alpha ei}$	Coefficient of secondary compression for reconstituted soil	PL	Plastic limit
$C_{\alpha e}^{ref}$	Reference coefficient of secondary compression	R_p	Plasticity ratio
e	Void ratio	SL	Shrinkage limit
e_0	Initial void ratio	$S_{y,x}$	Standard error of estimate
e_i	Void ratio at time t	V	Specific volume ($V = 1 + e$)
e^{ref}	Reference void ratio	$\mathbf{Y}_m$	Vector of experimental values
EOP	End of primary consolidation	$\mathbf{Y}_p$	Vector of predicted values
ESCS	European soil classification system	t	Time
G_s	Density of solid particles	ρ	Bulk density
LI	Liquidity index	σ'_p	Preconsolidation pressure
LL	Liquid limit	σ'_{pi}	Yield pressure for reconstituted soil
m	Coefficient of secondary compression change index	σ'_v	Vertical effective stress
N	Number of data points	σ'_T	Threshold stress
		ϖ	Creep coefficient-based structure indicator
		χ	Structure indicator

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1 Introduction

Development of road infrastructure passing through the areas of soft soils occurrence imposes more and more demanding standards in terms of assessing possible

behaviour of the ground and its strengthening treatment. An example of such an area is the Żuławy Fens deposit, which lies in the northern part of Poland. The deposit that covers the Vistula River delta is the area of occurrence of delta formations in the subsoil, which are soft mineral-organic soils with a significant spatial variability. This kind of soils is particularly high compressible and susceptible to the time-dependent deformations which are a serious challenge for the geotechnical design [12, 13, 51].

Some natural fine-grained soils exhibit time-dependent mechanical properties. Numerous studies point to the significant contribution of secondary compression in the deformation of the soil subjected to an external load [5, 9, 11, 20, 72, 83]. The consolidation process is traditionally divided into two successive phases: primary compression and secondary compression. During the primary phase, soil compression is controlled by the dissipation of excess pore water pressures. This dissipation is sometimes delayed by the viscous-plastic effects (creep) [7, 22, 67]. At the end of the primary consolidation (EOP), the excess is dissipated, and the initial total applied stress becomes fully effective. After that, the soil continues to deform, but at a strain rate controlled by the soil viscosity. This deformation is referred to as the secondary consolidation, and a point marking the transition between the two phases is the EOP state. Some suggest that viscous behaviour occurs during and after primary consolidation in accordance with Creep Hypothesis B, formally proposed by Ladd et al. [42]. Creep deformation occurs because of the breakdown of interparticle bonds [46, 48], jumping of the molecule bonds [41, 49], sliding among particles [39], double porosity [63, 75] and the strain rate dependence of the coefficients of friction at the interparticle contacts [34, 35]. The Creep Hypothesis B is referred to as the isotache theory, in which the strain at EOP increases with the thickness of the clay and leads to unique secondary compression [14–18, 24, 36, 44, 60, 65, 71]. In contrast, there is also evidence to suggest that similar strain levels at EOP, along any stage of consolidation, can occur independently of the thickness of the consolidated layer and the duration of primary consolidation [45, 47, 48]. This independence is often referred to as Creep Hypothesis A, which implies the same mobilized preconsolidation pressure in the field as the preconsolidation pressure determined from laboratory tests on thin samples.

Conventionally, the time-dependent behaviour of a soil can be described using the coefficient of secondary compression $C_{\alpha e}$ defined as follows:

$$C_{\alpha e} = \frac{\Delta e}{\Delta \log t}, \quad (1)$$

To simplify the foundation design calculations, $C_{\alpha e}$ is assumed as a constant value. However, this statement leads

