Liquefaction Behavior of Low to Medium Plasticity Sand-Fines Mixtures

Muttana S. Balreddy, S. V. Dinesh, and T. G. Sitharam

Abstract During earthquake, soil deposits are subjected to cyclic loading. Under undrained conditions, cyclic shear stress causes transient disturbance and gives rise to an increase in excess pore water pressure leading to loss of shear strength in saturated soil deposits, there will be excessive strains with continued loss of shear strength resulting in liquefaction. Researchers have identified a significant number of cases where ground failure took place during earthquake in soil deposits containing fines leading to considerable damage to buildings. Some studies show a decrease in liquefaction resistance and others show an increase in liquefaction resistance with an increase in fines content in soil deposit. There is no clarity regarding the effect of fines. In view of this, an experimental investigation has been carried out to evaluate the liquefaction potential of sand-clay mixtures. A series of undrained stress-controlled cyclic triaxial shear tests were performed on reconstituted samples obtained from Cauvery River sand and sand mixtures containing different percentages of plastic fines up to 30%. This paper reports the results of stress-controlled cyclic triaxial shear tests under undrained conditions at 100 kPa confining pressure at a frequency of 0.1 Hz. The results show that the cyclic strength decrease with an increase in fines content upto 20% beyond which it increases. In the present investigation, the limiting fines content is 20%. The results of CRR were analyzed in terms of relative density, fines content, and plasticity characteristics. The results indicate that flow liquefaction depends on the acceleration (CSR) and initial state (void ratio).

Keywords Fines content · Liquefaction · Cyclic resistance ratio

M. S. Balreddy (✉) · S. V. Dinesh
Department of Civil Engineering, Siddaganga Institute of Technology, Tumakuru 572103, India
e-mail: msb@sit.ac.in

S. V. Dinesh
e-mail: dineshsv2004@gmail.com

T. G. Sitharam
Indian Institute of Technology, Guwahati 781039, India
e-mail: proftgs@gmail.com

T. G. Sitharam et al. (eds.), *Soil Dynamics*, Lecture Notes in Civil Engineering 119,
https://doi.org/10.1007/978-981-33-4001-5_17

1 Introduction

Among the seismic hazards, soil liquefaction is a major threat to human life, infrastructure, and environment. Many historical earthquakes have noticed the phenomena of soil liquefaction, some of them are 1964 Niigata, Japan, 1964 Alaska, USA and 2001 Bhuj, India. The most important step towards mitigating liquefaction hazard is by evaluating liquefaction potential of soil accurately. Although loose saturated cohesionless soils are most susceptible to loss of strength, there are studies and evidences suggesting that soils containing fines are also prone to liquefaction during earthquakes**.** The presence of a certain amount of fines in sand was generally considered to increase the resistance but a detailed literature review of laboratory and field investigations has clearly highlighted the conflicting evidence of the effect of fines on the liquefaction behavior of sand-fines mixtures. The situation is, however, more complicated for low-plasticity silts and clays that are near the transition zone between "sand-like" and "clay-like" behavior.

The factors affecting and the mechanism associated with the liquefaction potential of sands have been reasonably understood. Many natural and artificial soils contain combinations of fine-grained (i.e., clay and silt) and coarse-grained (i.e., sand and gravel) soils. Such soils are called intermediate or mixed soils or mixtures, in which clay contents are present in different percentages. The literature review indicates that the presence of clay fines affects the liquefaction behavior. There are studies that suggest that sand-clay mixtures are non-liquefiable when clay fines are <20% and in some cases they are liquefiable when clay fines are >15%. Some studies have indicated decrease in liquefaction resistance with an increase in plasticity and vice-versa [2, 6, 7, 10]. Some studies have reported decrease of CRR till limiting fines content. Some researchers reported that increase in plasticity reduces the liquefaction resistance of silty soils [8]. There is wide variation in the recommendations on the limiting boundaries of liquid limit and PI values on the Atterberg's limit chart for liquefaction susceptibility. Literature suggests flow liquefaction for sand-fines mixtures till $PI = 7$ and cyclic mobility when $PI > 7$ [5]. This is based on plasticity of mixtures without any reference to the level of shaking as reflected by cyclic stress ratio and initial state (relative density). Hence, it is necessary to study the liquefaction behavior of sand-fines mixtures under cyclic loading to understand the above issues related to flow liquefaction dependence on plasticity characteristics, relative density, cyclic stress ratio. The present paper reports the effect of plasticity characteristics, relative densities, and CSR on the flow liquefaction of low to medium plasticity sand-fines mixtures.

