Evaluation of consolidation parameters in CL tests; theoretical and practical aspects

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The paper presents theoretical solutions of the consolidation problem with respect to the different conditions of continuous loading and its application. The author introduces modified consolidation parameters and dimensionless parameters characterizing the course of the consolidation process. Therefore it is possible to calculate the theoretical pore water pressure distribution for various loading procedures in continuous loading (CL) consolidation tests occurring in constant rate loading (CRL), constant rate of strain (CRS) or controlled gradient (CG) tests. The calculation results allow presentation of the attributes that differentiate CL consolidation and classical incremental loading (IL) consolidation. A new method of calculation c_v (coefficient of consolidation) is proposed using theoretical diagrams of pore water pressure distribution and results of laboratory measurements during the CL test. A comparative analysis of the methods currently used for c_v calculation and the new method is presented here. The c_v values estimated by means of method referring only to the seepage factor of consolidation, are usually higher than those based on the strain course. Proper projection of the seepage factor of consolidation makes it possible to shorten the time of consolidation tests in accordance with results of many field observations. The methods described herein can be useful in studying physical conditions of sedimentation, glacial geology, early diagenetic process and applied geology.

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INTRODUCTION

Theoretical and experimental analyses of soil consolidation are very important in forecasting changes in the geological environment. Strain in soils and increase and dissipation of pore water pressure are observed generally under loading of soil in natural and man-made processes.

Among natural factors causing various consolidation scenarios there are changes of: sedimentation rate, hydrogeological relations and glacial overburden. Anthropogenic impact can modify the geological environment, often significantly quicker than can natural processes. Sometimes loading of soil at engineering sites, dumps and landfill sites partly replicates the geological history of loading. A significant reason for settlement is changes of underground water level during mining drainage or groundwater exploitation. In anthropogenic impact phenomena we may observe results of processes which occurred in the geological past but which lasted for much longer.

The theory of consolidation with respect to filtration conditions is a very useful tool for geological analyses. The factor of time and rate of loading which are very important in geological process can be studied in applying this theory. In this theory different lengths of pore water drainage can be used as a scaling factor of consolidation time (Dobak, 2003). Analysis of such phenomena is possible thanks to the development of theoretical descriptions and the experience gained from laboratory tests. From the beginning of the 1960's, methods of testing consolidation using continuous loading (CL tests) have been gradually implemented. Initial distrust towards this method was caused by conflict with earlier loading procedure. The new approach was not consistent with theoretical solutions of Terzaghi's classic theory which analyses the process of soil strain and the distribution of pore water pressure under constant values of loading (IL tests). However, CL tests are better for presenting the natural continuity of geological and environmental processes. They also enable a wider range of stress to be obtained, corresponding to the field conditions, with fewer and less laboratory and interpretation mistakes. An additional advantage of CL

tests in geological applications is the possibility of modelling various loading rates which may correspond to changes of sedimentation or of external loading conditions.

Analysed methods of interpretation of CL tests are based on the assumption that pore space is fully saturated by water. This is similar to the conditions in young deposits. It should also be noted that consolidation of soil may be perceived as the earliest stage of diagensis. Therefore solutions obtained from theoretical and experimental analyses may provide new inspiration in the studies of sedimentary and diagenetic conditions.

PROGRAMS OF CL TESTS

The crucial parameter describing the consolidation process based on Terzaghi's assumptions is the coefficient of consolidation c_v. This parameter in continuous loading tests (CL) is obtained under different laboratory conditions than in traditional IL tests. However, the results are becoming generally accepted and used by researchers because:

— CL tests are short and the results include fewer errors than the long-term IL tests;
— the c_v values obtained from CL tests enable forecasting of settlement with better consistency with field observations;
— the seepage factor of the consolidation process is better reflected by the CL tests; this is necessary for proper scaling of the process duration with respect to the drainage path lengths in the laboratory and under field conditions (Dobak, 2003).

However, while employing CL tests, we still meet some problems in properly adjusting to observed field condition. There are several suggestions concerning loading mode in CL tests:

— CRS — the constant rate of strain tests (Smith and Wahls, 1969; Wissa et al., 1971);
— CG — the tests with controlled gradient loading adjusted to keep the base pore water pressure constant (Lowe et al., 1969);
— CRL — the constant load rate tests.

There is no procedure which could be used to compare results of different test methods, and recommendations for their application, in particular geotechnical cases:

— The c_v values estimated in CL tests are too high at the beginning, which is related to the unreliable transient (non steady) phase of the test. There is no precise criterion describing the attainment of this phase.
— The c_v values calculated according to formulae given by various authors often vary; thus, selecting the best calculation formula may pose a problem.
— The views concerning selection of a proper test rate are very different and controversial (Lee et al., 1993; Almeida et al., 1995).

The key to solve these problems seems to be an analysis of a theoretical model of consolidation taking into account different ways in which load is increased.

The paper presents a proposal of theoretical pore water pressure estimation under continuous loading. Changes of pore water pressure are analysed here because this factor has a major role in the seepage of water through the soil and determines the rate of settlement. When we are able to compare theoretical solutions and experimental results we may assess the correctness of consolidation solutions and the procedures of laboratory tests employed.

REVIEW OF CALCULATION METHODS FOR c_v ESTIMATION IN CL TESTS

The basic formula introduced by Smith et al. (1969) combines load rate $\Delta\sigma/\Delta t$ with the decreasing length of drainage path H_i and pore water pressure u_b increasing in the course of the test:

$$c_v = \frac{\Delta\sigma \cdot H}{\Delta t_{CL} \cdot 2u_b} \tag{1}$$

where: H — length of drainage path; σ — stress; t_{CL} — time from the start of CL test; u_b — pore water pressure.

According to the equation [1], with an initially very low value of pore water pressure, a very high (unrealistic) value of the coefficient of consolidation c_v is obtained. Mobilisation of the pore water pressure is expressed by initially fast increase and then stabilisation, or continuation of its increase. As a consequence, the value of the coefficient of consolidation falls quickly and then is stabilized or decreases slightly. The c_v value is quasi-stabilized which is consistent with Terzaghi's consolidation theory in conditions of constant loading.

Other suggested solutions dealt with the unreliability of c_v at the initial stage of the test and involved various corrections to the basic formula.

Wissa et al. (1971) presented a formula that allowed non-linearity of the consolidation process:

$$c_{v\,\text{non linear}} = -\frac{0.434 \dfrac{u_b}{\sigma}}{\log\left(1 - \dfrac{u_b}{\sigma}\right)} c_{v\,\text{linear}} \tag{2}$$

where: $c_{v\,\text{linear}}$ — calculated from formula [1].

Therefore overestimated initial values of the coefficient of consolidation and their slight decrease at the beginning of the experiment are limited in comparison with the results for the basic formula [1].

The formula recommended by ASTM [3]:

$$c_v = -\frac{H^2 \log \dfrac{\sigma_2}{\sigma_1}}{2\Delta t_{CL} \log\left(1 - \dfrac{u_b}{\sigma}\right)} \tag{3}$$

gives results similar to those obtained using the formula [2].

Janbu *et al.* (1981) introduced a method of correcting overestimated values at the initial stage of the test. The values calculated by means of the basic formula [1] are multiplied by coefficient — α_{c_v} calculated as follows:

$$\alpha_{c_v} = \frac{2\left[\cos h\,(a)-1\right]}{(a)^2 \cos h\,(a)} \qquad [4]$$

where: $\lambda = \dfrac{\Delta u_b}{\Delta\sigma}$ and $a = \operatorname{arccos} h\left(\dfrac{1}{1-\lambda}\right)$

While using this solution, a low value of α_{c_v} is obtained with a significant increase of the pore water pressure. As a result, a desirable decrease of the c_v value occurs at the initial stage of the test.