to unrealistic soil behaviour. As indicated by Yin [69], the logarithmic function has a limitation; when time is infinite, the deformation is also infinite and may overestimate the creep settlement. To overcome the limitation of the logarithmic function, some nonlinear creep formulations have been proposed. Yin et al. [73] formulated a nonlinear expression of $C_{\alpha e}$ with volumetric strain and time that considers the influence of the bulk density ρ and e . Wu et al. [68] also considered consecutively decreasing $C_{\alpha e}$ related to the ρ . Zhu et al. [82] and Yin et al. [74] demonstrated a decreasing $C_{\alpha e}$ as ρ increased. They introduced a modified $C_{\alpha e}^*$, defined in a double-logarithmic plane. On the other hand, Zhu et al. [80] discussed a nonlinear variation of $C_{\alpha e}$ as a function of $C_{\alpha e}$ at the liquid limit, structure indicator $\chi = \sigma'_p / \sigma'_{pi} - 1$ (χ reflects inter-particle bonding) and the creep coefficient-based structure indicator $\varpi = C_{\alpha e} / C_{\alpha ei} - 1$. Further, the nonlinear creep formulation accounting for both soil density and soil structure was proposed and the relationship between the LL , LP , as well as χ and $C_{\alpha e}$, was suggested.

Numerous attempts have been made to identify the stress and strain rate dependency of natural fine-grained soils. Generally, rate dependency is determined using the relationship between $C_{\alpha e}$ and vertical effective stress or compression. There are two opinions about the dependence of $C_{\alpha e}$ on stress. Newland and Alley [54] and Barden [4] noted that $C_{\alpha e}$ is almost independent of the effective stress except near the point of yield/preconsolidation pressure, where values of $C_{\alpha e}$ are slightly higher. Zhang and Wang [78] also postulated that, for some remoulded soils, $C_{\alpha e}$ is independent on vertical effective stress. In contrast, there are many more opinions that say $C_{\alpha e}$ is stress-dependent. This point of view is connected with the occurrence of the maximum value of $C_{\alpha e}$, which is closely related to the yield/preconsolidation stress. Several authors have reported that, as the effective stress increases, $C_{\alpha e}$ increases to the maximum and then decreases as the stress continues to increase [23, 43, 71, 74, 76, 79]. Considering the findings of the authors mentioned above, $C_{\alpha e}$ can be either very low for the vertical stresses well below the yield/preconsolidation pressure. However, $C_{\alpha e}$ increases rapidly as the yield/preconsolidation pressure is approached; it reaches its maximum at the yield/preconsolidation pressure and decreases under higher stresses. There is also evidence that $C_{\alpha e}$ either increases with the effective stress until the yield/preconsolidation pressure and remains constant thereafter [18, 33] or increases almost linearly with increasing effective stress [21]. Additionally, there have been some studies on the influence of the physical properties on the $C_{\alpha e}$ [1, 53, 58, 70, 77, 81]. However, many geotechnical correlations can only be applied in specific soil conditions and only for the limited soil types or regions of the occurrence.

In this study, the time-dependent, viscous properties of alluvial soils were related to the plasticity characteristics. Therefore, the ratio of the plastic limit PL (%) to the liquid limit LL (%) called plasticity ratio R_p (–) and the plasticity index PI (%) were considered. This study's primary objective was to investigate the decreasing trend of $C_{\alpha e}$ related to the density of the consolidated soil. Therefore, a double-logarithmic approach was used and its suitability for intact alluvial soils was evaluated. The preliminary results of the long-term multiple-stage oedometer tests on different intact organic soils from the same geological origin were presented in the light of the influence of the plasticity of the soil on the changes in the creep rate. Based on the results, correlational models of the coefficient of secondary compression change index m using plasticity parameters were proposed.

2 Experimental program

2.1 Test materials

This study was conducted using intact samples retrieved from the Żuławy Fens located near the urban area of Gdansk, Poland. The study area is shown in Fig. 1. There, a deltaic plain was created by the accumulation processes and filled with quaternary sediments. The Żuławy Fens subsoil is extremely heterogeneous because of a very complex depositional history and varies site to site. The geological model for the study area [6] is generalized as shown in Fig. 2. In the model weak mineral-organic soils such as aggragate organic muds with medium compact fine-grained sands (denoted as A1 or A2 due to the depth of occurrence), weak mineral soils such as a fine-grained sands with an admixture of decayed plant debris with silt and silty clay (denoted as B1 or B2 due to the depth of

occurrence) and finally alluvial sands (denoted as C) were identified.