2 Materials and Experimental Methods

To understand the effect of fines on liquefaction resistance, sand and fines were used in the present investigation. Sand was procured from the Cauvery River bed, Karnataka,

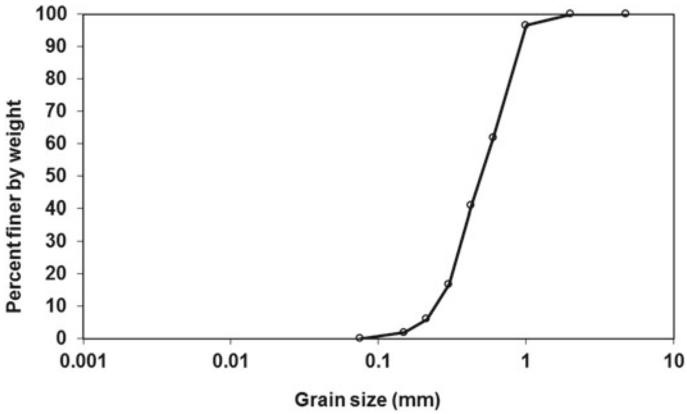


Fig. 1 Grain size distribution curves of Cauvery River clean sand (S)

India. Clayey sand was procured from locally available sources in Tumakuru region, Karnataka, India. Figure 1 shows grain size curve of clean Cauvery river sand (S). The grain size curve of sand lies within the boundaries of most liquefiable soils as proposed by Tsuchida [9]. This clearly indicates that sand under undrained conditions would liquefy during an earthquake. The specific gravity (G_s) of sand is 2.62, coefficient of curvature (C_c) is 0.9, coefficient of uniformity (C_u) is 2.4, maximum void ratio (e_{max}) is 0.99, and minimum void ratio (e_{min}) is 0.72. From the above data according to IS classification, the sand is classified as poorly graded sand (SP).

Clayey sand is available at shallow depths of 1.5–15 m from the ground level in the southern parts of Karnataka. It is white-yellowish in color. The fines (<75 μ) extracted from the clayey sand soil are designated as F. The specific gravity (G) is 2.65. The soil composition consists of gravel 01%, sand 53%, silt 28%, and clay 18%. The liquid limit is 46, plastic limit is 27, and the plasticity index is 19. The fines chosen are plastic in nature. From the above data according to IS classification, the soil is Clayey Sand (SC).

In order to study the influence of fines in the sand on liquefaction resistance, fines were separated from clayey sand and mixed with clean Cauvery River sand (S) to prepare sand-fines mixtures. The details of sand-fines mixtures prepared are shown in Table 1. The fines fraction of size less than 0.075 mm was separated from clayey sand and used as fines in the experimental program. These clay fines were mixed with the clean Cauvery River sand (S) in various proportions ranging from 0 to 30%. The three sand-fines mixtures considered are SF10, SF20, and SF30. The index properties of sand-clay mixtures refer Balreddy et al. [3]. With the addition of fines to sand, the sand-fines mixtures changes to non-plastic sand-silt mixture at 10%, sand-clay or sand-silt of low plasticity at 20% fines and sand-clay of medium plasticity at 30% of fines.

Table 1 Details of liquefaction susceptibility for soil mixtures

Soil mixtures	Fines content (%)	Liquid limit (%)	Plasticity index	Relative density (%)	CSR	CRR	Residual pore pressure ratio	Sample failure by Flow Liquefaction (FL)/Cyclic Mobility (CM)
SF00	0	0	0	50	0.15	0.162	1.0	FL
SF10	10	22	0		0.145	0.13	1.0	FL
SF20	20	26	6		0.144	0.11	1.0	FL
SF30	30	32	17		0.15	0.125	1.0	FL
SF10	10	22	0	70	0.168	0.17	1.0	FL
SF20	20	26	6		0.17	0.14	0.95	CM
SF30	30	32	17		0.165	0.155	0.92	CM

