Lecture Notes in Civil Engineering

C. N. V. Satyanarayana Reddy
K. Muthukkumaran
Neelima Satyam
Ravikiran Vaidya *Editors*

Ground Characterization and Foundations

Proceedings of Indian Geotechnical Conference 2020 Volume 1

Springer

Lecture Notes in Civil Engineering

Volume 167

Lecture Notes in Civil Engineering (LNCE) publishes the latest developments in Civil Engineering—quickly, informally and in top quality. Though original research reported in proceedings and post-proceedings represents the core of LNCE, edited volumes of exceptionally high quality and interest may also be considered for publication. Volumes published in LNCE embrace all aspects and subfields of, as well as new challenges in, Civil Engineering. Topics in the series include:

- Construction and Structural Mechanics
- Building Materials
- Concrete, Steel and Timber Structures
- Geotechnical Engineering
- Earthquake Engineering
- Coastal Engineering
- Ocean and Offshore Engineering; Ships and Floating Structures
- Hydraulics, Hydrology and Water Resources Engineering
- Environmental Engineering and Sustainability
- Structural Health and Monitoring
- Surveying and Geographical Information Systems
- Indoor Environments
- Transportation and Traffic
- Risk Analysis
- Safety and Security

To submit a proposal or request further information, please contact the appropriate Springer Editor:

– Pierpaolo Riva at pierpaolo.riva@springer.com (Europe and Americas);
– Swati Meherishi at swati.meherishi@springer.com (Asia - except China, and Australia, New Zealand);
– Wayne Hu at wayne.hu@springer.com (China).

All books in the series now indexed by Scopus and EI Compendex database!

More information about this series at http://www.springer.com/series/15087

C. N. V. Satyanarayana Reddy ·
K. Muthukkumaran · Neelima Satyam ·
Ravikiran Vaidya
Editors

Ground Characterization and Foundations

Proceedings of Indian Geotechnical Conference 2020 Volume 1

Springer

Editors
C. N. V. Satyanarayana Reddy
Geotechnical Engineering Division
Department of Civil Engineering
Andhra University College of Engineering
Visakhapatnam, Andhra Pradesh, India

K. Muthukkumaran
Geotechnical Engineering Division
Department of Civil Engineering
National Institute of Technology
Tiruchirappalli
Tiruchirappalli, Tamil Nadu, India

Neelima Satyam
Discipline of Civil Engineering
Indian Institute of Technology Indore
Indore, Madhya Pradesh, India

Ravikiran Vaidya
Geo Dynamics, Vadodara, India

ISSN 2366-2557 ISSN 2366-2565 (electronic)
Lecture Notes in Civil Engineering
ISBN 978-981-16-3382-9 ISBN 978-981-16-3383-6 (eBook)
https://doi.org/10.1007/978-981-16-3383-6

This Springer imprint is published by the registered company Springer Nature Singapore Pte Ltd.
The registered company address is: 152 Beach Road, #21-01/04 Gateway East, Singapore 189721, Singapore

Contents

About the Editors

Dr. C. N. V. Satyanarayana Reddy is a Professor of Civil Engineering at College of Engineering, Andhra University, Visakhapatnam, India, and has 28 years of teaching, research, and consultancy experience. His expertise and research interests are in the areas of reinforced soils, ground improvement, landfills, soil retention in excavations, deep foundations, and forensic geotechnical engineering. He obtained B.Tech. in Civil Engineering from Nagarjuna University, M.Tech. in Geotechnical Engineering from Indian Institute of Technology (IIT) Madras, M.E. (Structures) from Andhra University, and Ph.D. from National Institute of Technology (NIT) Warangal, India. He has guided 8 Ph.D. scholars and 92 M.Tech. dissertations. He has more than 110 publications in various national and international journals and seminars/conference proceedings. He has organized more than 30 seminars, workshops, and training programs. He is fellow of Indian Geotechnical Society, The Institution of Engineers (India) and Life member of Indian Roads Congress, Indian Concrete Institute, Indian Society for Technical Education, Indian Society for Rock Mechanics and tunneling Technology.

He handled several research projects funded by DST, UGC and AICTE. He received the Engineer of the Year 2006 Award from Govt. of Andhra Pradesh and The Institution of Engineers (India) A.P. State Centre, Best Academician Award for the year 2014 from Andhra University and Andhra Pradesh Scientist Award 2020 in Civil Engineering Discipline from A.P. State Council of Science and Technology, Govt. of A.P. He served as National Executive Committee member of Indian Geotechnical Society during the period 2013–2020. He served as a member in H-4 Committee on Embankment, Ground Improvement and Drainage Committee of Indian Roads Congress (IRC) for the term 2018–2020. He is serving as member of TC 213 on "Scour and Erosion" of International Society for Soil mechanics and Geotechnical Engineering (ISSMGE).

Dr. K. Muthukkumaran is currently Professor of Civil Engineering at National Institute of Technology (NIT), Tiruchirappalli, India. He obtained his Ph.D. in Marine Geotechnical Engineering from Indian Institute Technology (IIT) Madras. He has published more than 120 papers in international and national journals and conferences. He has completed 5 R&D (including ISRO Chandrayaan-2 Mission project) &

50 major consultancy projects in geotechnical engineering and published two patents. Dr. Muthukkumaran's area of research is in geotechnical engineering, which includes pile foundation, soil–structure interaction, marine geotechnics & foundations, field instrumentation, geotechnical physical modeling, ground improvement, and forensic geotechnical engineering. He has received DST Young Scientist Award, IGS-Smt. Indra Joshi Biennial Award and Keynote Paper Award—GEOMATE Conference 2015 at Osaka, Japan.