When pore water pressure becomes constant in the course of the CL test, $\lambda \to 0$, $\alpha_c \to 1$ and the c_v value is the same as the one obtained from the basic formula [1]. The Janbu *et al.* (1981) solution does not change the c_v value for the "steady" phase of CL tests.

Figure 1 presents the schematic model of c_v changes according to different employed formulas for the assumed quasi-stabilised value of c_v. The diagrams present the dependencies described above.

However, limited, the variability of the c_v values presented above calculated from the CL tests needs to be compared in detail with reference to:
— various loading systems;
— changes of permeability and consolidation parameters in the course of increasing loading;
— assumptions and implications of the classical theory of consolidation.

A loading system is crucial for the course of the CL test. Figure 2 presents a scheme of mutual relations between deformation, stress and strain rate for CRL and CRS tests.

It is clearly visible that keeping a constant load rate causes significant change of the strain rate, particularly for normally consolidated soils. On the other hand keeping a constant strain rate in the CL tests causes a curvilinear dependence of $\sigma = f(t_{CL})$.

Various loading procedures have to be taken into account while analysing the theoretical course of the CL consolidation process, and comparing the dependencies obtained with results of laboratory tests.

PRINCIPLES OF THEORETICAL COMPUTATIONS

PARAMETERS

Application of Terzaghi's theory to describe the CL consolidation process under continuous loading requires development and conversion of the consolidation parameters.

The following parameters have been used in the theoretical analysis:
— pore water pressure — u_b, measured at the bottom of the sample, when drainage is directed to the upper surface;
— dimensionless parameter of pore water pressure $C = u_b/\sigma$ named as C_{IL} for IL tests, and as C_{CL} for CL tests,
— new parameter — a specific consolidation time $t_{(T=1)}$. Figure 3 presents the dependence between the values of the $t_{(T=1)}$ parameter and the coefficient of consolidation for a typical length of a drainage path, obtained in CL tests. Introduction of the $t_{(T=1)}$ parameter allows calculation of pore water pressure distribution with regard to the changes of H_i length of the drainage path. This point
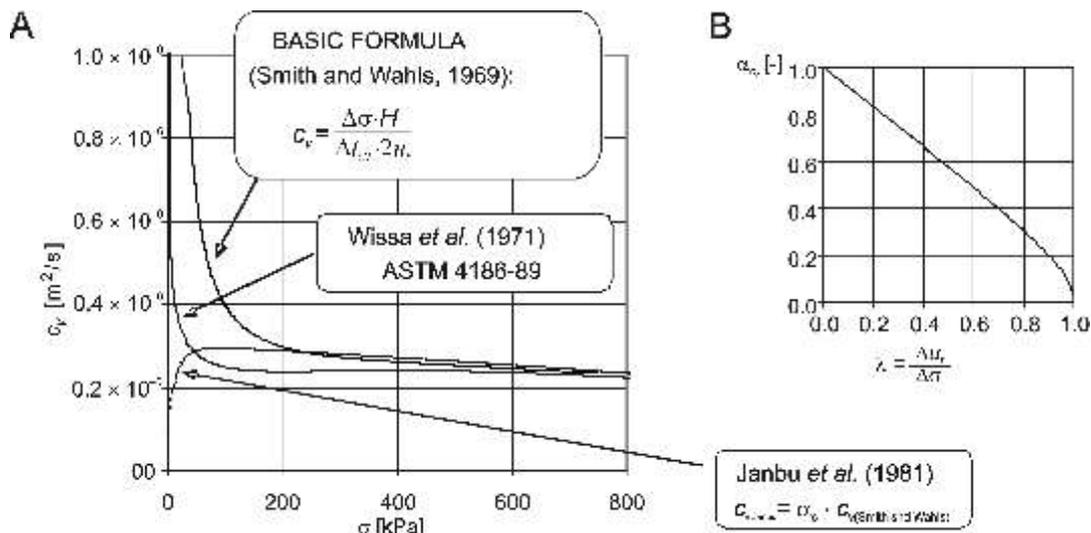
Fig. 1. Comparison of c_v characteristics estimated by various methods

A — estimation of c_v value by different formula; **B** — changes of c_v v. λ
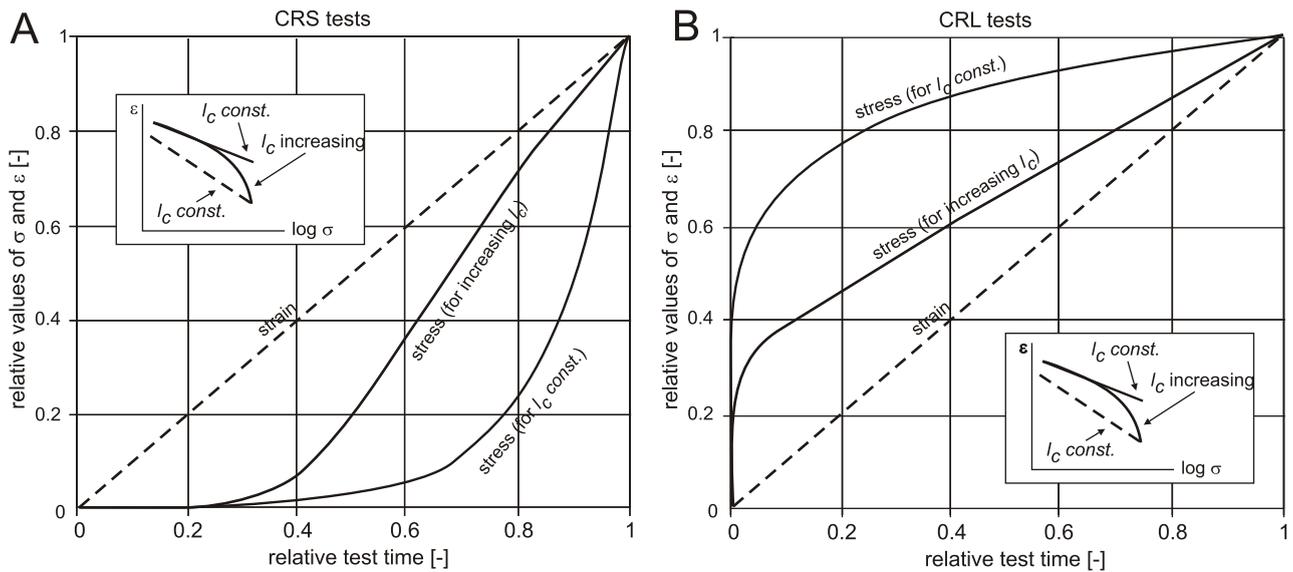
Fig. 2. Stress and strain changes in the CRS tests (A) and CRL tests (B)
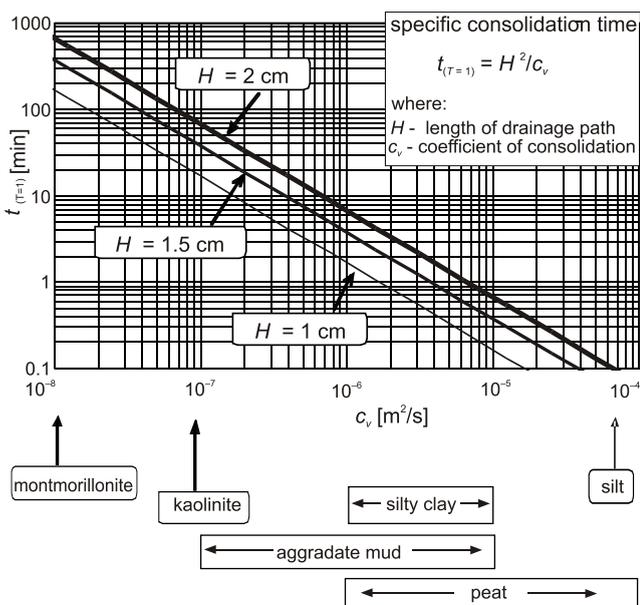
Fig. 3. Dependence between the coefficient of consolidation c_v and the specific time of consolidation $t_{(T=1)}$ for various lengths of the drainage path

Typical values of c_v after Gudehus (1981)

of view was taken into consideration in analysis of c_v changes by Butterfield and El-Bahey (1995). Because changes of H observed in the CL tests can be greater than these in the IL tests, they should be taken into account while analysing the entire consolidation process.