2.1 Sample Preparation

For achieving the loosest possible density, triaxial test specimens of 50 mm diameter and height 100 mm were prepared by dry deposition method. Required quantity of sand and fines were weighed. Sand and fines were divided into 10 equal parts. Each of the sand and fines portions was mixed thoroughly to have a uniform mixture. These mixture portions were filled in a membrane lined to the split mold attached to the bottom platen of the triaxial cell by using a spout of 12 mm diameter at the bottom. The spout was slowly raised along the axis of symmetry of the specimen, such that the soil was not allowed to have any drop height. The sand-fines slide down slowly. The mold was gently tapped with a wooden mallet to achieve the required density for each layer. Then the sample was subjected to low-pressure vacuum and carbon-di-oxide gas was passed for one hour. After this, de-aired distilled water was passed through the soil sample at a very small head. The specimens were saturated by applying a back pressure of 100 kPa and saturation time was maintained till the Skempton's pore pressure coefficient '*B*' value exceeds 0.95.

An effective confining pressure of 100 kPa was applied and specimens were isotropically consolidated till the end of Casagrande's primary consolidation. The reduction in the void ratio before and after consolidation is found to be in the range of 0.027–0.066. An increase in the relative density after consolidation by 5–10% was observed compared to the initial relative density. In this way, the samples were prepared at relative densities of 50, 60, 70, and 80%.

2.2 *Stress-Controlled Tests*

In the present work in order to simulate the stress conditions existing during earthquakes, the cyclic triaxial test facility was used to conduct stress-controlled undrained cyclic triaxial shear test. The tests were performed at 100 kPa confining pressure and frequency of 0.1 Hz. ASTM: D 5311-92 (Reapproved 1996 [1]) was followed for conducting tests.

The load, deformation, cell pressure, pore water pressure data were obtained by a computer-controlled data acquisition system. In each loading cycle, 128 data points were captured. Tests were continued till an excess pore pressure is equal to confining pressure or 5% double amplitude axial strain is reached. For more details regarding the full experimental program and cyclic stress ratio amplitudes applied during the tests see Balreddy et al. [3].

3 Results and Discussion

Figure 2 shows the results of cyclic triaxial test under stress-controlled conditions on a sample of sand (SF00) having a void ratio of 0.855 ($D_r = 50\%$) at 100 kPa confining pressure at a loading frequency 0.1 Hz.

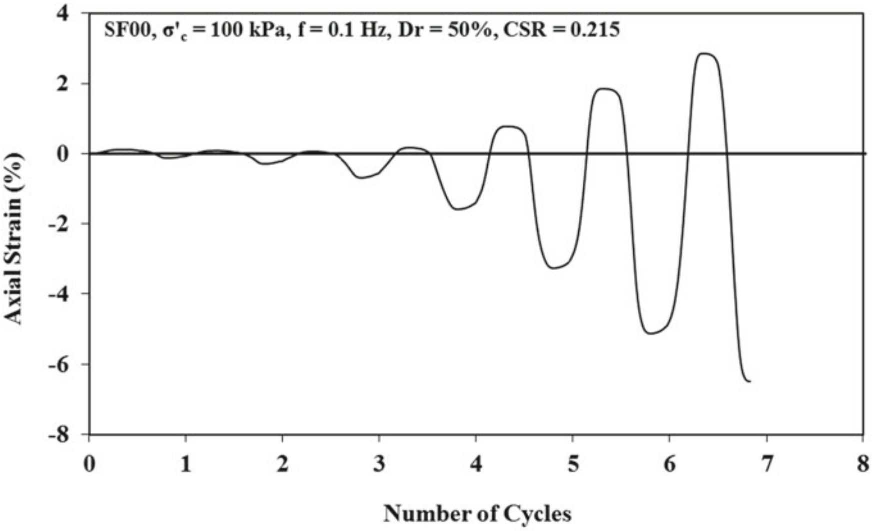
Figure 2a shows the plot of axial strain versus number of cycles. Beyond 5 cycles, the rate of development of axial strain is faster. The sample shows a large amount of axial strain on the extension side when compared to the compression side. The sample is considered to be failed when the double amplitude axial strain is 5% or when 100% excess pore water pressure develops. In this case, the sample has failed at 6th cycle.