He has guided 8 PhD, 5 MS (by research) and more than 40 M.Tech. students in Geotechnical and allied research areas. He is a member of Technical Committee (TC-301 on "Preservation of Historic Sites") of International Society for Soil Mechanics and Geotechnical Engineering. He served as a member of IGS National Executive Committeeduring the period 2017–18. He has received DST Young Scientist Award, IGS-Smt. Indra Joshi Biennial Award and Keynote Paper Award—GEOMATE Conference 2015 at Osaka, Japan. He has received NIT Trichy Achiever Awards for research publications, research projects, maximum citation and consultancy projects. He is a recipient of Bharat Vikas Award–2018.

Dr. Neelima Satyam is currently Associate Professor and Head in Discipline of Civil Engineering at Indian Institute Technology (IIT) Indore. She obtained her Ph.D. and M.Tech. from IIT Delhi and B.Tech. from SV University, Tirupati. She is actively engaged in teaching, research, and consultancy in the field of geotechnical engineering, particularly in geotechnical earthquake engineering, microzonation and site response studies, landslide hazard and monitoring, liquefaction modeling, micro-seismic data processing, and rock engineering. She received research grants from DST, MHRD, AICTE, ITRA, DAE, NIOT, NRDMS, ISRO, and MoES. She has 97 papers published in journals and conferences of national and international repute. She was Recipient of IEI Young Engineers Award 2011, AICTE Career Award 2012, JSPS fellowship in 2013, and INWES Young Woman Engineer Award in 2012. She is elected National Executive Member of Indian Geotechnical Society (2018–2020) and Indian Society of Engineering Geology (2020–2021).

Mr. Ravikiran Vaidya is Principal Engineer and Founder of Geo Dynamics, a company involved in deep foundation testing. He has been instrumental in popularizing and pioneering various methods for deep foundation testing in India. It is to his credit that methods like high strain dynamic testing, integrity testing, sonic logging, etc., are now routinely used all over India to verify pile foundations and also form a part of all specifications of all major projects. He has recently worked on thermal integrity profiling for bored piles and indigenized the bidirectional load testing technology. He has written several technical papers on the subject, and his work has been cited by several authors in various articles and dissertations. He is also Guest Lecturer at academic and industry forums.

Effect of Plasticity of Fines on Properties of Uniformly Graded Fine Sand

M. Akhila, K. Rangaswamy, N. Sankar, and M. R. Sruthy

1 Introduction

Even though researchers separate soils based on particle size as sand, silt and clay, in the field, soil always exists as a combination of all these. There are many studies concentrating on the effect of fines on the shear characteristics of sand [1–3] and liquefaction [4–7] but only a few studies have considered the other properties.

Yang and Wei [8] have analysed the change in critical state friction angle for Fujian and Toyoura sands. For clean sand without fines, the critical state friction angle tends to decrease with increasing roundness of sand particles. When those sands were tested with fines (round shape), the critical state friction angle of the mixture tends to decrease with an increase in fines content. But for fines with an angular shape, the critical state friction angle tends to increase with fines content. Phan et al. [9] have conducted one-dimensional consolidation tests on sand–silt mixtures (with low-plastic fines at a constant void ratio and constant relative density) and indicated that the behaviour of the mixtures were similar to those of loose sand. The effect of fines on void ratios was studied by Cubrinovski and Ishihara [10]. The authors reported that the void ratio initially decreases as the fines content increases from 0–20% and above 40% fines, the maximum and minimum void ratios were seen to increase steadily.

It is clear from the literature that the studies on the effect of plasticity of fines on the properties of sand are limited. Hence, the present study is focused on the effect of the amount of fines and the type of fines (or plasticity index of fines) on various properties of sand like specific gravity, limiting void ratios, grain size characteristics, angle of internal friction and compression index.

M. Akhila (✉) · M. R. Sruthy
SCMS School of Engineering and Technology, Ernakulam, Kerala, India

K. Rangaswamy · N. Sankar
NIT Calicut, Kozhikode, Kerala, India

C. N. V. Satyanarayana Reddy et al. (eds.), *Ground Characterization and Foundations*, Lecture Notes in Civil Engineering 167,
https://doi.org/10.1007/978-981-16-3383-6_1

2 Material Tested

The soil materials utilized in the present study are natural sand, M sand, natural clay and kaolinite clay. The natural sand was collected from Turavoor region in Kerala state. The M sand was collected from a local quarry in Calicut, and the non-plastic silt powder was derived after sieving the M sand through 75-µm sieve. The natural clay was collected at a depth of 3 m from Pantheerankavu which is 12 km far from south of Calicut city in Kerala. The index and strength properties of natural clay are listed in Table 1. The commercial kaolinite clay was procured from Sajeev and Co. Ltd. at Calicut district of Kerala state. The physical and chemical properties of kaolinite clay have been provided by the manufacturer as shown in Table 2. The results of tests conducted on natural sand are listed in Table 3.

