Physical Modelling in Geotechnics

Static liquefaction of fine sands in laboratory conditions

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ABSTRACT: The static liquefaction phenomenon is discussed in the article. The results of laboratory tests of fine sands static liquefaction along with the tests methodology are presented. The tests were conducted on a unique stand on three fine-grained sands characterised by three different density indexes. The impact of soil density on hydraulic condition of static liquefaction occurrence was performed. The critical hydraulic gradient at which liquefaction occurs is discussed. The authors compared the values of critical hydraulic gradient obtained from laboratory tests to the theoretical values that are available from the literature.

Keywords: find sand, static liquefaction, density index, quicksand, critical hydraulic gradient

1 INTRODUCTION

Static liquefaction can be classified as a macroscale deformation due to seepage. The macroscale deformation results from seepage force acting per unit of soil volume, and it strives for movement the whole volume of soil in the water flow direction. Marcoscale deformation usually occurs in relatively large volumes of soils, causing changes in the soil stress state in the volume and destroying its structure (Chugaev, 1985; Dąbska, 2021).

The static liquefaction is observed at upwards vertical flow through the soil, when the submerged unit weight of soil γ' is buoyed up by a seepage force per unit of soil volume j_{Vcr} (critical seepage force):

$$\gamma' = j_{Vcr} \tag{1}$$

The conditions of static liquefaction are defined as critical hydraulic load conditions: the permissible value of water pore pressure or the value of critical hydraulic gradient i_{cr} (critical flow velocity). The hydraulic gradient that causes the critical seepage force and static liquefaction is called a critical hydraulic gradient:

$$j_{Vcr} = \gamma_w \ i_{cr} \tag{2}$$

where γ_w is unit weight of water. When liquefaction occurs, the effective stress of soil is reduced to zero $\sigma' = 0$, soil particles do not lean against each other but float in the water. The soil loses the properties of a solid and turns into a liquid state. The soil shear resistance is 0. Liquefaction is more likely to occur in fine sands, silty sands, and silts (Kollis, 1966; Wiłun, 1958). It is often observed in loose to medium dense or not compacted soils. Hence, the soil susceptibility to static liquefaction and the values of critical gradients at which the soil is liquefied at vertical upward flow are affected by the soil graining and porosity. Istomina (1957) states that static liquefaction as deformation due to seepage dominates in poorly graded soils characterized by the coefficient of C_U < 10 than well-graded soils. Based on the research Istomina (1957) determined the permissible value of the hydraulic gradient for soils C_U < 10, i_{per} = 0,4.

The most common equation that defines the critical hydraulic gradient and includes porosity dependency is (Terzaghi, 1933):

$$i_{cr} = (\rho_s - 1)(1 - n) \approx 1 \tag{3}$$

The problem of soil resistance to failure due to seepage and loss of bearing capacity as a result of deformation due to seepage has been included in the geotechnical standard EN 1997-1:2004 Eurocode 7: Geotechnical design – Part1: General rules as one of the five ultimate limit states, the Hydraulic Heave ultimate limit state. That ultimate limit state applies to soil failure by hydraulic heave (soil particles liquefaction), internal erosion and piping. Condition of a limit state of failure due to heave by seepage of water (static liquefaction) occurrence is formulated based on equation (1) to allow for partial factors.

Soil liquefaction is also called soil boiling or quicksands.

The paper presents the results of laboratory tests of critical hydraulic gradients, at which the liquefaction phenomenon occurs, performed on fine sands of different densities. The values of critical hydraulic gradients from experimental tests are compared with the theoretical values determined according to the formula (3).

2 MATERIALS AND METHODS

The three uniformly graded fine sands were used for the static liquefaction tests. Table 1 summarises the grain size distribution of the sands. Table 2 presents the minimum and maximum porosities and void ratios. The particle size distributions, porosities and void ratios were carried out according to PN-B-04481:1988.

Table 1. Fine sands grain size distributions.

Soil	d_{50} (mm)	d_{10} (mm)	d_{60} (mm)	C_U (-)
Sand 1	0.17	0.13	0.18	1.36
Sand 2	0.18	0.10	0.19	1.89
Sand 3	0.19	0.12	0.19	1.56

Table 2. Void ratios and porosities, minimum and maximum of fine sands.

Soil	e_{min} (-)	e_{max} (-)	n_{min} (-)	n_{max} (-)
Sand 1	0.50	0.88	0.33	0.47
Sand 2	0.47	0.79	0.32	0.44
Sand 3	0.52	0.96	0.34	0.49

The tests were carried out on cylindrical specimens 0.051 m height, with a diameter of 0.05 m, prepared in rigid-wall cylinders with porous filters placed bottom. The specimens were formed directly in the cylinders by compacting sands up to the demanded density indexes. The critical hydraulic gradients were determined at density indexes I_D= 0.45, 0.60 and 0.75 for three sands. At each density for each sand, the tests were conducted twice. Eighteen tests were done in total. As a permeating liquid, tap water was used. The hydraulic gradient was increased by 0.4 every 60 seconds until sample failure due to seepage occurred. The observation of soil failure was made during the tests.

The testing stand is presented in Fig. 1.

Fig. 1. The testing stand used for the investigation of the static liquefaction phenomenon.