Figure 2b shows the plot of residual pore pressure ratio versus cycle ratio. The residual pore water pressure buildup is increasing uniformly till cycle ratio of 0.7 beyond which it develops at a faster rate and attains a value of 1. The sample failed by flow liquefaction. The residual pore water pressure is responsible for the reduction in effective stress and strength loss.

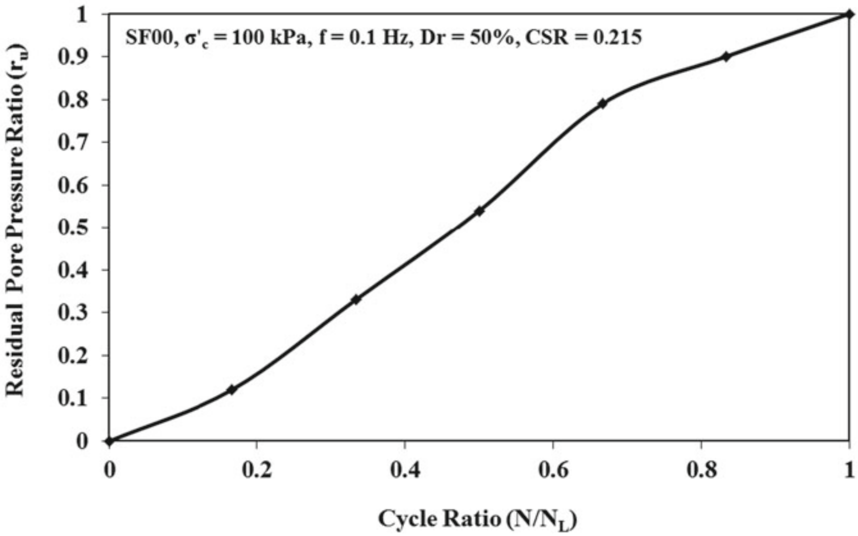
Figure 2c shows the plot of double amplitude axial strain versus number of cycles. It is observed from the plot that the double amplitude axial strain is constant till 4th cycle. Beyond 4th cycle, there is a sudden buildup of axial strain. Failure is observed at 6th cycle where the double amplitude axial strain value reaches more than 5%.

Figure 2d shows the plot of deviator stress versus effective mean stress. It is observed that in the beginning the sample is subjected to low axial strain and remains constant till 4th cycle. After 4th cycle, the loop widens and enlarges with large deformation on the extension side.

Figure 3 shows the plot of cyclic resistance ratio (CRR) versus fines content at varying void ratio. The cyclic strength is more for the clean sand. For details of cyclic strength of SF00 and SF10 refer Balreddy et al. [3]. The results show that the cyclic strength decrease with an increase in fines content upto 20% beyond which it