Table 1 Index and strength properties of natural clay

Property	Value
Index property	
Specific gravity	2.56
Liquid limit (%)	79
Plastic limit (%)	48
Shrinkage limit (%)	27
Plasticity index	31
Clay size (%)	50
Soil classification	MH
Strength property	
Maximum dry density (kN/m^3)	17.5
Optimum moisture content (%)	32
UCS (kPa)	64

Table 2 Properties of kaolinite clay

Physical (Mass %)		Chemical (Mass %)	
Acid soluble	0.94	SiO_2	44
Water soluble	0.35	$Al2O_3$	38
Oil absorption (mm/100 g)	35	$Fe2O_3$	0.25
Specific gravity	2.62	TiO_2	0.35
pH	5 ± 0.5	CaO	0.05
Moisture percentage	1.5 ±0.06 0.5	Na_2O	0.06
TDS	100	K_2O	0.05
		MgO	0.07

Table 3 Basic and index properties of sand

Property	Value
Specific gravity	2.62
D_{50}, mm	0.28
Uniformity coefficient, C_u	2.36
Coefficient of curvature, C_c	0.87
e_{max}	0.858
e_{min}	0.578

Among the numerous trial combinations of low-plastic soils mixtures processed, the combinations of 50% kaolinite + 50% silt, 100% kaolinite and 20% clay + 80% kaolinite mixtures were found to possess plasticity indices of 5%, 10% and 15%, respectively and hence decided to be used for the current work. A total of 17 soil combinations were prepared by mixing the above-mentioned low-plastic soil combinations to the fine sand. The effect of presence of fines in the sand matrix on its properties was investigated by conducting various test including grain size analysis, relative density tests, specific gravity, direct shear test and one-dimensional consolidation tests. The tests were performed as per IS test procedures at different percentage fines (0, 10, 20, 30 40%) and plasticity index of fines (0, 5, 10, 15%).

3 Results and Discussions

Test results for index properties and some engineering properties on all the soil combinations adopted in the study are presented in the following sections.

3.1 Effect of Particle Size Characteristics

A combined dry sieve and hydrometer analysis was performed on all the soil combinations to obtain the particle size distribution. The gradation curve of fine sand is shown in Fig. 1. The values of average particle size, D_{50}, were found from the gradation curves, and its variation with respect to amount and PI of fines added is reported in Fig. 2. D_{50} gives an understanding of physical properties of the soil which in turn affect its strength and load bearing properties. It is clear from Fig. 2a that the D_{50} of sand decreases with the addition of fines at every tested value of PI. But, the plasticity index of fines has no much influence on the gradation of the soil (Fig. 2b).

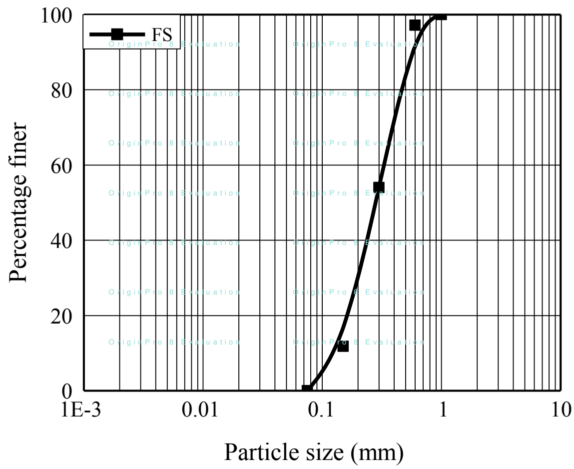

Fig. 1 Particle size distribution of sand

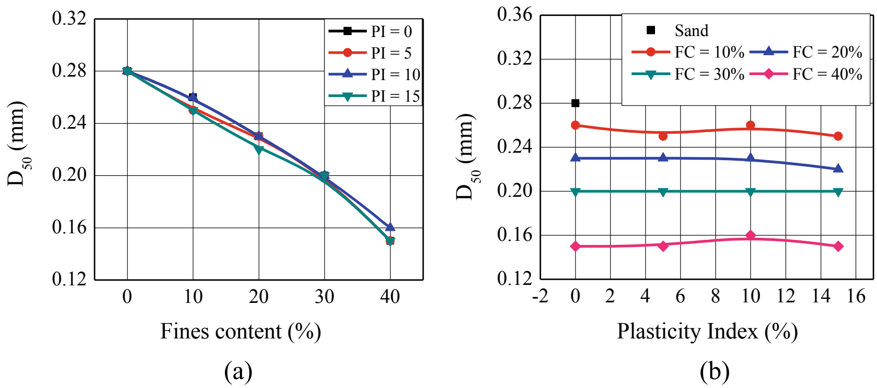

Fig. 2 Effect of **a** fines content and **b** plasticity index on D_{50}

3.2 Effect on Void Ratio

Relative density tests were performed as per the IS code procedure to arrive at the maximum density of all soil combinations. The minimum densities of soil combinations were attained by pouring it steadily with zero height using paper cone into the CBR mould of 150 mm size. The average values of maximum and minimum densities of soil combinations are reported in tables after repeating the tests thrice. Based on limited densities, the maximum and minimum void ratios are estimated by using the empirical equations, and its variations are shown in Figs. 3 and 4.