The aggragate organic muds are yellow and grey, to black or grey, to brown and composed of admixtures and interlayers of fine sand, silty sand, loamy sand, silty clay and gyttja. Figure 3 shows a sample of the loamy soil retrieved from a borehole in the sampling area. The alluvial soils of the Żuławy Fens are characterized by very high variability in organic content and water content. This variability translates into different physical and mechanical properties [13, 59]. The term ‘alluvial mud’ refers to organic fine-grained soil particles transported by water and deposited on the bottom of the riverbed or flooded area as the speed of flowing water decreased. As a formation, the mud (called also “dy”) occupies an intermediate place between peat and gyttja. It differs from peat in its high degree of plant mass humification, in which there is no macroscopically recognizable plant debris. There is also no roughly fibrous structure typical for peats. The difference between mud and gyttja is that the mass of mud is humus, while the organic mass of gyttja is detritus.

3 Testing procedures

3.1 Index properties and classification of soils

To study the influence of the physical properties on the time-dependent behaviour of the fine-grained organic soils, standard index tests were performed. The primary properties (e.g., PL , LL and SL , mineral composition, grain-size distribution) and secondary properties (e.g., water content, density, and consolidation/stress conditions for all the soils) were considered. The LL was determined according to CEN-ISO/TS 17,892–6 [30], and the PL was determined in compliance with ISO/TS 17,892–12 [31]. It should be noted that determining the PL using the thread soil rolling method may have some major limitations [25, 37] and is prone to human error [2, 3, 19, 55, 64]. Based on PL and LL values, the plasticity index $PI = LL - PL$ (%) and the reference parameter named the plasticity ratio $R_p = PL/LL$ (–) [57] were determined. The PI of the soil indicates the size of the range of water contents where the soil exhibits plastic properties. The R_p also points to the plasticity of the soil and can be correlated to the specific surface area SSA ($m^2 g^{-1}$), clayey fraction content CF (%) and the liquidity index LI (–) [62] or its equivalent water content ratio WCR (–). Based on the sizable data gathered by Spagnoli and Shimobe [57], the smaller the R_p , the higher the plasticity of the soil. The particle-size distribution was identified prior to CEN-ISO-TS 17,892–4 [29], and the specific gravity tests were performed following the procedure in CEN-ISO-TS 17,892–3 [28]. The soil organic

Fig. 1 The Gulf of Gdansk, Poland, and the location of the sampling area

Fig. 2 Geological cross-section of the study area. Note: A1 or A2 denotes aggragate organic muds with medium compact fine-grained sands; B1 or B2 denotes fine-grained sands with an admixture of decayed plant debris with silt and silty clay; C denotes alluvial sands

Fig. 3 Example of the soil material used in the investigation

matter SOM (%) was determined on the basis of the total carbon content in soil after dry combustion according to ISO 10,694 [27]. In this work the soils were classified based on the Atterberg limits and the grain-size distribution consistent with the European Soil Classification System (ECS) [38]. The results of the classification were also verified under the criteria given by Moreno-Maroto and Alonso-Azcárate [50]. Following their definition, the term

Fig. 4 Model for creep degradation in $\text{Log } C_{ae}$ – $\text{Log } e$ plane

clay refers to the fine-grained material whose PI is equal to or higher than $LL/2$.

3.2 Consolidation tests

Consolidation characteristics were investigated based on the results of the long-term (ten days) multiple-stage loading (MSL_{10}) oedometer tests. The dimension of the specimen was 20 mm high and 60 mm in diameter. The tests with uniform stress distribution and drainage conditions in the vertical direction were done using pervious top and pervious bottom (PTPB). Two loading procedures were followed. For the one-dimensional consolidation tests on samples with the highest plasticity, the loading scheme was $6.25 \rightarrow 12.5 \rightarrow 25 \rightarrow 50 \rightarrow 100 \rightarrow 200 \rightarrow 400$ kPa. For the low plasticity samples, the loading scheme was $12.5 \rightarrow 25 \rightarrow 50 \rightarrow 150 \rightarrow 250 \rightarrow 500$ kPa. It was assumed that the compression curves (in semilogarithmic planes) exhibit two regions with compressibility that varied depending on the stress level. In general due to the soil structure, the soil's stress–compression behaviour develops along the recompression/reloading line (overconsolidation line). The effective vertical stress σ'_v at the intersection of the overconsolidation line and the normal consolidation line indicates the pre-consolidation pressure σ'_p [10, 26, 52]. In this study the σ'_p values are determined on the basis of the bilogarithmic method [10], which utilizes the plot of the $\ln V$ and $\log \sigma'_v$. To assess the stress and strain rate dependency, C_{ae} was calculated from the linear slope of the $e = f(\log t)$ curve beyond the transition from the primary to secondary compression and plotted with the effective vertical stress and the void ratio.

