Influence of Fines Content on Consolidation and Compressibility Characteristics of Granular Materials

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Abstract. Various behaviour of soil under loading results to large extent from kind of soil considered. There is a lot of literature concerning pure sand or plastic clays, while little is known about materials, which are from classification point of view, between those soils. These materials can be considered as cohesionless soils with various fines content. The paper present results of tests carried out in large consolidometer on three kinds of soil, containing 10, 36 and 97% of fines content. Consolidation, permeability and compressibility characteristics were determined. Analysis of the test results allowed to formulate conclusion concerning change in soil behaviour resulting from fines content.

1. Introduction
Majority of scientific papers dealing with granular materials concerns clean homogeneous sands. There are some academic sands, which are tested around the world in well-recognized laboratories. Special programs are set for testing these soils in calibration chamber, centrifuge, true triaxial test or torsional shear hollow cylinder etc. Such tests can deliver deep methodological insight in to the nature of soil behaviour under complex loading conditions. However, conclusion from these tests cannot be applied to sands, which contains considerable amount of fines. In natural and man-made subsoils, rarely exists strata which fulfil the condition of homogeneity. It creates many problems in interpretation of geotechnical tests. It concerns in situ and laboratory tests as well carried out on cohesionless materials. The major problems with test interpretation arises from change of compressibility in layered soils. For instance in field static or dynamic probe tests, change of penetration resistance is usually interpreted as result of change in state of soil. However, this is not always true since major penetration resistance change is an effect of soil compressibility change resulting from different fines content. Fines content differentiates the mechanical behaviour of soils, which from classification point of view are between cohesive and cohesionless materials. These soils are perceived as the transitional materials and little is known about their mechanical characteristics and how they change with fines content. The earliest work concerning a role of fines content in mechanism of bearing stress in grains and particles mixture was published by Mitchel in 1976 [1]. Subsequent research concerned influence of fines content on susceptibility to liquefaction [2], or initial stiffness [3].

One of the most important kind of characteristics, which deliver essential information of soil behaviour are compressibility characteristics. In comparison of cohesive and cohesionless soils, compressibility curves for the first loading differ entirely. In case of clean sands, there are large
number of curves which starts at various initial void ratios, while for a given cohesive soil this characteristic is unique. This difference has fundamental consequences for the way the soil are tested (e.g. sample preparation method) and how they behave under loading conditions.

Characteristics which are associated with compressibility are consolidation and permeability parameters. It is important to know how these characteristics change, especially between soils with considerable fines content. The paper presents experimental data from laboratory tests carried out in large diameter consolidometer. Three granular soils of various fines content ranging from 10 to 97% were tested. Fines content was selected in such a way that sandy, purely fine and intermediate materials are presented. The test data were elaborated in the form of consolidation, permeability and compressibility characteristics. Parameters derived on the basis of the obtained results were presented against fines content what allowed to characterized transition zone between sand like and clay like behaviour with respect to fundamental characteristic obtained in one dimensional loading conditions.

2. Material and test procedure

The tests were carried out in consolidometer on three kinds of granular material containing various amount of fines i.e. 10, 36 and 97%. Fines content in this paper is understood as a percent of soil mass passing sieve 0.075 mm (No. 200 according to ASTM). Grain size distributions of tested materials are shown in figure 1. For each kind of material a few tests were done, characterized by various initial void ratio. Altogether 14 tests were carried out. Initial void ratio of tested material is shown in table 1.

![Figure 1. Grain size distribution of tested soils](image)

**Table 1.** Void ratios of tested soils.

| Soil        | Initial void ratio e₀ |
|-------------|------------------------|
| 10% of fines| 1.094 0.943 0.853 0.814 0.779 0.711 |
| 36% of fines| 1.014 0.811 0.726 0.615 - - |
| 97% of fines| 2.168 1.141 0.981 0.909 - - |
The tests were carried out in large scale consolidometer on specimens which were 150mm in diameter and 60 mm in height. Specimen were reconstituted by dry deposition in 4 layers. Due to large area of a specimen a big force was necessary to imposed assumed destiny vertical stress of 1740 kPa, therefore multi lever load system capable of application dead load of 60 kN was used. A view of a system and consolidometer chamber is shown on photos in figure 2.

![Figure 2. Consolidometer and loading system used for tests](image)

Since representative consolidation, permeability and compressibility characteristics can be obtained only on two phase \((S_r=1)\) material, it was necessary to ensure full saturation. Construction of consolidometer cell made possible to carry out the whole procedure of saturation. Therefore after reconstruction of a specimen and assembling a cell, soil specimen were flushed with carbon dioxide, and then flushed with de aerated water. After that a specimen underwent back pressure saturation procedure. Usually 300 kPa of back pressure was sufficient to obtain Skempton's parameter B higher than 0.96. When a specimen was fully saturated loading in steps began. A soil was loaded in stages by load controlled method with load increment ratio equal 1 (existing load was doubled). Loading started at 25 kPa and ended at 1740 kPa. In fine grained soil loading unloading cycle was done. During loading vertical deformation and volume of water expelled from a specimen was measured with time.