It was observed that both the maximum and minimum void ratios decrease as the fines content increases at all tested values of PI of fines. The variation with respect to the PI of fines showed different trends with different fines contents. At low fines

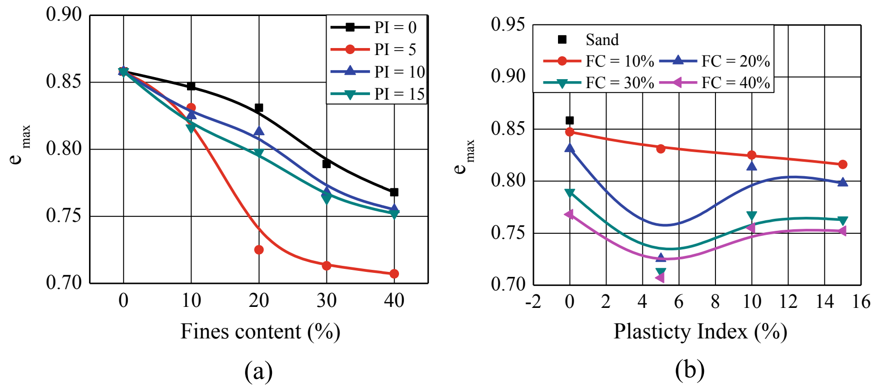

Fig. 3 Effect of **a** fines content and **b** plasticity index on e_{max}

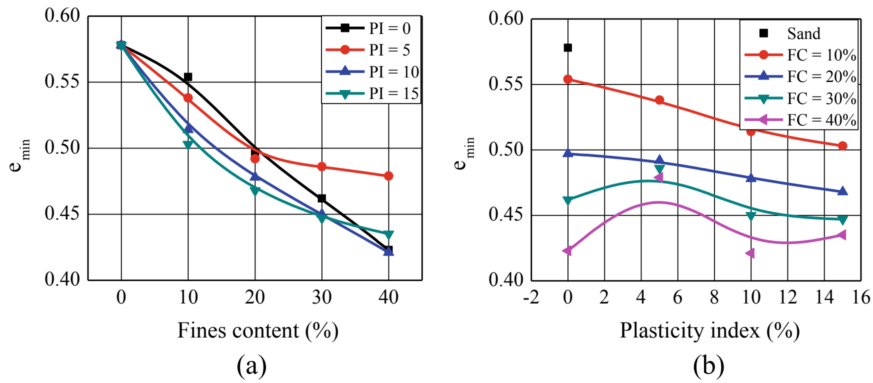

Fig. 4 Effect of **a** fines content and **b** plasticity index on e_{min}

content (10%), both e_{max} and e_{min} decrease with the increase in the plasticity of fines. At higher fines content, a contradiction was observed between the variations in e_{max} and e_{min}. At higher fines content, e_{max} shows an initial decrease with the increase in the PI of fines and then shows an increase. But the variation of e_{min} is exactly the opposite.

3.3 Effect on Specific Gravity

The specific gravity of all the soil combinations was found using pycnometer. Figure 5a and b shows the influence of fines on the initial specific gravity of natural fine sand which was found to be 2.62. It is clear from Fig. 5a that specific gravity increases with an increase in fines content. The influence of PI of fines on specific

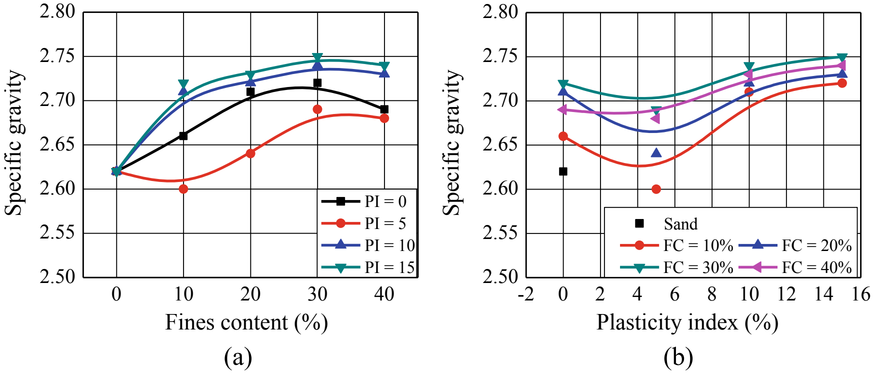

Fig. 5 Effect of **a** fines content and **b** plasticity index on specific gravity

gravity of sand is shown in Fig. 5b. The specific gravity of natural sand showed an initial decrease followed by a gradual increase with increase in PI of the fines added at all tested fines content.

3.4 Effect on Angle of Internal Friction

Direct shear tests were carried out in the small direct shear box (6 × 6 × 5 cm) to understand the changes in shear strength properties of natural sand due to addition of fines. The soil samples were filled in the direct shear box under loosest as well as densest states, and tests were conducted at normal stresses ranging from 100–300 kPa. From the test data, maximum and minimum angle of friction were found and are plotted in Figs. 6 and 7. It can be observed from Figs. 6a and 7a that the value