Following the methodology given by Zhu et al. [80], the C_{ae} for each applied stress level was plotted against the e in a double-logarithmic scale for the normally consolidated state. This approach was originally referred to reconstituted

Table 1 Physical properties of the soils utilized in the present study

Sample	Soil classification (ESCS)	G _s (—)	Grain-size distribution			Atterberg limits					R _p (—)	
			Sand (%)	Silt (%)	Clay (%)	PC (%)	SOM (%)	w (%)	PL (%)	LL (%)		PI (%)
orSiV	Organic very high plasticity SILT	2.62	2	56	42	42.86	9.46	83.81	54.22	109.78	55.56	0.49
orCII	Organic intermediate plasticity CLAY	2.61	13	68	19	21.84	7.07	60.28	33.33	82.96	49.63	0.40
orsaSiH (A)	Organic sandy high plasticity SILT	2.63	18	61	21	25.61	6.78	54.01	34.86	68.86	34.00	0.51
orsaSiH (B)	Organic sandy high plasticity SILT	2.64	19	67	14	17.28	3.93	50.36	29.35	51.75	22.40	0.57
orsaSiH (C)	Organic sandy high plasticity SILT	2.64	37	52	11	17.46	4.19	48.62	33.89	53.38	19.49	0.63
orsaSiH (D)	Organic sandy high plasticity SILT	2.64	29	62	9	12.68	4.27	48.68	33.48	55.08	21.60	0.61
orsaSiH (E)	Organic sandy high plasticity SILT	2.62	24	62	14	18.42	5.45	47.51	29.89	51.55	21.66	0.58
orsaSiH (F)	Organic sandy high plasticity SILT	2.64	20	61	19	23.75	5.19	54.88	32.75	58.95	26.20	0.56
orsaCII (A)	Organic sandy intermediate plasticity CLAY	2.62	23	40	37	48.05	5.00	42.97	26.55	66.11	39.56	0.40
orsaCII (B)	Organic sandy intermediate plasticity CLAY	2.59	38	26	36	58.06	8.15	73.08	46.04	103.69	57.65	0.44
orsaCII (C)	Organic sandy intermediate plasticity CLAY	2.65	29	53	18	25.35	3.49	42.69	27.08	51.49	24.41	0.53

fine-grained soils. However, it is interesting to check whether this approach is also valid for an intact soils. The proposed model assumes consecutive decreases of the C_{ae} as the applied stress level increased and the compression decreased. These changes were attributed to increased density resulting from the compression. Within the projection of the $\log C_{ae} - \log e$ plane, there is a straight line with gradient m for the normally consolidated state (see Fig. 4). In this work parameter m is the coefficient of the secondary compression change index and is used to calculate the change in the creep rate as the void ratio decreases. Its determination is analogous to the permeability change index C_k , which illustrates the change in permeability with changes in the e . The parameter m describes the rate at which creep degradation process occurs. High values of the m indicate that the creep degradation is rapidly slowing down. In turn, low values of the m illustrate slow degradation rate of C_{ae} during compression. The adopted formulation for the creep degradation is expressed as follows [74]:

$$\frac{C_{ae}}{C_{ae}^{ref}} = \left(\frac{e}{e^{ref}} \right)^m, \quad (2)$$

In this study, the reference point (C_{ae}^{ref} , e^{ref}) was arbitrarily selected and corresponded to the soil's LL . Since the reference point is chosen, Eq. (2) can be rewritten as:

$$C_{ae} = C_{ae}^{ref} \left(\frac{e}{e^{ref}} \right)^m, \quad (3)$$

Application of the given formulation (Eq. 2) implies:

- (i) a decreasing C_{ae} over time for an applied stress level as the void ratio decreases during creep,
- (ii) the repeated decreases in C_{ae} with applied stresses as the void ratio is consecutively lowered during loading,
- (iii) a positive void ratio at the end of the creep phase resulting from compression.

4 Results and discussion

This section presents the results of the tests performed on fine-grained alluvial soils. Particular emphasis was given to the aspects of compression behaviour, time dependency and correlations between the creep parameters and the plasticity of the soil.