— a new parameter — relative time of the CL consolidation — T_{CL}. This is a relation of time t_{CL} from the start of the CL test and the current value of the $t_{(T=1)}$ parameter.

The consolidation degree U used in IL tests is useless for CL tests because completion of the consolidation process can-

not be specified in conditions of a constant increase of loading. According to the theoretical analysis the dimensionless parameter T_{CL} described above reveals specific connection with the features of pore water pressure distribution. Thus, it may constitute a measure of progress in the consolidation process for IL and CL tests. Its advantages will be shown in the theoretical analysis.

CALCULATIONS

The purpose of calculations was to estimate the development of theoretical pore water pressure distribution under continuous loading conditions.

The input data were:
— continuous changes of loading during the test $= f(t_{CL})$;
— changes of properties of consolidated soils occurring in the course of the test; they are expressed by the function $t_{(T=1)} = f(t_{CL})$;
— assumed discretization of test course and consolidation parameters.

The author conducted variant analyses of the calculation discretization. Calculations may be conducted assuming the initial discretization of the stress for instance $\Delta\sigma$ = const or of the time parameters. According to the theoretical analysis, assuming $\Delta\sigma$ = const to be the discretization criterion, the calculation results are not comparable for various values of consolidation parameters. However, in the other case, when $\Delta t_{CL}/t_{(T=1)}$ is the discretization criterion, the calculation results do not depend on changes of the soil properties. When $\Delta t_{CL}/t_{(T=1)}$ the error $\Delta C_{CL}/C_{CL} \leq 1\%$ occurs, which is acceptable while comparing the laboratory and field-test results.

One can assume that $\Delta t_{CL}/t_{(T=1)}$ the discretization steps of $\Delta\sigma$ became different in the course of the test. For the next steps of $\Delta\sigma_n$ calculated as above, excess of the pore water pressure

THE SCHEME OF CALCULATIONS – ALGORITHM "SUM"

INPUT DATA	relationships: $\sigma = f_\sigma(t_{CL})$ and $t_{CL} = F_\sigma(\sigma)$ $t_{(T=1)} = f_T(t_{CL})$ calculation (discretization) step: $CDS = \dfrac{\Delta t_{CL}}{t_{(T=1)}} = const$

sequences of values:

* $t_{CL\,0}, t_{CL\,1}, \ldots, t_{CL\,i}$ where: $t_{CL\,i} - t_{CL\,i\text{-}1} = \Delta t_{CL\,i} = CDS \cdot f_T(t_{CL\,i\text{-}1})$

* $(t_{(T=1)})_0, (t_{(T=1)})_1, \ldots, (t_{(T=1)})_i$, where: $(t_{(T=1)})_i = f_T(t_i)$

* $\sigma_0, \sigma_1, \ldots, \sigma_n$ according to $\sigma_n = f_\sigma(t_i)$

calculations for each cell (i, n) of 2D space (t_{CL}, σ) defined for

* $t_{CL} \subset\ <t_{CL\,0}, t_{CL\,i\,max}>$
  where: $t_{CL\,i\,max}$ correspond to the end of test

* $\sigma \subset\ <\sigma_0, \sigma_{n\,max}>$
  where: $\sigma_{n\,max} = f_\sigma(t_{CL\,i})$
  (for any time below the line $\sigma = f_\sigma(t_{CL})$)

$$T_{(i,n)} = \frac{t_{CL(i)} - F_\sigma(\sigma_n)}{t_{(T=1)\,for(i-1)}}$$

$$C_{IL\,(i,n)} = \left[\sum_{M=\pi/2}^{\infty} \frac{2}{M} \sin\frac{Mz}{H} \exp\left(-M^2 T_{(i,n)}\right)\right]$$

where

* $M = (\pi/2)(2m+1)$ for $m = 1, 2, \ldots$
* z - coordinate in vertical direction
* $z/H = 1$

$$\Delta u_{b\,(i,n)} = C_{IL\,(i,n)} \cdot (\sigma_n - \sigma_{n\text{-}1}) = C_{IL\,(i,n)} \cdot \Delta\sigma_n$$

$$C_{CL(i)} = \frac{\sum\limits_{n=1}^{n\,max} \Delta u_{b(i,n)}}{\sigma_{n\,max(i)}}$$

Fig. 4. The block scheme of calculation — the SUM algorithm

L — linear increase of $t_{(T=1)}$; $C_{0.1}$ — constant value of $t_{(T=1)} + 0.1\,t_{max}$;
$C_{0.1}$ — constant value of $t_{(T=1)} + 0.2\,t_{max}$

$\Delta u_{b\,(i,n)}$ is estimated with regard to pore water pressure dissipation in the time t_{CL} function.

As a result we obtain a two-dimensional table containing results of $\Delta u_{b\,(i,n)}$ calculation for every level of $\Delta\sigma_n$ and times $t_{CL\,i}$.

The addition of $\Sigma\Delta u_{b\,(i,n)}$ for successive time $t_{CL\,i}$ is the final result of discretized calculation as a theoretical value of the dimensionless parameter $C_{CL,\,i}$ Figure 3. Dependence between the coefficient of consolidation c_v and specific time of consolidation $t_{(T=1)}$ for various length of the drainage path.

The calculation with formulas applied is shown in Figure 4.

THEORETICAL ANALYSIS OF THE CALCULATION RESULTS

The key issue for both theoretical and experimental analysis of consolidation is the course of pore water pressure changes. The purpose of theoretical calculations based on the assumptions noted above is to analyse pore water pressure distribution under continuous loading. These conditions have not been a subject of classical consolidation analysis.

The solutions presented from theoretical calculations (Dobak, 1999) extend the characteristics of consolidation process for a continuous loading program.

The basic factors influencing variability of pore water pressure distribution are examined alternatively as stable and variable consolidation characteristics of the soil (expressed as $t_{(T=1)}$) and different scenarios of loading increase.

Because of the fact that the analyses presented are general, the input parameters have been reduced to comparable the dimensionless variables. Initial time t_{CL} and stress σ have the value 0 here and the final ones 1. Calculated u_b values are presented as a fraction of maximum stress value and the results of analyses are presented as the dimensionless parameters (T_{CL}, C_{CL}) defined above.

Referring to the assumptions discussed the analysis was conducted for the following conditions:

— Three cases of consolidation features expressed by the specific time of consolidation ($t_{(T=1)}$). Two variants of $t_{(T=1)} = const$ marked as lines C$_{0.1}$ and C$_{0.2}$ (Fig. 5B) were included. The values in index correspond with the relation $t_{(T=1)}/(t_{CL\,max} = 1)$. The case of linearly increasing $t_{(T=1)}$ is marked as line L (Fig. 5A).