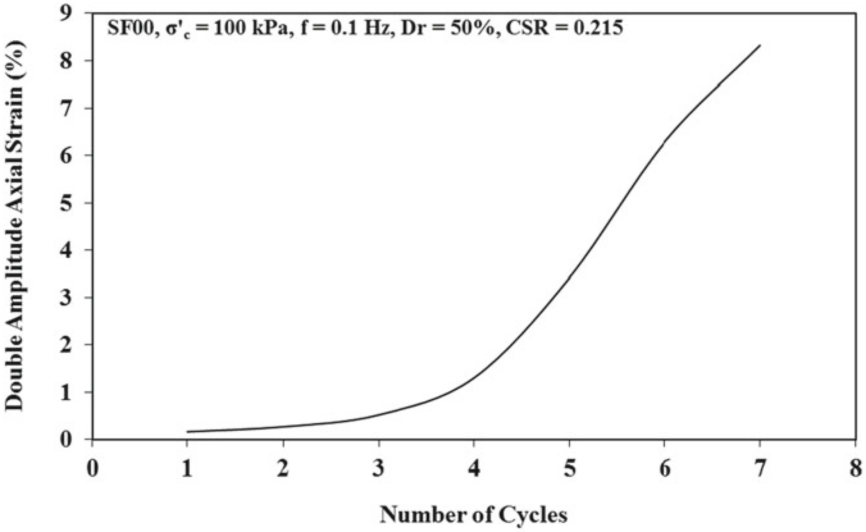


a) Axial strain versus number of cycles

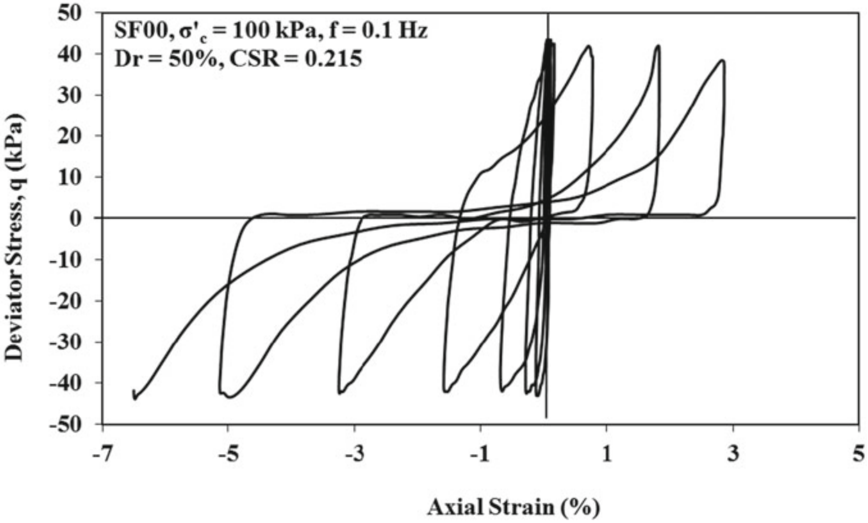


b) esidual pore pressure ratio versus cycle ratio

Fig. 2 Typical cyclic triaxial test results of sand (SF00) sample having $e = 0.855$



c) Double amplitude axial strain versus number of cycles



d) Deviator stress versus Effective mean stress

Fig. 2 (continued)

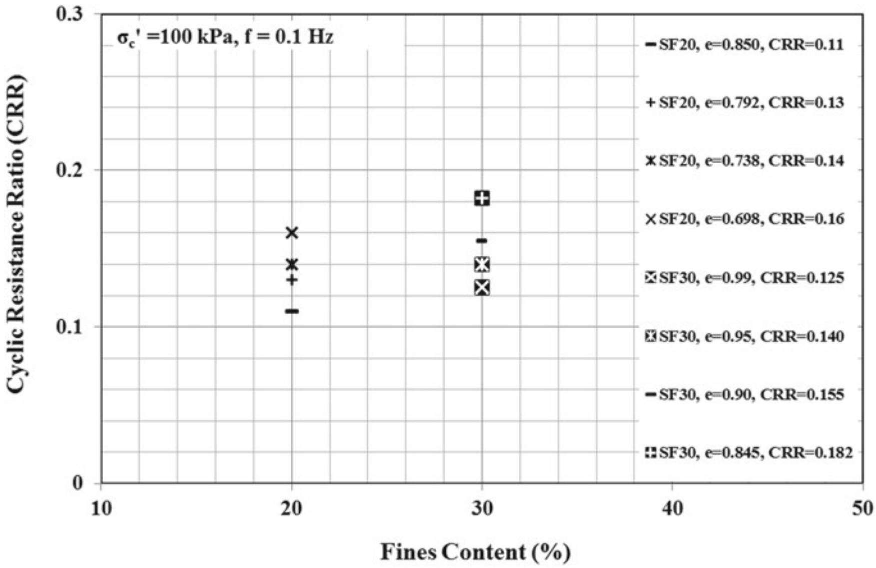


Fig. 3 Cyclic resistance ratio (CRR) versus fines content

increases. In the present investigation, the limiting fines content is 20%. Upto 20% fines content, the voids in the sand specimen will not be completely filled by fines. This results in a decrease in friction as the fines in the voids facilitate sliding. There will be a rearrangement of particles leading to increased compressive tendency which results in buildup of a large amount of excess pore water pressure and suppression of dilation thereby; there will be a reduction in strength. Cyclic strength of sand-fines mixtures at this stage is minimum due to the combined effect lubrication effect between sand and fines and suppression of dilation. Further, cyclic strength increases with the increase in the fines content beyond 20% fines content. This increase of cyclic strength is due to the fact that the fines matrix will control the behavior and sand-fines mixture shows clay-like behavior.