4.1 Physical properties and soil classification

Table 1 presents some physical properties of the selected soils. Figure 5 shows the plasticity for each classification under the ESCS, combined with the C-Line and M-Line to further delimit the clays and other less-plastic soils. The PI

values between $LL/3$ and $LL/2$ were defined as moderately or slightly clayey material. The CF of the analysed samples varied from 9 to 42%. The activity of these soils was evaluated using the relationship between CF and PI in the activity chart (see Fig. 6). Different liquid limits have an impact on determining soil activity [56] and the degree of expansiveness [61]. As a result, the investigated soils were found to be very different in terms of activity. The general expansiveness of these soils was established as low and medium, except for the sandy intermediate plasticity clay (orsaCII (C) sample) and very high plasticity silt (orSiV sample).

4.2 Compression behaviour

The void ratio–effective vertical stress and specific volume–effective vertical stress relationships for all soil samples are shown in Fig. 7. Most of the compression curves were characterized by a clearly marked breakdown associated with preconsolidation pressure or the influence of the destructuration process. The consequence of this was a lower constant slope of the curves at σ'_p values and a significant increase in the constant slope after exceeding this boundary values. In the tested soils, a tendency was observed to increase the slope of the curves with increasing stress, which may result from both the high presence of dispersed organic substance (e.g., sample orSiV and orsaSiH (B)), high natural water content and the plastic state of relatively young sediments (e.g., sample orCII and orsaCII (B)). The σ'_p of the ten intact investigated soils ranged from 18 to 70 kPa. In addition, the values of the compression index C_c pertain to the normally consolidated

region were calculated from the compression curves. The C_c is an important compressibility parameter with particular relevance to primary settlement calculations for normally consolidated or lightly overconsolidated soils [45]. In the geotechnical practice the C_c is often related to the LL [46]. The C_c of the investigated soils for the normally consolidated state is plotted against LL as shown in Fig. 8, and the equation of the straight line: $C_{c-} = 0.0096(LL) - 0.249$ has been obtained. The performance of the empirical model is commonly evaluated with the regression coefficient R^2 defined as follows [66]:

$$R^2 = \frac{\sum_{i=1}^N (Y_m)^2 - \sum_{i=1}^N (Y_m - Y_p)^2}{\sum_{i=1}^N (Y_m)^2}, \quad (4)$$

In the analysis, the value of R^2 was 0.90 and indicates good predictive ability of the model due to the fact that the compressibility of soft soils closely associates with the mineral compositions which also control the LL of the soil.

Figure 9 shows procedure for identifying parameters from the consolidation results of the sample orsaSiH (E). The relationship between C_{ae} and e as an example illustrating the calculation procedure of the m value was also presented. For those purposes the results of three tests on the same soil material were used. As can be seen from Fig. 9a, the threshold stress σ'_T was found to exist and is corresponded to the maximum value of the C_{ae} . After reaching the σ'_T , the gradual decrease in C_{ae} with the vertical effective stress was observed. In the discussed case σ'_p was determined by the bilogarithmic method and was about 20 kPa (see Fig. 9b). Thereafter, values of σ'_T and σ'_p were compared. From the comparison it was observed

Fig. 5 Classification of soils based on liquid limit and plasticity index. Note that symbols Cl and Si are clay and silt, respectively; L, I, H and V are low plasticity, intermediate plasticity, high plasticity and very high plasticity, respectively

Fig. 6 Data plotted in an activity chart. Note that the boundary lines for inactive ($A < 0.75$), normal ($0.75 < A < 1.25$) and active ($A > 1.25$) clay are from [56]

that the σ'_T was 5 times the σ'_p (see Fig. 9c). Moreover, it is not surprising that the C_{ae} decreases with decreasing void ratio e . Then, the relationship between $\log C_{ae}$ and $\log e$ was used for the evaluation of the changes in C_{ae} during compression (see Fig. 9d). As presented for the normally consolidated state the $C_{ae}-e$ was plotted in double log space, where linear relationship was found to exist with the gradient $m = 5.48$. Parameter m was taken from the equation of that linear trend line. Relatively high value of the m indicates rapid changes in C_{ae} and the fast creep degradation rate.