— Various values of load rate: two different values of constant load rate (the CRL conditions) marked on Figure 5B as I and II; two models of the increasing load rate (Fig. 5C) for conditions similar to the CRS test marked on Figure 5C as III and IV.

The results of pore water pressure distribution in CL conditions are described below.

Figure 5D, E present theoretical changes of the pore water pressure value. Initially, a considerable increase of pore water pressure and then a quasi stabilisation of the u_b values are observed for the linear increase of stress (the CRL tests, I and II cases) and constant values of $t_{(T=1)}$. The quasi stabilisation of pore water pressure takes place when the consolidation degree U for the first infinitesimal increase of stress is equal to 0.99…. and the T parameter from the classical Terzaghi and Taylor solution is equal to 2, which means when $t = 2t_{(T=1)}$. The further linear stress increase results in the constant value of $u_{b\,stabil}$.
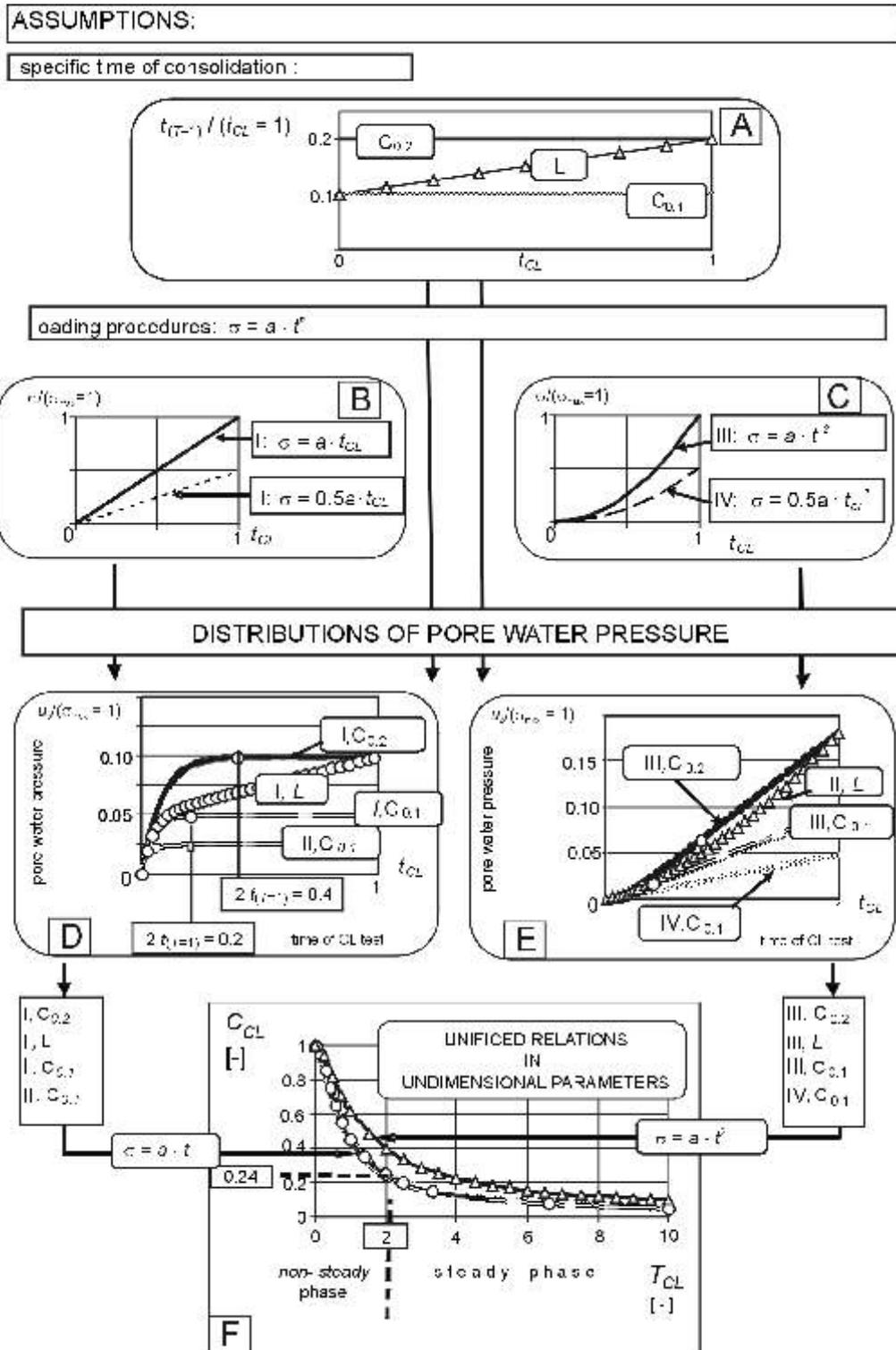
Fig. 5. Theoretical pore water pressure distribution during the CL tests

The relative values of time are comparative in all the diagrams

The $u_{b\ stabil}$ value depends on $t_{(T=1)}$ value (compare I, $C_{0.2}$ and I, $C_{0.1}$ cases) and load rate (the I, $C_{0.1}$ and II, $C_{0.1}$ cases).

During CRL tests with a linear increase of $t_{(T=1)}$ we do not observe stabilisation of pore water pressure but a quasi linear increase of u_b between extreme lines (I and II) for $t_{(T=1)}$ = const on the diagram (Fig. 5D).

When the load increase is not linear and $\Delta\sigma/\Delta t^2 > 0$, a considerable constant increase of pore water pressure is observed. It appears that stabilisation of pore water pressure occurs during CRL tests only with $t_{(T=1)}$ = const (Fig. 5D).

At the same time it is worth noting that intentionally maintaining the constant value of pore water pressure (the CG test) requires proper control of the load rate change. It is possible only when the measurement apparatus is specially interactively programmed.

Results of the analyses presented show that the theoretically expected stabilization of pore water pressure in the CL test may occur only if two conditions are met at the same time:

$\Delta\sigma/\Delta t$ = const: (CRL tests)

$$t_{(t=1)} = \frac{H^2 \cdot \gamma_w}{k \cdot M_0} = \text{const} \qquad [5]$$

where: γ_w — unit weight of water; k — coefficient of permeability; M_0 — modulus of compressibility.

In practice, changes of the H, k, M_0 parameters cause a increase of the $t_{(T=1)}$ value. Therefore not only in CRS and CG tests, but also in CRL tests we can observe a continuous increase of pore water pressure.

Unification of description of pore water pressure distribution in the course of CL tests is obtained by means of introducing dimensionless parameters.

When the C_{CL} parameter is on the y-axis and T_{CL} is on the x-axis we obtain only two curves for every case analysed. Standardisation of the diagrams shows that the course of the C_{CL}–T_{CL} curve depends only on the model of loading increase

(Fig. 5F). One curve characterizes every CRL test independently of the rate of loading. We obtain other curves while the increase of loading is expressed by the $\sigma = at^n$ function and the course of the C_{CL}–T_{CL} curve depends only on exponent index n. This situation usually occurs in the CRS test.

To sum up, curves analysed do not depend on the consolidation properties of the soil and depend only on load rate. An important item with regard to CL tests is $T_{CL} = 2$, because it constitutes a theoretical boundary between non-steady and steady phases of the process. This is the time when dissipation of pore water pressure caused by the first infinitesimal stress increase is practically finished. In the CRL test this limit is related to $C_{CL} = 0.24$, but in the CRS test C_{CL} for $T_{CL} = 2$ is higher.