3. Consolidation and permeability characteristics
Consolidation characteristics for each kind of tested materials are depicted in figure 3. In order to show influence of fines content and initial void ratio as well, consolidation curves for dense and very loose material were compared. Consolidation curves for each loading step are shown. As can be deduced from the chart both variables (fines content and initial void ratio) considerable change consolidation curves. As far as fines content is concerned volumetric strain increases with fines content for dense and loose material. Fines content increases time to the end of consolidation and volumetric strain. It might be considered surprised that these differences are bigger between soil containing 10 and 36% of fines than between specimens containing 36 and 97% of fine material. It is also to worth to notice that for a few initial loading stages (up to 200 kPa), increase in volumetric strain is the biggest, especially for soil containing 36 % of fines. As to influence of initial void ratio, again the difference is the biggest for intermediate soil i.e. containing 36% of fines. It is also worth to emphasize that one can look at the data, in absolute numbers and as relative increase of volumetric strain due to increase of initial void ratio. With respect to this criterion \(\Delta V/V\) also increased 4 times. This phenomenon can be explained by already mentioned separation of compressibility curves in sands.
Consolidation characteristics are immanently associated with permeability parameters. Since direct measurement of coefficient of permeability $k$ for each material requires adjustment of a proper method [4], it is advisable to derive this parameter from consolidation data. This approach requires determination of coefficient of consolidation $c_v$ from consolidation curve. Usually Casagrande or Taylor’s procedure is used. Both procedures are well known since Taylor described them in 1948. In this research the later [5] method was used. For each loading step the time $t_{90}$ was read from the consolidation curve and coefficient of consolidation was calculated on the basis of the following formula:

$$ C_v = T_v \frac{h^2}{t_{90}} $$  \hspace{1cm} (1)

where:
- $T_v$ - time factor
- $h$ - half of a specimen height (drainage path)

Then coefficient of consolidation was determined on the basis of the following formula:

$$ k = \frac{C_v \cdot \gamma_w}{M_0} $$  \hspace{1cm} (2)

Where $M_0$ is constrained modulus and $\gamma_w$ unit weight of water. The results of these calculation were shown in figure 4 in the form of charts were void ratio is shown against coefficient of permeability. The elaborated data shown for three kinds of soil explicitly show that initial void ratio has significant influence coefficient of permeability only for sandy material in the fines material as well for that containing 36% of fines relation between void ratio and coefficient of permeability is almost unique.

4. Compressibility characteristics

Compressibility characteristics are key feature from engineering point of view. Compressibility curves for all tests carried out are presented in figure 5. There are three separate charts at which compressibility curves for all initial void ratio were shown. If one disregards unloading reloading cycles and absolute values of void ratio, it can be stated that curves for soils containing 36% and 97% of fines, tend to be unique in the majority of applied stress range. The data for sandy material (10% of fines) are entirely different. In this case, stress increase hardly changes relation among void ratios. These observation suggests that soil containing 36% of fines reveals clay like behaviour from compressibility point of view.

In order to refine observations concerning compressibility of the tested material the data were elaborated in terms of compression index $C_c$. The index is defined as follows:

$$ C_c = - \frac{de}{d \log \sigma_{\nu}} $$  \hspace{1cm} (3)

For calculation of $C_c$ a void ratio corresponding to the end of primary consolidation was used. Change of calculated values of compression index against vertical effective stress is shown in figure 6. Similarly, as in the case of compressibility curves the data were grouped into three charts representing various soil kind. The observation can be summarized as follows:
- homogeneity of compression index distribution is observed in stress range 200-1740 kPa;
- higher values of compression indices are observed for soils containing more fines;
- for sandy soils $C_c$ values reveal increasing trend in stress range 200-1740 kPa and their values clearly depend on initial void ratio;
- for soil containing 36% of fines, values of compression index seem do not change with respect to stress range and initial void ratio as well;
- for fines material (97% of fines) values of compression indices apparently decrease with increase of stress level and their values are almost equal irrespectively initial void ratio.

Figure 3. Consolidation characteristics of three kind of materials prepared at low and high values of initial void ratio
Figure 4. Relation between void ratio and coefficient of permeability for tested materials.
Some of the above observation are even better seen if one considers average value of compression index $C_c$ for each kind of soil. Such chart is depicted in figure 7. Looking at the absolute values of compression indices it can be stated that they are proportional to fines content. This feature is stronger for smaller stress and diminish with stress increase. It can be generalized that stress increase decrease differences among various soils. This conclusion can supported if the compression indices are shown against fines content (figure 8). To enhance readability of the chart the data are shown only for selected stress levels (200, 830 and 1740 kPa). As it results from the chart, for relatively small stress, typical for engineering applications, values of compression indices are significantly different for each
soil kind. The change is from 0.04 for sandy material to 0.35. So it is almost 9 times difference. These differences diminish with stress increase and for stress level 1740 kPa the range of $C_c$ is only 0.08-013. It can be also deduced from the chart that $C_c$ significantly decreases with stress level for finest material and increase of compression index for sandy with stress range. Although the first observation seems to intuitively explained, the latter is rather unexpected. This can results from various mechanism of stress bearing in sandy and very fine material. This issue deserves further research. It is also worth to note that for soil having 236% of fines there is an inversion of compression indices what comply with postulate of so called threshold fines content at which susceptibility to liquefaction changes.

**Figure 6.** Compression indices against vertical effective stress for various materials
Figure 7. Average compression indices for materials of various fines content against vertical effective stress

Figure 8. Compression indices $C_c$ against fines content for selected value of loading

5. Conclusions
The test programme carried out allows formulating the following conclusions:

- Granular soil containing 10% of fines content behaves as pure sand with respect to compressibility, permeability and consolidation characteristics while soils consisting almost entirely of nonplastic fines reveals typical clay like behaviour. Soil containing 36% of fines exhibits more clay like behaviour, however, depending on initial void ratio it can have feature typical for sands and clays.
- Distributions of compression indices reveals different mechanism of load bearing in sandy and very fine material.
- Compression indices of soil having small and large amount of fines reveals strong conversion. Inversion of compression indices corresponding to 36% of fines confirms postulate of existence of threshold fines content, which divides soils with respect to susceptibility to liquefaction.

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