4.2.1 Stress rate dependency of C_{ae}

Figure 10 shows the variation of C_{ae} under increasing effective vertical stress owing to the destructuration effect. As can be seen, C_{ae} is stress-dependent and curve of the function $C_{ae} = f(\log \sigma'_v)$ has a concave downward shape with the maximum at the threshold stress σ'_T . In general, for each soil sample, the shape of the soil characteristic curves $C_{ae} = f(\sigma'_v)$ was developed, and each exhibited a well-defined maximum C_{ae} . In some cases latter part of the curve tended to be stable. With the increase in pressure, the C_{ae} of the orSiV, orCII, orsaSiH (A), orsaSiH (E) and orsaSiH (F) sample increased faster and their C_{ae} was significantly higher than that of other samples, except for orsaSiH (E) sample. For the above-mentioned samples, the C_{ae} reached its peak when the pressure was 100 kPa. For other samples, the peak was reached, when the pressure was 50 kPa. The values of the σ'_T , for each soil sample, were significantly higher than values of the σ'_p . Table 2

provides the threshold values of the σ'_T/σ'_p ratio for the studied soils. As can be seen, threshold ratio ranged from 1.43 to 5 and indicates that the soil structure has a significant influence on the variation pattern of the C_{ae} rather than yielding or preconsolidation. As a result, this variation is due to the destructuration effect with the peak value of C_{ae} at a void ratio corresponding to the vertical stress well-above the σ'_p . Increased consolidation pressure induced the gradual destructuration of the soil structure and, hence, C_{ae} increased. In this pre-peak stage the collapse of the structure occurs with the grains readjustment and the damage to the grain cementation increases the creep [81]. Near the σ'_T , the soil structure broke down, and the destructuration process slowed (post-peak stage), resulting in a lower C_{ae} value at higher stresses.

4.2.2 Coefficient of secondary compression change index m

The soil data were plotted in a double-logarithmic plane, and the relationship between $\log(C_{ae})$ and $\log(e)$ was then evaluated (see Fig. 11). For all tested soils, $\log(C_{ae})$ was linearly related to $\log(e)$ in the normally consolidated state. The resulting regression coefficient R^2 was between 0.87 and 0.99. As shown in Fig. 9, the material constant m depended strongly on the type of material (grain-size distribution, PL and LL) and initial void ratio. In general the soils classified as very high plasticity silt, high plasticity clay and sandy high plasticity clay were characterized by low values of m and corresponded to the wide range of LL and PI . However, in the case of sample orsaCII (C) is

Fig. 7 Soil compression curves: a) semilogarithmic plane; b) bilogarithmic plane

clearly an exception to this general trend and this soil behaves more like the silty soil than the clayey soil. According to the ESCS classification, the soil is classified as organic sandy intermediate plasticity clay. Following the classification principles, if the fines represent more than or equal to 50% in the sample and LL lies between 35 and 50%, the soil is classified as clay. Moreover, this soil was found to plot slightly above an A-line in the plasticity chart. However, taking into account the definition of clay

given by Moreno-Maroto and Alonso-Azcárate [50] this soil does not meet their criteria, because the discussed soil was found to plot below the C-line (PI is not equal to or higher than $LL/2$). In that context, classification is reinterpreted and the soil may be regarded as a moderately clayey material, whose PI value is between $LL/3$ and $LL/2$, what brings its behaviour closer to the silt soil. The smaller initial void ratio and lower compressibility of this sample compared to that of samples orsaCII (A) and orsaCII

Fig. 8 Relationship between compression index and liquid limit

(B) are also not without significance. Unlike the less-plastic soils, the values of m were high and applied to the relatively narrow ranges of LL and PI . The obtained results show that the plasticity of the soil greatly affects a rate at which C_{ae} change during one-dimensional compression.

Subsequently, the averaged C_{ae} was calculated within the normally consolidated range for each soil and plotted against the LL , PI and SOM (see Fig. 12). Figure 12 shows a tendency for an increasing C_{ae} with the increasing LL , PI and SOM . These results were consistent with those of Bjerrum et al. [8], Nakase et al. [53], Yin [70], Jesmani et al. [32] and Zhu et al. [82]. Present study and the results of the above-mentioned researchers showed that plasticity and organic matter to a great extent influence the time-dependent behaviour of fine-grained soils.