THEORETICAL CHARACTERISTICS OF CL CONSOLIDATION

The results of calculations carried out for various assumed conditions confirm and extend the dependencies between the consolidation parameters presented above. As a result of presenting pore water pressure distribution by means of the dimensionless parameters C_{CL} and T_{CL} we may introduce generalised curves for different CL test procedures. The curves are independent of the consolidation parameters of soil (c_v), and depend only on load mode. For the tests with constant (linear) load increase, only one C_{CL} vs. T_{CL} curve is obtained. The curve characterises CRL consolidation just as the U vs. T curve characterises Terzaghi's theory. The factor responsible for course of the C_{CL} vs. T_{CL} curve is not the load rate ($\Delta\sigma/\Delta t$) but a change of stress increase ($\Delta\sigma/\Delta t^2 \neq 0$). Figure 6 presents a set of C_{CL} vs. T_{CL} curves obtained for test procedures defined by power functions $\sigma = a \cdot t^n$. The n value characterises particular curves.

Continuous loading in the IL tests may be treated as a special case of the power function $\sigma = a \cdot t^0 = a =$ const. Figure 6
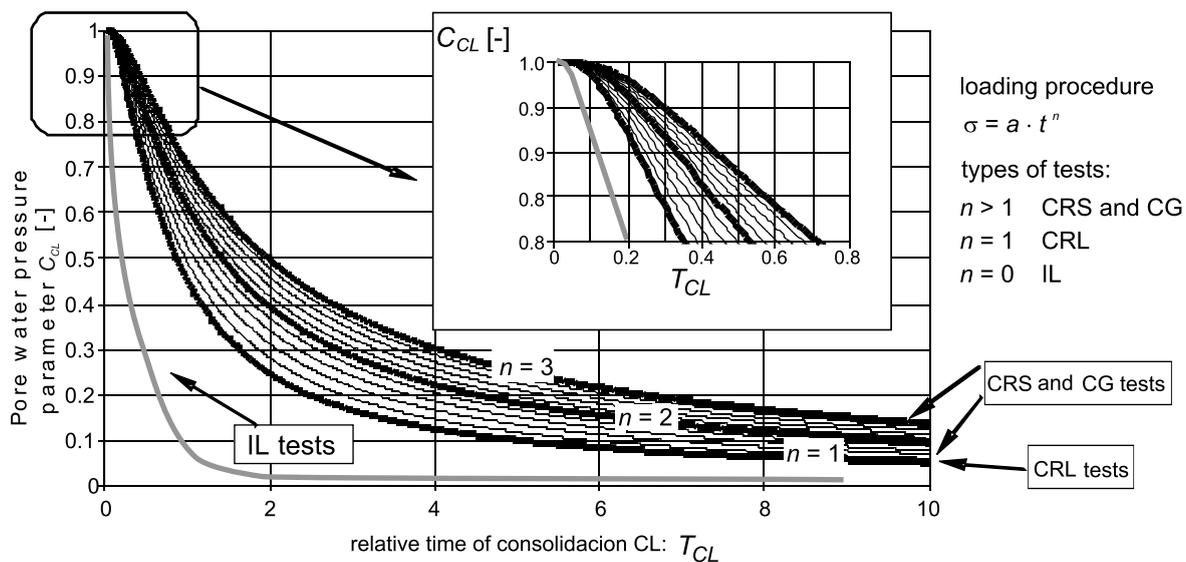
Fig. 6. Theoretical pore water pressure distribution for various loading procedures (after Dobak, 2000, modified)

also presents pore water pressure distribution according to Terzaghi's theory.

For loading procedures described by the multinomial function $\sigma = a_2 \cdot t^2 + a_1 \cdot t$ at $a_2/a_1 = $ const, a set of unified C_{CL} vs. T_{CL} curves is obtained.

It should be pointed out that modelling of loading procedures in CRS tests might be strongly individualised. The theoretical pore water pressure distribution is very sensitive to changes of loading procedure. Therefore selection of the multinomial function should be done very carefully.

The results of theoretical analysis of the CL consolidation enable:

— comparison of various c_v formulas against the background of Terzaghi's consolidation theory applied to CL tests;

— calculations of c_v on the basis of analysis of C_{CL} vs. T_{CL} curves;

— consistency of consolidation theory with experimental results.

CALCULATION OF c_v FROM CL TESTS — ASSESSMENT OF VARIOUS METHODS

Figure 7 presents assumptions and results of comparative calculations of consolidation parameters for various test procedures. The purpose of this analysis was to assess differences between c_v evaluated according to widely used formulae and the
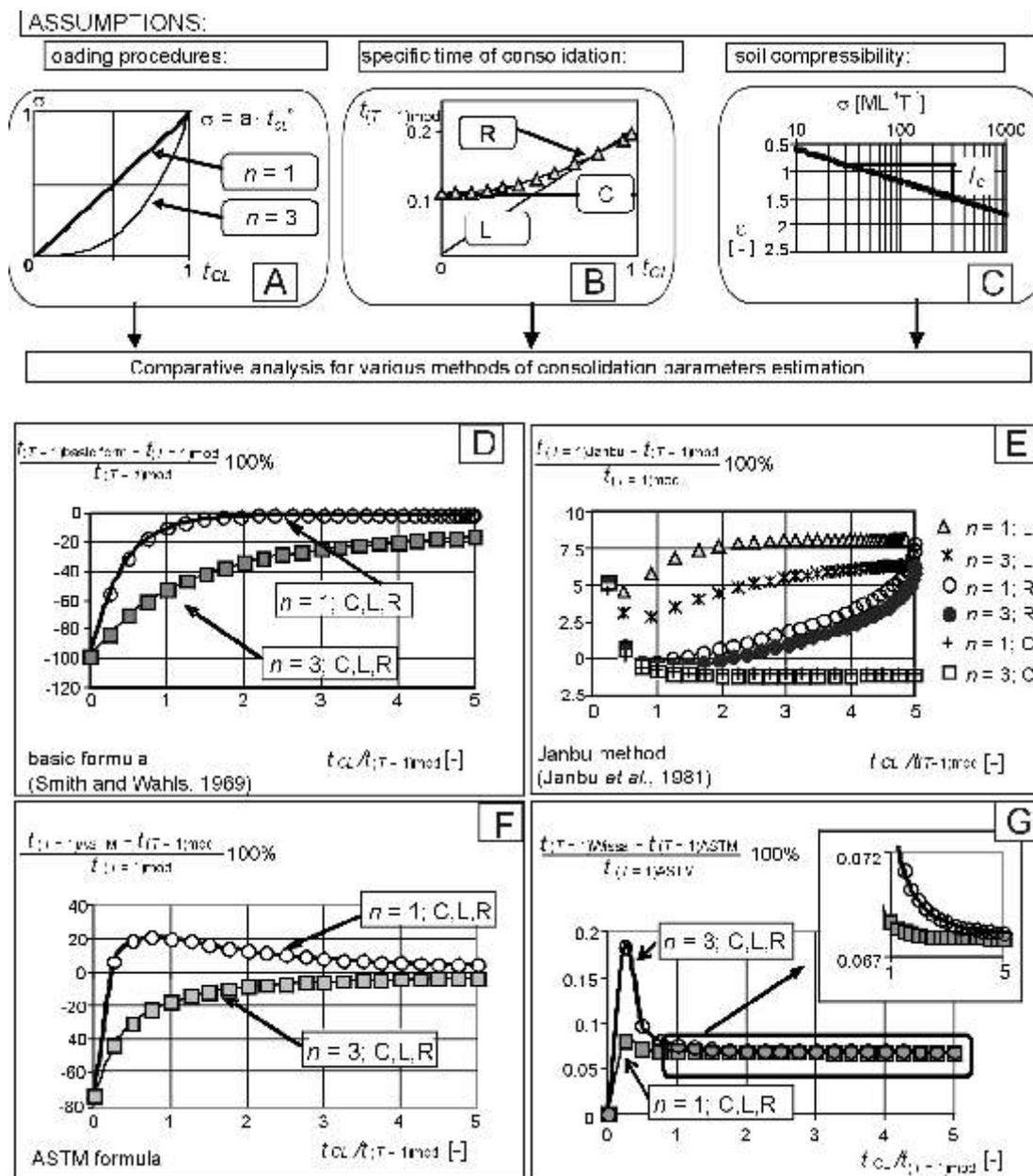
Fig. 7. Comparison of different methods of c_v estimation in the context of theoretical analysis
of the CL consolidation process