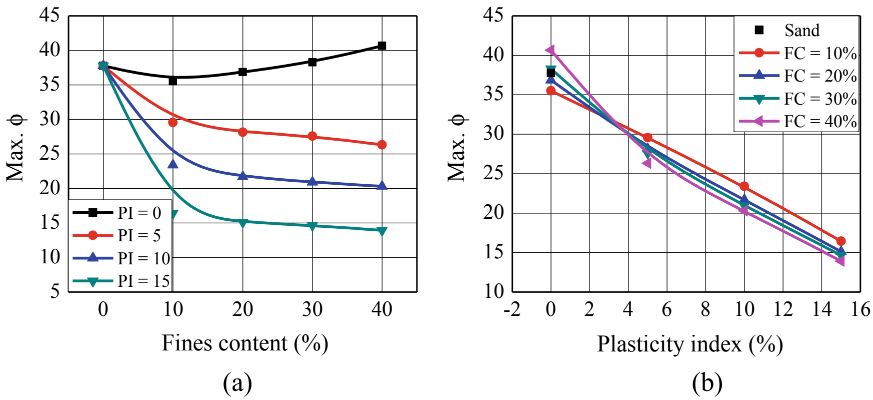

Fig. 6 Effect of **a** fines content and **b** plasticity index on maximum angle of internal friction

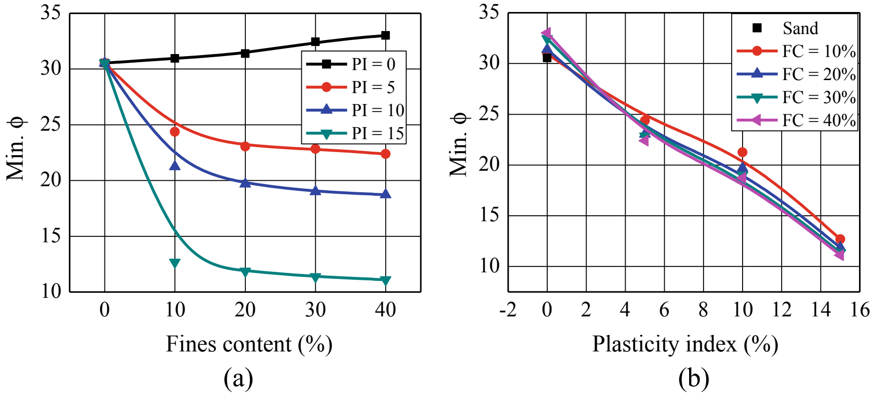

Fig. 7 Effect of **a** fines content and **b** plasticity index on minimum angle of internal friction

of the angle of internal friction increases with the addition of non-plastic fines (both in loosest and densest state of soil). But on addition of low-plastic fines to the sand, an opposite trend can be seen with a decrease in friction angle corresponding to an increase in fines content.

An average reduction of 62.5% from the initial values was observed in friction angles when low-plastic fines of PI = 15% was added to sand (Figs. 6b and 7b). The reduction trends can be seen overlapping at all fines content. The angle of internal friction decreases significantly with increase in the plasticity of fines.

3.5 Effect on Compression Index

One-dimensional oedometer tests have been conducted on all the soil samples in the loosest and densest possible states. The *e*-log p curve of natural sand is shown in Fig. 8. From this, the compression index is found as 0.04 in the loosest state and 0.03 in the densest state. The effect of fines on compression index of sand is shown in Figs. 9 and 10. It is clear from Figs. 9a and 10a that the addition of low-plastic fines increases the compression index of sand. But the variation in compression index due to the addition of non-plastic fines is negligible. Figures 9b and 10b indicate that at all tested fine content, C_c increases with an increase in PI of fines. Also, if the value of sand is excluded in Figs. 9 and 10, the remaining points will show a linear trend between C_c and PI.

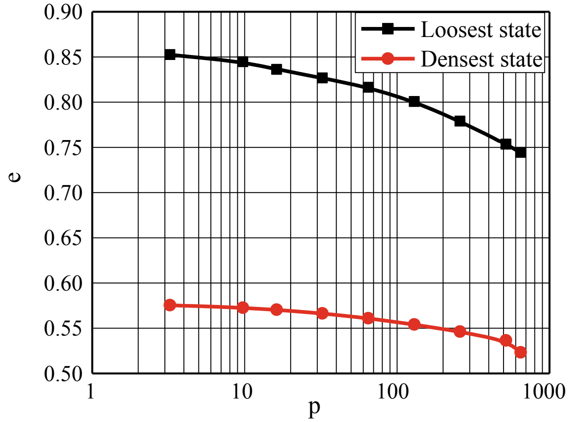

Fig. 8 *e*-log p plots of sand in loosest and densest state

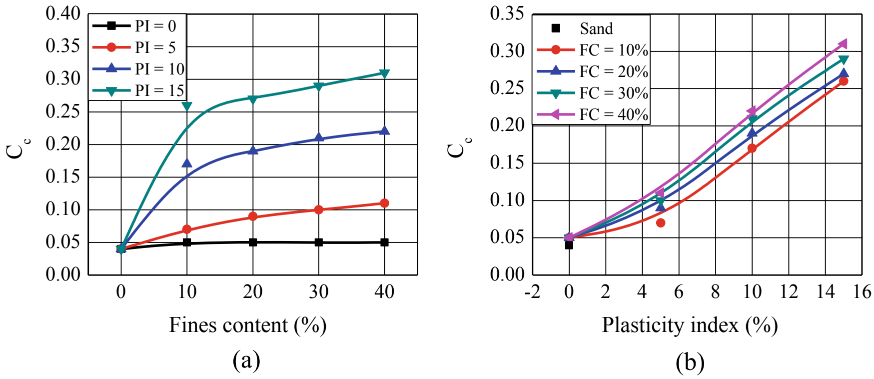

Fig. 9 Effect of **a** fines content and **b** plasticity index on C_c (loosest possible state)