4.3 The effect of the PI and R_p on m

To examine the effect of the soil plasticity on m in the $\log C_{ae} - \log e$ relationship, two approaches were considered:

- m as a function of plasticity index PI .
- m as a function of the plasticity ratio R_p .

Figure 13 shows the plots of m with PI and R_p for the available test results on 11 intact alluvial organic soils. It was found that the accurate description of the dependency

Fig. 9 An example of the identification parameter for the analysis: **a** relationship between C_{ae} and effective vertical stress; **b** bilogarithmic approach for the determination of the preconsolidation pressure; **c** relationship between C_{ae} and ratio of σ'_v / σ'_p ; **d** double-logarithmic approach for the determination of the reference values of C_{ae} . Note that: symbols e_L and e_p are void ratios at the liquid limit and plastic limit, respectively; C_{aeL} and C_{aeP} are relevant coefficients of secondary compression index

Fig. 10 Stress-rate dependency of the investigated soils

Table 2 Threshold values of the σ'_T/σ'_p ratio

Sample	σ'_p (kPa)	σ'_T (kPa)	σ'_T/σ'_p (-)
orSiV	70	100	1.43
orCII	28	100	3.57
orsaSiH (A)	25	100	4.00
orsaSiH (B)	20	50	2.50
orsaSiH (C)	30	50	1.67
orsaSiH (D)	30	50	1.67
orsaSiH (E)	20	100	5.00
orsaSiH (F)	29	100	3.45
orsaCII (A)	18	50	2.78
orsaCII (B)	25	50	2.00
orsaCII (C)	30	50	1.67

between the plasticity expressed as PI or R_p and parameter m is an exponential equation. Therefore, the exponential regression analysis was executed on the data, and the $m-PI$ and $m-R_p$ relationships were obtained. As shown in Fig. 13a, the parameter m decreased with the increase in the PI . Thus, soils with considerable size of the range of water contents where the soil exhibits plastic properties have small value of m . In this case, soils with higher plasticity exhibit slow creep degradation rate. From the plot of m with PI , the following relation could be obtained with a correlation coefficient $R^2 = 0.82$:

$$m = 676.80PI^{-1.49}, \quad (5)$$

Similarly, Fig. 13b shows a general trend for the $m-R_p$ relationship: with decreasing R_p the m value decreases. As can be seen, the m values were lower for the soils with higher plasticity. According to the experimental evidence, the R_p values of the tested soil were in the range of 0.4–0.63. This results correspond to the ordinary general sediment soils range of approximately 0.2–0.8 [57]. From the plot of m with R_p , the following relation could be achieved with a correlation coefficient $R^2 = 0.87$:

$$m = 0.07 \exp^{7.93R_p}, \quad (6)$$

The influence of the plasticity expressed by the R_p on the creep degradation rate is significant and larger than those determined with the PI . PI and R_p characterize plasticity of the soil and both the high value of the PI and low value of the R_p indicate high plasticity. Therefore, in this case resulting C_{ae} degradation rate is low. The value of R^2 in Eq. (6) is relatively higher compared to its value in Eq. (5). From these results, it appears that the choice of R_p as the correlating parameter for the m is preferable, because it gives the higher correlation coefficient.

Interestingly, $m-PL$ and $m-LL$ relationships do not give the definite trends and the data are more scattered. Yin et al. [74] correlated the PL with the slopes of $\log C_{ae}-\log e$ plot and observed that for a database of mainly reconstituted Finnish clays the parameter m can be approximately determined by the PL of soils by a power-type formula ($R^2 = 0.8423$). However, the direct comparison of the test results for soils that are in a different state

Fig. 11 Creep degradation curves in the $\text{Log } C_{ae}$ – $\text{Log } e$ plane

Fig. 12 Effect of the liquid limit, plasticity index and organic matter on the C_{ae}

(reconstituted and intact) as well as from different geological origin is not appropriate and cannot give clear conclusive remarks. Furthermore, soils could be from the same geological origin; however, with a different R_p , they can behave very differently [40]. The differences might be due to the different clay minerals, their proportions and the different clay size proportions. Experimental evidence from the literature shows the clear trend, where lower R_p values correspond to higher clay percentage to fines PC (%) values [57]. Figure 14 shows the R_p – PC semilogarithmic relationship for the investigated soils. As a result, the smaller is the R_p , the higher the PC . Therefore, it can be assumed that the PC also affects the value of m . Hence, m was plotted with PC in the linear plane (see Fig. 15). Results show a clear trend, where lower m values