theoretical solution presented above. Assumed input data in dimensionless form is similar to Figure 5.

The calculation was carried out for the following conditions:

— two types of loading procedure;
— three types of soil with different consolidation properties; the properties were expressed by means of $t_{(T=1)\text{mod}}$ (Fig. 7B);
— the assumed soil compressibility for normally consolidated clay (Fig. 7C); this assumption was necessary to evaluate c_v, but calculation results were converted into $t_{(T=1)}$ using values of ε and H_i.

The assumed values of the $t_{(T=1)\text{mod}}$ and $t_{(T=1)}$ parameters calculated according to the formulae of Smith and Wahls, Wissa *et al.*, ASTM and Janbu, were compared. Further analysis would refer to the inversely proportional relation of c_v and $t_{(T=1)}$ (see Fig. 3) — increase of the $t_{(T=1)}$ value results in decrease of c_v and the opposite.

According to the basic formula (after Smith and Wahls, 1969) the $t_{(T=1)}$ values are underestimated in the initial part of the test and overestimated later on. The author observed this phenomenon in the majority of CL tests, but their quantitative assessment was impossible. That assessment would be possible after taking into account the results of calculation of theoretical pore water pressure distribution. The simulations conducted resulted in the following conclusions:

— a divergence of assumed values of parameters and values calculated according to Smith's formula occurs usually in the non-steady (transient) phase of CL tests ($T_{CL} < 2$);
— for the tests with an increasing load rate ($\Delta\sigma/\Delta t^2 > 0$), the divergence of $t_{(T=1)\text{Smith}}$ and $t_{(T=1)\text{mod}}$ is more visible and occurs not only at the beginning of the test but also at its later stage;
— the recorded divergences between $t_{(T=1)\text{Smith}}$ and $t_{(T=1)\text{mod}}$ depend on the assumed loading procedure and not on the consolidation parameters.

The correction coefficient introduced by Janbu changes reliability of the consolidation parameters in the initial phase of the test. This coefficient is evaluated on the basis of changes of the pore water pressure value which depends both on the loading procedure and on the variability of consolidation parameters [c_v, $t_{(T=1)}$]. Consequently divergence between $t_{(T=1)\text{Janbu}}$ and $t_{(T=1)\text{mod}}$ would also depend on both factors:

— in the case of soil showing constancy of the conventional time of consolidation (the C case), the $t_{(T=1)\text{Janbu}}$ value is close to the $t_{(T=1)\text{mod}}$ value; a small underestimation of $t_{(T=1)\text{Janbu}}$ in the stabilised phase (Fig. 7E) reflects similar underestimation of $t_{(T=1)\text{Smith}}$, that is not clearly visible in Figure 7D because of the diagram scale used;
— when the $t_{(T=1)}$ value still rises, there is an increase of pore water pressure even in the later stages of the test (for $T_{CL} > 2$); thus Janbu's correction coefficient still modifies consolidation parameters; as a result some overestimation of the $t_{(T=1)\text{Janbu}}$ values may be observed during comparison with $t_{(T=1)\text{mod}}$ values.

While using the formula recommended by ASTM, the divergences between $t_{(T=1)\text{ASTM}}$ and the assumed values depend on loading procedure:

— at the beginning there is a significant overestimation of $t_{(T=1)\text{ASTM}}$ values in the CRL tests, however, these gradually decrease;
— instead, $t_{(T=1)\text{ASTM}}$ values are underestimated in CRS tests.

The values of consolidation parameters calculated using ASTM's formula and Janbu's method are similar to each other (Fig. 7G).

A METHOD OF c_v ESTIMATION FROM PORE WATER PRESSURE DISTRIBUTION

Due to the peculiarities of C_{CL} vs. T_{CL}, it is possible to estimate the c_v value by means of a newly suggested method called the MD method (method of pore pressure dissipation). This method is based on theoretical curves of pore water pressure dissipation in various courses of continuous loading. We may analyse all CRL tests that produce uniform C_{CL} vs. T_{CL} curves. The following parameters are recorded during the test:

— load increase $\sigma = f(t)$;
— axial strain h, current length of the drainage path $H_i = H_0 - h$;
— pore water pressure u_b at the bottom of the sample.

We can estimate the C_{CL} parameter as u_b/σ at any time t and subsequently its corresponding value of T_{CL} using the C_{CL} vs. T_{CL} theoretical dependence. Then C_{CL} vs. T_{CL} is simply transformed into consolidation parameters: $t_{(T=1)}$ and c_v (Fig. 8).

In other CL tests (CRS and CG tests) the other $\sigma = f(t_{CL})$ function should be used depending on the compressibility of the soil. It is necessary to obtain a specific approximation of the function $\sigma = f(t)$ and calculate the relation of C_{CL} vs. T_{CL} by means of the SUM algorithm. Then the next steps of estimation should be as described above.

Figure 9 presents the results of of c_v value calculation according to all methods analysed. The c_v values form a range that is wider in the initial phase of the test and more narrow later on. The c_v values calculated using the MD method in general are similar to the highest estimations of c_v with the exception of Smith *et al.* basic formula in the initial part of the test.

ANALYSIS OF THE CONSISTENCY BETWEEN THE THEORETICAL SOLUTION AND EXPERIMENTAL TEST RESULTS

Consolidation of soils having various geneses were interpreted using the method of theoretical analysis presented above (Dobak, 1999). Typical patterns are presented herein. Properties of the soils analysed are listed in Table 1.