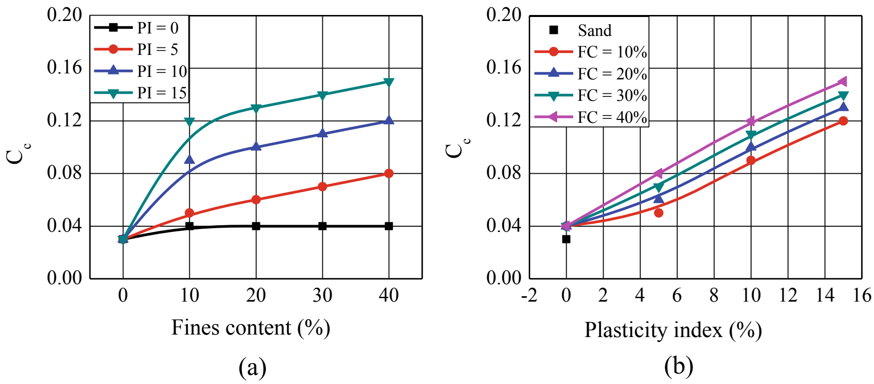

Fig. 10 Effect of **a** fines content and **b** plasticity index on C_c (densest possible state)

4 Summary and Conclusions

The present paper discussed the effect of non-plastic and low-plastic fines on the properties of sand. Effect on grain size characteristics, limiting void ratio, specific gravity, angle of internal friction and compression index is elaborated. The following conclusions are derived:

1. The D_{50} of sand decreases with the addition of fines at every tested value of PI. But, if particular fines content is taken, the plasticity index of fines has no much influence.
2. Both maximum and minimum void ratios decrease as the fines content increases at all testes values of PI of fines. The variation of $e_{\max}$ and $e_{\min}$ with respect to the PI of fines is contradicting to each other.
3. Value of angle of internal friction is increased with the addition of non-plastic fines (both in loosest and densest state of soil). But an opposite trend is found with the addition of low-plastic fines. The angle of internal friction decreases with increase in the plasticity of fines.
4. The addition of low-plastic fines increases the compression index of sand, but the effect of non-plastic fines is negligible. At all tested fine content, C_c increases with increase in PI of fines.

References

1. Kuerbis, R., Nagussey, D., Vaid, Y.P.: Effect of gradation and fines content on the undrained response of sand. Proc. Hyd. Fill. Struc. Geotech. Spec. Publ. **21**, 330–345 (1988)
2. Belkhatir, M., Missoum, H., Arab, A., Della, N., Schanz, T.: Undrained shear strength of sand-silt mixture: effect of intergranular void ratio and other parameters. KSCE J. Civ. Eng. **15**(8), 1335–1342 (2011)
3. Naeini, S.A., Baziar, M.H.: Effect of fines content on the steady-state strength of mixed and layered samples of sand. Soil Dyn. Earthq. Eng. **24**, 181–187 (2004)
4. Amini, F., Qi, G.Z.: Liquefaction testing of stratified silty sands. J. Geotech. Geoenviron. Eng. **126**(3), 208–217 (2000)
5. Dash, H.K., Sitharam, T.G.: Undrained cyclic pore pressure response of sand–silt mixtures: effect of non-plastic fines and other parameters. J. Geotech. Geol. Eng. **27**, 501–517 (2009)
6. Shen, C.K., Vrymoed, J.L., Uyeno, C.K.: The effect of fines on liquefaction of sands. In: Proceedings of 9th International Conference on Soil Mechanics and Foundation Engineering, pp. 381–385 (1977)
7. Hernández, Y.A., Towhata, I., Gunji, K., Yamada, S.: Laboratory tests on cyclic undrained behaviour of loose sand with cohesionless silt and its application to assessment of seismic performance of subsoil. Soil Dyn. Earthq. Eng. **79**, 365–378 (2015)
8. Yang, J., Wei, L.M.: Collapse of loose sand with the addition of fines: the role of particle shape. Geotechnique **62**(12), 1111–1125 (2012)
9. Phan, V.-A., Hsiao, D.-H., Nguyen, P.-L.: Effects of fines contents on engineering properties of sand-fines mixtures. Procedia Eng. **142**, 213–220 (2016)
10. Cubrinovski, M., Ishihara, K.: Maximum and minimum void ratio of sand characteristics. Soils Found. **42**(6), 65–78 (2002)

Comparison of Theoretical and Laboratory Permeability for Coarse-Grained Soil at Different Ground Conditions

Satyajit Roy, R. K. Bharti, V. K. Jain, Manish Gupta, and R. Chitra

1 Introduction

Permeability is a direct function of average grain size distribution of granular porous media [1]. The inter-relationship is quite effective for preliminary investigation, especially at prefeasibility stage. But proper investigation of soil is required during the designing stage, and it is important to know the actual response of soil towards permeability for structural integrity by laboratory methods. Several researchers made an effort to calculate the co-efficient of permeability and develop several indirect empirical formulae as laboratory testing sometimes takes considerable time in arriving at meaningful conclusion. Empirical correlations are function of grain sizes, porosity/void ratio, C_u, C_c and viscosity of pore fluids.