Fig. 13 Influence of the plasticity of the soil on m : **a** effect of plasticity index on m ; **b** effect of plasticity ratio on m

correspond to higher PC values (based on power-type formula with $R^2 = 0.83$). Resulting trend also supports the validity of m – PI relationship, because PI of the soil

Fig. 14 Relation between plasticity ratio and percentage clay to fines

Fig. 15 Effect of percentage clay to fines on m

strongly depends on PC . Considering the types of soil, m is smaller for clay (lower R_p and higher PI along with PC) than for silt (higher R_p and lower PI along with PC).

Figure 16 compares the calculated m values using a double-logarithmic approach to that derived using

Fig. 16 Comparison of two methods for the determination of m

developed correlations. To examine the robustness and assess the performance of the empirical models (bearing in mind that the R^2 may not give sufficient information to determine the validity of the correlation [66]), the standard error of estimate $S_{y,x}$ is used:

$$S_{y,x} = \sqrt{\frac{\sum_{i=1}^N (Y_m - Y_p)^2}{N}}, \quad (7)$$

$S_{y,x}$ is the measure of the scatter around the regression curve [62] and explains how wrong the regression model is on average, using the units of the response variable. Approximately 95% of the observations should fall within plus/minus $2S_{y,x}$ from the fitted line, which is also an approximation of a 95% confidence interval. Smaller values are better because it indicates that the observations are closer to the fitted line. The scattering of the points in both studied relationships was clearly along the equality line. The correlations using PI and R_p produced values for m close to the experimental values derived from the double-logarithmic approach in the case of soils with higher plasticity. In other words correlations are more suitable for fine-grained soils with the small values of m . The largest discrepancies were observed for the interval $4 < m < 6$ for the values obtained using R_p . Considering data produced with the use of PI , the scattering was high for the $m > 4$. It led to the conclusion that an empirical models are more accurate for high plasticity soils; however, it must be used with great care. The invented correlations were established for soils from one area and of the same geological origin. Consequently, more research and more research data are needed to confirm their validity. Nevertheless, the trends found and the relationship between changes in soil creep parameters and plasticity parameters allow for a better understanding of the mechanism of the time-dependent visco-plastic straining.

5 Conclusions

The time-dependent behaviour of intact alluvial organic soils was investigated by means of the basic soil properties such as PL , LL and PC , which are commonly identified during laboratory analysis. This paper presents the results of experimental research to determine the effect of the soil plasticity on the creep behaviour of natural clayey soils. The results of experiments on 11 types of soil with the same geological origin showed that $C_{\alpha e}$ was both stress and rate-dependent. It was found that soil structure has a significant influence on the pattern of the $C_{\alpha e}-\sigma'_v$ curve with the characteristic threshold stress which corresponds to the maximum value of $C_{\alpha e}$. The pattern of the $C_{\alpha e}-\sigma'_v$ curve has a concave downward shape for all the soils. To model

the creep degradation rate in the normally consolidated states, the double-logarithmic approach was successfully applied. The results of the analysis indicate that the smaller value of the parameter m refers to the low creep degradation rate which typically occurs in clays and clayey soils. It should also be noted that m strongly depends on the plasticity of the soil expressed by PI and R_p . The following conclusions were drawn based on the findings of the study:

1. The PI and R_p have a significant influence on the value of m . Soils with both higher value of PI and lower value of R_p indicated higher plasticity. For the investigated soils, a definite trends were observed: while PI increased, m decreased and while R_p decreased, m also decreased.
2. Clay percentage to fines affects soil's plasticity and plays an important role in the creep degradation process. Soils with higher values of PC have smaller values of m and lower creep degradation rates.
3. The relationships are valid only for soils with the same geological origin, and in the limited range of PI and R_p . Further investigations should be conducted that include intact fine-grained soils with different geological origins.
4. Considering the type of the soil, low creep degradation rates occur in clays than in silts and other less-plastic soils.

Declarations

Conflicts of interest The author declares that they have no conflicts of interest.

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