3. RESULTS AND DICUSSION

3.1 Liquefaction tests results

The results of fine sands failure due to seepage tests are summarised in Table 3. The critical hydraulic gradients range between 0.98 and 1.27. For the examined sands, deformation due to seepage in the form of static liquefaction was observed except for three tests. At the density index of 0.75 for sand 2 (two tests) and sand 3 (one test), failure occurred in the form of liquefaction, but locally on the sample surface, the soil was partially uplifted from the upper layer with a thickness of up to about 0.01 m. A transient state of deformation between liquefaction (L) and uplift (U) was noticed in those points. As the deformation due to seepage occurred, the soil structure was destroyed, and the volume of soil samples increased.

Table 3. Fine sands test results.

Soil	I_D (-)	n (-)	i_{cr} (-)	Phenomenon observed
Sand 1	0.45	0.41	0.98	L
	0.45	0.41	0.98	L
	0.60	0.39	1.08	L
	0.60	0.39	1.18	L
	0.75	0.37	1.27	L
	0.75	0.37	1.27	L
Sand 2	0.45	0.39	0.98	L
	0.45	0.39	1.08	L
	0.60	0.37	1.27	L
	0.60	0.37	1.18	L
	0.75	0.35	1.27	L/U
	0.75	0.35	1.27	L/U
Sand 3	0.45	0.43	0.98	L
	0.45	0.43	0.98	L
	0.60	0.41	0.98	L
	0.60	0.41	0.98	L
	0.75	0.39	1.08	L
	0.75	0.39	1.18	L/U

3.2 Density effect on the critical hydraulic gradient

The critical hydraulic gradient versus the density index for three fine sands are presented in Fig. 2. For all sands, the critical hydraulic gradient shows an upward trend with an increase in sand density. The highest values of critical hydraulic gradient are obtained for sand 2 (d_{10}= 0.10 mm and d_{50}= 0.18 mm), slightly lower for sand 1 (d_{10}= 0.13 mm and d_{50}= 0.17 mm) and the lowest for sand 3 (d_{10}= 0.12 mm and d_{50}= 0.19 mm). Such a dependency suggests that the critical hydraulic gradient is affected by soil density.

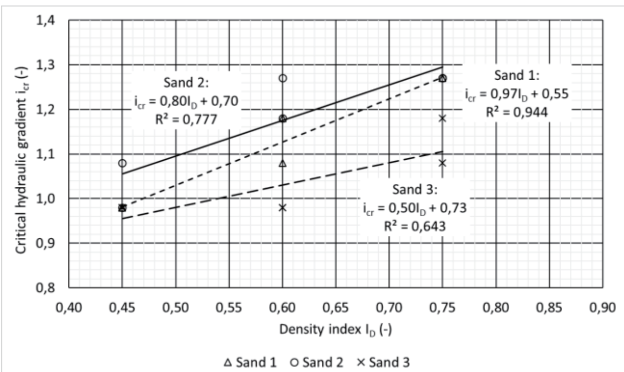

Fig. 2. The critical hydraulic gradient versus the density index.

3.3 Theoretical and experimental hydraulic gradients values

The dependence of the critical hydraulic gradient determined in the laboratory tests for three fine sands with d_{50} = 0.17-0.19 mm and the critical hydraulic gradient calculated according to the Terzaghi formula (3), on the density index is presented in Fig. 3.

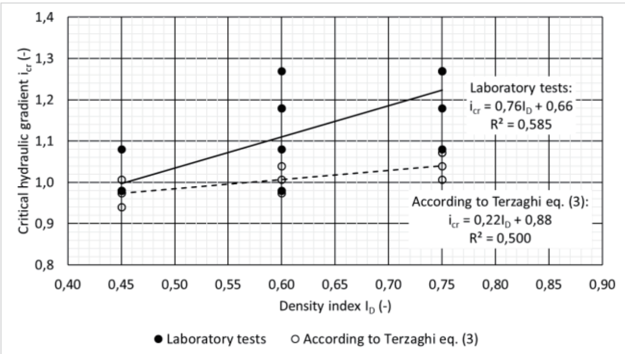

Fig. 3. The critical hydraulic gradients versus the density index.

It is clearly seen that the critical hydraulic gradient values from laboratory tests are higher than theoretical values. The values are similar at a density index of 0.45, while the difference between theoretical and experimental values rises with an increase in compaction of up to 22% at a density index of 0.75. It can be assumed that equation (3) allows estimating values safer for engineering practice.

4. CONCLUSIONS

Based on the study results, the following conclusions were drawn for uniformly graded fine sands with d_{50}= 0.17-0.19 mm and density indexes range 0.45 and 0.75:

1. The liquefaction is a dominant deformation due to seepage at vertical upward flow. The destruction of the soil structure accompanies it. It is manifested by an increase in the soil volume after failure occurrence.
2. The critical hydraulic gradients range between 0.98 and 1.27.
3. The soil density affected the soil's susceptibility to static liquefaction. Less dense soils are more susceptible to liquefaction. More dense soils require a greater gradient for failure due to seepage occurring.
4. At higher densities, the probability of failure due to seepage occurring in the transient state between liquefaction and uplift increases.
5. The theoretical, critical hydraulic gradients are lower than those obtained from the laboratory tests. Their differences are more significant with an increase in density index, even up to about 22% at greater density.

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