There are various empirical correlations available in the literature such as Hazen, Kozeny-Carman, Breyer, Slitcher, Terzaghi, USBR, Alyamani and Sen. Several investigators have studied these relationships and modified these formulae based on experimental work. The applicability of these formulae depends on the type of soil and compactness of the soil for which co-efficient of permeability is required to be estimated. As per Vukovic and Soro [2], the applications of different empirical formulae to the same porous medium material can yield different values of co-efficient of permeability. Again, soil is not homogeneous, and permeability varies from location to location. Actual ground conditions vary from place to place. Moreover, soil profile is not uniform but varies from one section to other. In some places, it may be dense, partially dense, and loose or submerged in water. Depending upon the condition of ground, permeability will also vary from place to place. Keeping in mind the various ground conditions, attempt has been made to determine the co-efficient of permeability on the soil samples remoulded at different compactness and moisture conditions. In the present study, attempt has been made to correlate permeability

S. Roy (✉) · R. K. Bharti · V. K. Jain · M. Gupta · R. Chitra
Central Soil and Materials Research Station, New Delhi 110016, India

C. N. V. Satyanarayana Reddy et al. (eds.), *Ground Characterization and Foundations*, Lecture Notes in Civil Engineering 167,
https://doi.org/10.1007/978-981-16-3383-6_2

values obtained through different correlations with laboratories values. The study has been carried out with four different types of soils viz. (i) Sind river sand (MP), (ii) Yamuna river sand, Delhi (iii) Rajasthan crushed sand and (iv) Ennore standard sand. The paper discusses the value of co-efficient of permeability determined in the laboratory, and the values are obtained theoretically at different conditions and present the factors affecting the values.

2 Established Empirical Formulae

Vukovic and Soro [2] summarized several empirical methods from former studies and presented a general formula:

$$\kappa = g/\nu.C.f(n).d_e^2 \tag{1}$$

where κ = co-efficient of permeability; g = acceleration due to gravity; ν = kinematic viscosity; C = sorting coefficient; f(n) = porosity function, and d_e = effective grain diameter. The kinematic viscosity (ν) is related to dynamic viscosity (μ), fluid (water) and density (ρ) as follows:

$$\nu = \mu/\rho \tag{2}$$

The values of *C, n* and d_e are dependent on the different methods used in the grain size analysis. According to Vukovic and Soro [2], porosity (*n*) may be derived from the empirical relationship with the co-efficient of grain uniformity C_u as follows:

$$n = 0.255\left(10.83^{Cu}\right) \tag{3}$$

where C_u is the co-efficient of grain uniformity and is given by

$$C_u = d_{60}/d_{10} \tag{4}$$

Here, d_{60} and d_{10} in the formula represent the grain diameter in (mm) for which 60% and 10% of the sample, respectively, are finer than d_{60} and d_{10}.

Former studies presented the following formulae which took the general form and presented in Eq. (1) but with varying *C, f(n)* and d_e values and their domains of applicability.

2.1 Hazen Formula [3]

It was widely used for the estimation of co-efficient of permeability of uniformly graded soils ranges from fine sand to gravel of diameter 0.1 to 3 mm, respectively, and uniformity co-efficient less than 5. This formula only depends on the effective size of grains as.

$$\kappa = (g/\nu) \times 6 \times 10^{-4}[1 + 10(n - 0.26)]d_{10^2}$$

2.2 Kozeny-Carman Equation [4]

The KC equation is not appropriate for soil with effective size above 3 mm or clayey soil [5]. The KC equation is widely used and accepted for co-efficient of permeability estimation because it depends on both the effective grain size and porosity (number of pores) of the porous media as given below.

$$\kappa = (g/\nu) \times 8.3 \times 10^{-3}\left[n^3/(1 - n)^2\right]d_{10^2}$$

2.3 Breyer

This method does not consider porosity, and therefore, porosity function takes on value 1. Breyer formula is often considered most useful for materials with heterogeneous distributions and poorly sorted grains with uniformity co-efficient between 1 and 20 and effective grain size between 0.06 and 0.6 mm [6].

$$\kappa = (g/\nu) \times 6 \times 10^{-4} \times \log[500/U]d_{10^2}$$

2.4 Slitcher [7]

This formula is most applicable for grain size between 0.01 and 5 mm.

$$\kappa = (g/\nu) \times 1.0 \times 10^{-2} \times n^{3.287} \times d_{10^2}$$

2.5 Terzaghi [8]

$$\kappa = (g/\nu) \times C_t \times \left[(n - 0.13)/\sqrt[3]{(1 - n)}\right]^2 \times d_{10^2}$$

where the C_t = sorting co-efficient and. In this study, an average value of C_t is used. Terzaghi formula is most applicable for coarse grain sand [9]

2.6 USBR

$$\kappa = (g/\nu) \times 4.8 \times 10^{-4} \times d_{20}^{0.3} \times d_{10^2}$$

US Bureau of Reclamation (USBR) formula calculates co-efficient of permeability from the d_{20} and does not depend on porosity; hence, porosity function is a unity. The formula is most suitable for medium grain sand with uniformity co-efficient less than 5 [9].

2.7 Alyamani and Sen [10]

$$\kappa = 1300 \times [I_o + 0.025(d_{50} - d_{10})]^2$$

where κ is the co-efficient of permeability (m/day), Io is the intercept (in mm) of the line formed by d_{50} and d_{10} with the grain size axis, d_{10} is the effective grain diameter (mm), and d_{50} is the median grain diameter (mm). The method considers both sediment grain sizes d_{10} and d_{50} as well as the sorting characteristics. This formula, therefore, is exceptionally different from those that take the general form of Eq. (1) above.