The results of laboratory tests have different divergences in the various parts of the tests. In the majority of cases the value of $C_{CL\,\text{max}}$ in the initial phase of the test is lower than the theoretical value $C_{CL\,\text{max}} = 1$. This may be caused by:

— delay of pore water pressure mobilisation under initial loading: incomplete saturation of pores with water in many samples; undistured structure and natural humidity of the samples; pre-consolidation effects.
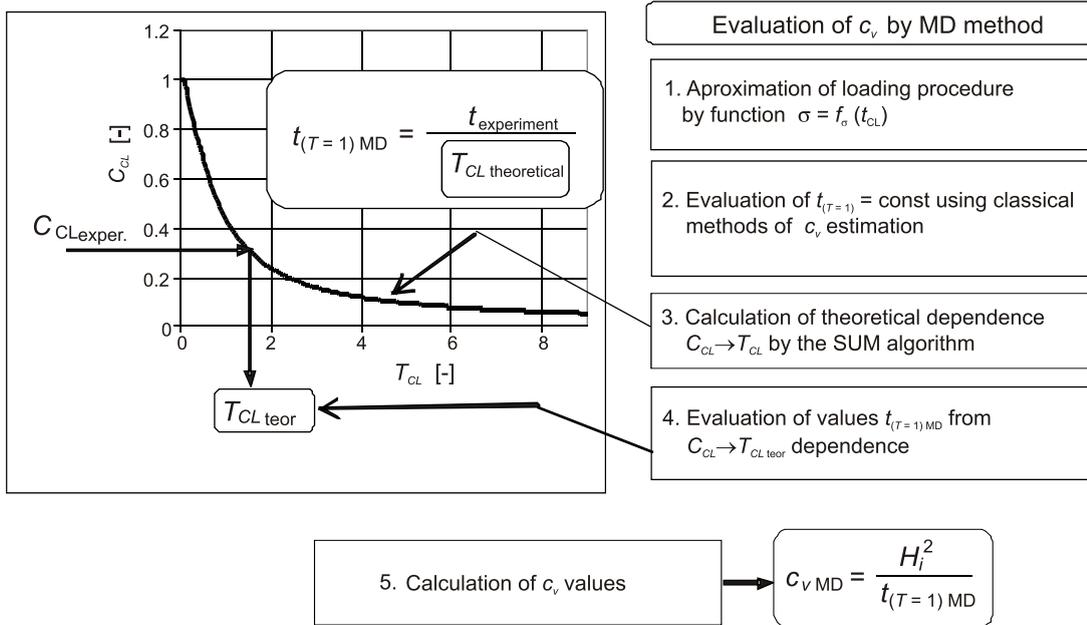
Fig. 8. Rules of c_v estimation from the theoretical pore water pressure distribution (the MD method)
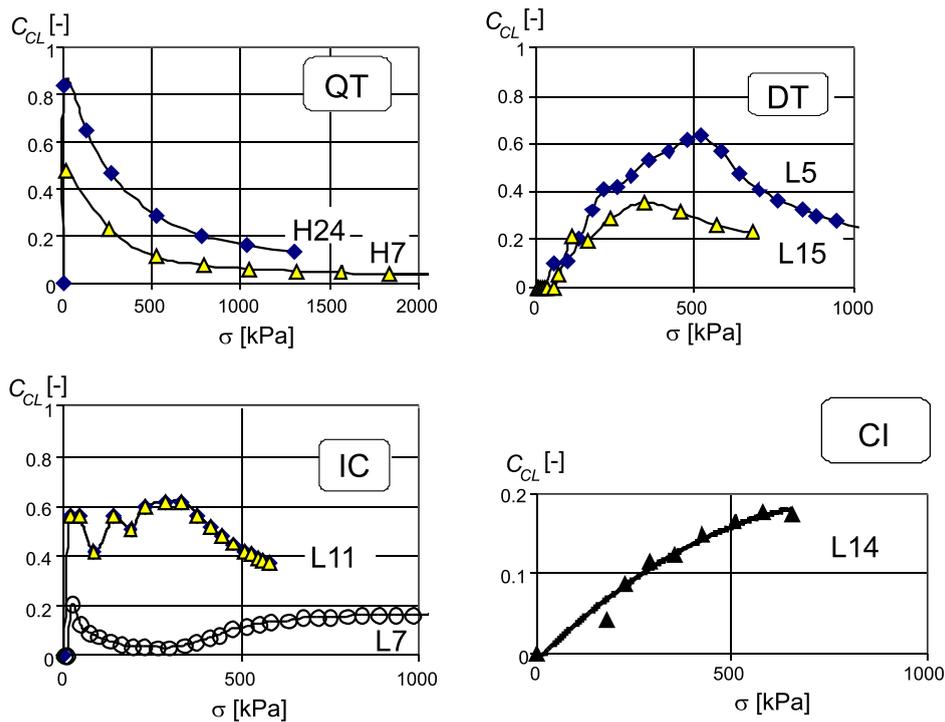
Fig. 9. Types of c_v distributions obtained from the tests

H24, H7, L5, L15, L11, L14 — tests (see Table 1)

The development of loading caused a decrease in soil porosity and an increase in the quasi-saturation of the pores. Therefore the distribution of pore water pressure in the subsequent part of the test is consistent with the model. Such behaviour can be named quasi-theoretical (QT) pore water pressure distribution.

There are three more types of C_{CL} vs. σ dependence observed, which are not in accordance with the theoretical solution presented above (Fig. 9):

— Displaced theoretical (DT) distribution: in the same tests pore water pressure distribution is delayed in compari-

Geotechnical properties of soils and conditions of CL tests

Type of soil		Sample no	Initial soil properties			Average velocity of CL test		Attaining $S_r = 1$ in the course of CL test	
			Density of soil ρ_0 [Mg $\cdot$ m^{-3}]	Water content w [%]	Initial degree of saturation $S_{r\,0}$ [-]	$\Delta\sigma/\Delta t$ [kPa s^{-1}]	$\Delta\varepsilon/\Delta t$ [s^{-1}]	$\sigma_{(Sr=1)}$ [kPa]	$_{(Sr=1)}$ [-]
Odra glaciation tills		H 8	2.17	11.9	0.85	0.37	$4 \cdot 10^{-5}$	110	0.050
Neogene complex	level III	H 24	1.71	33.5	0.92	1.23	$2 \cdot 10^{-4}$	130	0.090
	level IV	L 5	2.05	23.3	1.00	0.22	$3 \cdot 10^{-5}$	0	0
		L 11	1.86	31.4	0.96	0.11	$5 \cdot 10^{-5}$	43	0.027
	level VII	L 14	2.09	11.7	0.73	0.22	$2 \cdot 10^{-5}$	225	0.080
		L 15	2.04	15.2	0.80	0.19	$6 \cdot 10^{-5}$	116	0.071

son to the theoretically required situation. In this case the C_{CL} value gradually increases and reaches the maximum value lower than 1 in the more advanced phase of the test. Later its decrease is similar to the theoretical solution.

— Irregular variable (IV) distribution: in the tests of the non-consolidated soils (pastes) irregular changes of the C_{CL} value are sometimes observed. This can be caused by reconstruction of the soil structure and local changes of its permeability.

— Constantly increasing (CI) distribution. In this case the C_{CL} value increases constantly which is related to a rapid increase of the $t_{(T=1)}$ value in the course of the test.

After changing the co-ordinate system into C_{CL} vs. T_{CL} we obtain an equal route of the curves.

It is worth mentioning that, when the consolidation process is presented in the C_{CL} vs. T_{CL} system diagram, for other types of pore water pressure distribution interesting results are obtained as well. Due to converting the C_{CL} vs. σ system into C_{CL} vs. T_{CL} the experimental curve runs close to the theoretical one and T_{CL} values first decrease and than return up along the theoretical curve (Fig. 10). This is because of a significant increase of $t_{(T=1)}$ during the test. This may be a result of too high a load rate and a limited possibility of pore water pressure dissipation in the consolidated soil, especially in clays with a high value of the plasticity index (Dobak and Kowalczyk, 2008).

The author conducted some calculations assuming an increase of $t_{(T=1)}$ described by the function the $t_{(T=1)} = a \cdot (t_{CL})^b + c$. The results showed that a possibility of T_{CL} recession occurred when the second derivative of the function $t_{(T=1)} = a \cdot (t_{CL})^b + c$ (for a, b, $c > 0$) had a positive value (Dobak, 1999).

Analysis of C_{CL} diagrams may provide important information concerning assessment of the load rate in CL tests. The tests should be carried out with reference to the results of proper load velocity according to the field conditions (Dobak, 1995, 1999).

DISCUSSION

The suggestion for assessment and selection of the optimum CL consolidation parameters presented above is closely con-

nected with Terzaghi's solution. Terzaghi's theory of consolidation, introduced in the 1920's, has for many years been a basis for further advanced theoretical solutions (Davis and Raymond, 1965; Gibson *et al.*, 1967; Fredlund and Rahardjo, 1993; Butterfield and El-Bahey, 1995) and is still often used for settlement forecasts. The most interesting element seems to be the solution of the most difficult problem — scaling of the process time. Terzaghi explained the similarity of the consolidation time in the laboratory tests and field observations by a physical connection between settlement time and pore water drainage conditions. Due to that, a long-lasting settlement forecast concerning the soil layer might be connected to the results of short laboratory tests. According to Terzaghi's theory, the settlement time is proportional to the square length of the drainage path, although results of further laboratory experiments indicate a necessity of more general presentation of this dependence:

$$\frac{t_{1lab}}{t_{2field}} = \left(\frac{H_{1lab}}{H_{2field}} \right)^n \qquad [6]$$

where: the n value is sometimes not equal to 2.

However, treating the relation of drainage path lengths as a scaling coefficient is physically dependant on seepage conditions in the consolidation process.

According to the test results for various types of soils, except in seepage conditions, soil creep also influences the consolidation process. In order to properly scale the consolidation time, effects of fluid flow should be separated from creep. This problem was presented in many papers, where authors distinguished two stages of consolidation: primary — governed by the fluid flow, and secondary — creep (Schiffman *et al.*, 1964). Disagreement between the c_v values estimated using various methods (Duncan, 1993) shows that the separation of these elements of consolidation is still not satisfactory. In particular the assumption that seepage is the only feature of the initial stage of the consolidation process may be questioned. It seems that the creep factor is significant for many pre-consolidated and even fresh or unsaturated soils.

In order to properly estimate the coefficient of consolidation connected with fluid flow it is most important to precisely control the pore water pressure distribution. Pore water pres-
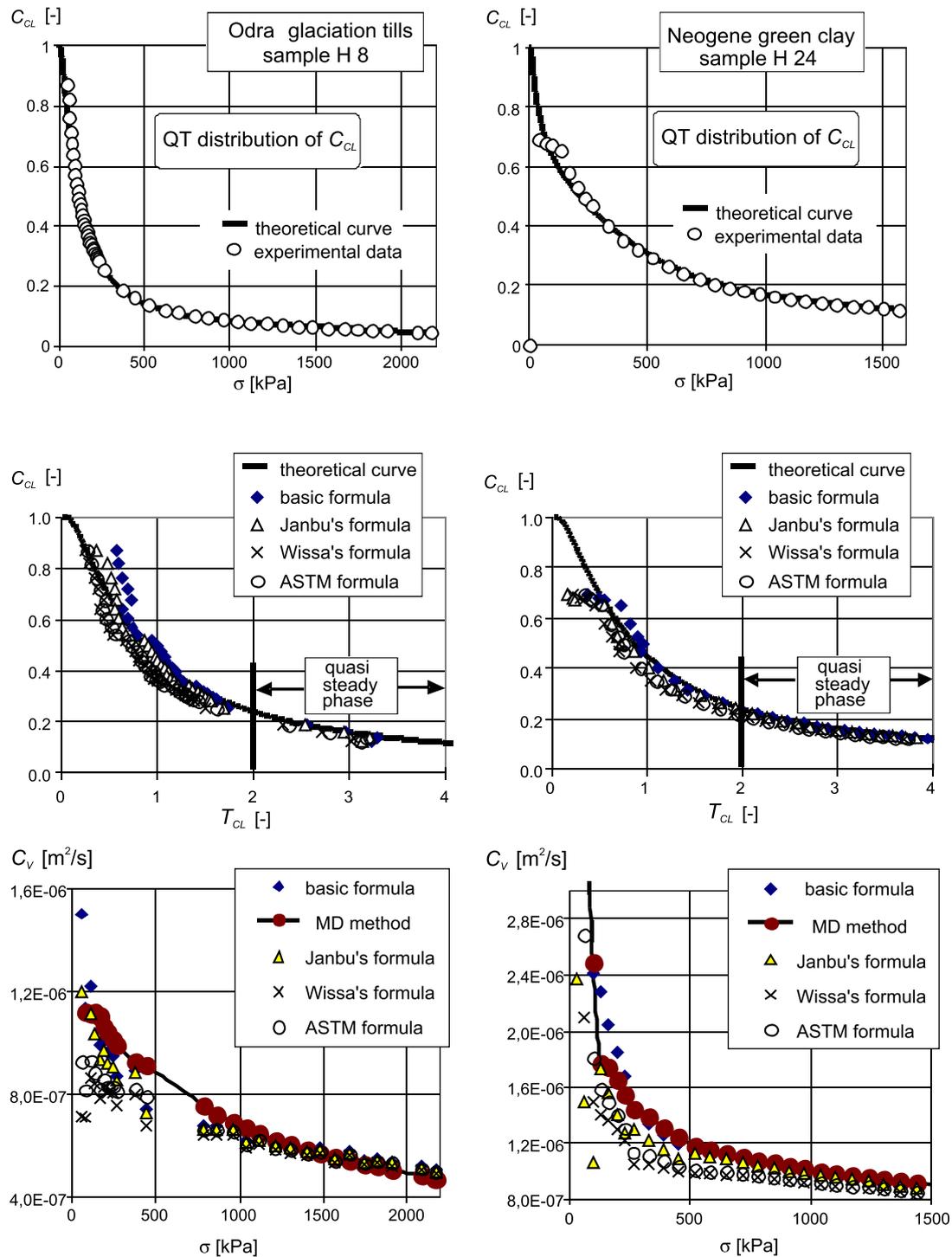
Fig. 10. Comparison of the c_v coefficient values estimated according to the MD and other methods

sure changes are greater and shorter in duration in CL tests than in IL tests. Consequently, measurement errors are minor and application of Terzaghi's solution for CL tests enables selection of the fluid flow aspect in the process analysed.

The C_{CL} vs. T_{CL} diagrams have been prepared on the basis of Terzaghi's theory. Therefore the c_v value estimated using MD tests mainly reflects the seepage aspect of the consolida-

tion process. This is why c_v values obtained using the MD method are higher than those estimated using previously applied methods which, to a various extent, allowed for the creep factor. Selecting the fluid flow factor is crucial for proper use of the time scale effect for long-term forecast.

CONCLUSIONS

1. The consolidation process in the continuous loading condition possesses attributes that make it different from classic consolidation under constant loading.

2. Application of the introduced dimensionless parameters C_{CL} vs. T_{CL} makes it possible to present characteristic diagrams of pore water pressure changes depending only on the loading procedure.

3. The algorithms presented allow for comparison of the interpretation methods applied up to now for CL tests with theoretical solutions based on the classical Terzaghi model of consolidation.

4. On the basis of the C_{CL} vs. T_{CL} relation the value of the consolidation coefficient may be estimated depending on the fluid flow and permeability factor of the consolidation process (MD method). As a rule, c_v values are slightly higher than the values obtained from the interpretation methods for CL tests used up to the present time.

5. Comparison of the theoretical diagrams and the experimental results enables distinguishing of the types of pore water pressure distribution and their consistency with the assumptions of the filtration consolidation theory.

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