3 Materials and Methods

In order to compare results obtained from these correlations with the laboratory test results, 4 sandy soil samples were selected from different sources for comparing the permeability values, and the photographs of samples selected are presented in Fig. 1. The particle size distributions of all 4 samples are performed [11] and presented in Table 1. The grain size distributions indicate that sample 1 and sample 4 have predominance of medium sand. Sample 2 has predominance of fine sand, whereas sample 3 has fine to medium sand in equal proportions. The effective size 'd_{10}' value of these samples varies from 0.019 to 0.40. C_u for 3 samples which are less than

Fig. 1 Types of sand selected

5, whereas for sample no. 3, it is 28.95. The different grain sizes of the sands are presented in Table 2.

The maximum density, minimum density and required density at 95, 85 and 75% of relative density for all selected samples as well as relative density achieved due to wet packing are presented in Table 3. Samples were packed in permeability mould in dry loose packing, wet loose packing, packed at 95% of relative density, packed at 85% of relative density and packed at 75% of relative density.

The photographs of the samples are presented in Fig. 1.

The particle size distributions of all 4 samples are presented in Table 1

The different grain sizes of samples are presented in Table 2

The relative density of the soil samples was determined according to IS 2720 (Part 14) [13]

Here, for calculation of required density γ_d at which samples are packed, relative density (I_d) were considered as 95, 85 and 75%. Moreover, the maximum density,

Table 1 Particle size distribution

Sample	Mechanical analysis						Soil type (IS:1498)	Remarks
	0.002 mm and less	0.002–0.075 mm	0.002 to 0.075 mm	0.425 to 2.0 mm	2.0 to 4.75 mm	4.75 mm and above		
	Clay	Silt	Fine sand	Medium sand	Coarse sand	Gravel		
1	0.0	1.0	10.7	74.5	11.9	1.9	SP	Medium sand
2	0.0	7.4	75.8	7.9	7.7	1.2	SP-SM	Fine sand
3	0.9	17.9	35.5	37.2	8.5	0.0	SM	Fine sand
4	0.0	1.7	26.5	70.7	1.1	0.0	SP	Medium sand

Table 2 Different grain sizes of samples

Parameters	Sind river sand (MP)	Yamuna river sand	Rajasthan crushed sand	Ennore standard sand
d_{10}	0.40	0.09	0.019	0.19
d_{20}	0.55	0.13	0.085	0.32
d_{30}	0.69	0.17	0.16	0.47
d_{50}	1.00	0.22	0.35	0.72
d_{60}	1.30	0.26	0.55	0.90
C_u	3.25	2.89	28.95	4.74

Table 3 Calculated values of minimum, maximum and required density as per IS code [12]

Type of soil	Sind river sand (MP)	Yamuna river sand	Crushed sand Rajasthan	Ennore standard sand
Min density, γ_{min} (g/cc)	1.57	1.37	1.32	1.55
Max density, γ_{max} (g/cc)	1.99	1.76	1.66	1.79
Required density @ 95% of relative density, γ_d (g/cc)	1.96	1.74	1.64	1.78
Required density @ 85% of relative density, γ_d (g/cc)	1.91	1.69	1.60	1.75
Required density @ 85% of relative density,γ_d (g/cc)	1.87	1.64	1.56	1.72
Relative density achieved due to wet packing (%)	32.3	47.2	58.6	36.2

minimum density and required density at 95, 85 and 75% of relative density for all selected samples as well as relative density achieved due to wet packing are presented in Table 3.

4 Preparation of Test Sample

In the present study, the theoretical values of co-efficient of permeability's are to be compared with laboratory co-efficient of permeability's compacted at different ground conditions like (i) 95% relative density, (ii) 85% relative density, (iii) 75% relative density (iv) dry loose packing, i.e. at minimum density and (v) wet loose packing. These ground conditions can be achieved by compacting by rodding, dry pouring and placing under water as par Head [14]. The methods of compaction are given below.

4.1 Compacting by Rodding

For achieving compactness closer to 95, 85 and 75% of maximum relative densities, compacting by rodding is used.

4.2 Dry Pouring

When the sample is to be packed at minimum/low density, dry pouring of sample is used. Here, a funnel fitted with a length of flexible tubing, long enough to reach the bottom of the permeameter cell is used for pouring the sample; the pouring is to be continued until the surface of the sand is at the correct level. The surface is to be levelled carefully with the minimum disturbance. The jolting of the cell or agitating the sample is to be avoided for packing the sample in low density.

4.3 Placing Under Water

Here, the valve on the base of the permeameter cell to be connected to the de-aired water supply and then valve to be opened to allow water to enter the cell to about 15 mm above the porous disc. Now, a large funnel fitted with a bung attached to a string or wire is to be supported over the cell, so that tubing reaches to the surface of water in the cell. Sample is now poured into the funnel. Now, the funnel is to be raised so that the end of tubing is just at the water surface. The water surface is to be maintained at about 15 mm above the surface of the placed soil by admitting more water through the base valve. The process is to be continued until the required amount of soil has been deposited in the cell and water added.

The laboratory permeability was determined by constant head method as described in IS 2720 (Part 17) [13], and results are presented in Table 4.

The co-efficient of permeability calculated from grain size analysis using empirical formulae is presented in Table 5.