Table 1 shows the details of liquefaction susceptibility for soil mixtures. Sand-fines mixtures with $D_r = 50\%$ (loose state) and 70% (dense state) containing 10% fines have failed by flow liquefaction. However, dense sand-fines mixtures with 20 and 30% fines content at $D_r = 70\%$ with PI between 6 and 17 fail by cyclic mobility when the CSR > CRR. These samples exhibit clay-like behavior. The pore pressure buildup is faster with an increase in PI in loosely compacted specimens ($D_r = 50\%$) when compared to dense specimens ($D_r = 70\%$) and these samples have failed by flow liquefaction and with these sand-like behavior is observed.

Figure 4 shows the plot of cyclic resistance ratio (CRR) versus fines content at constant void ratio. It is observed that at any given void ratio, the cyclic resistance ratio decreases with an increase in fines content till limiting fines content (LFC = 20%). This indicates that the fines in the voids facilitate sliding producing soft contacts. At any given constant void ratio, CRR is minimum at limiting fines content.

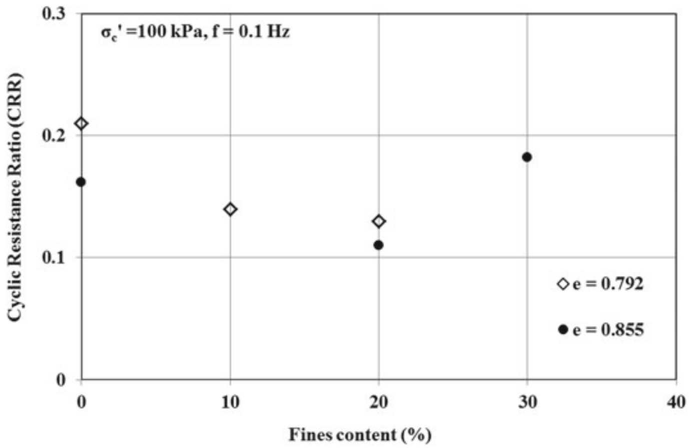


Fig. 4 Cyclic resistance ratio versus fines content at constant void ratio

The reduction in CRR is of the order 65–68% of the base sand strength (Balreddy et al. [3]).

Figure 5 shows the plot of cyclic resistance ratio versus sand skeleton void ratio for sand and sand-fines mixtures. It is observed that CRR decreases with an increase in sand skeleton void ratio. Beyond the limiting fines content of 20%, CRR increases due to the plasticity effect. For a given fines content, CRR decreases with an increase

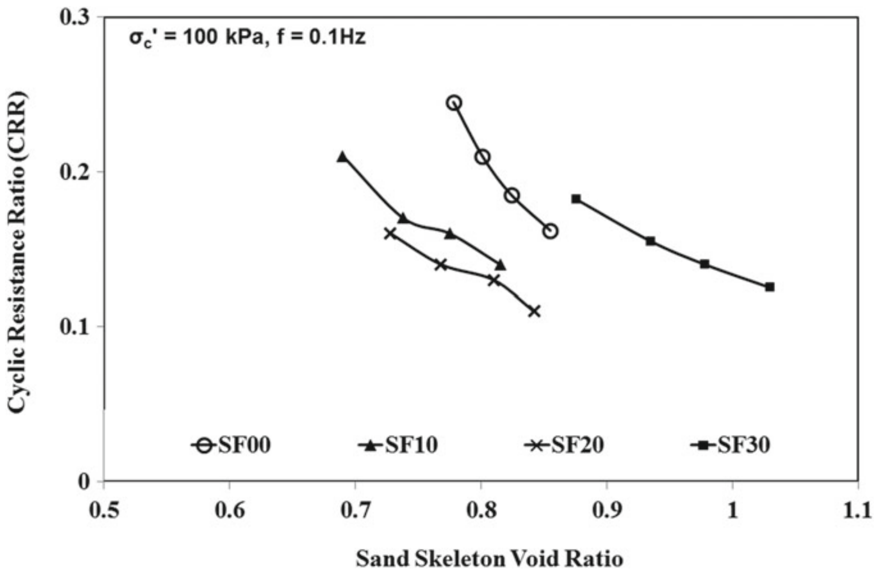


Fig. 5 Cyclic resistance ratio (CRR) versus sand skeleton void ratio

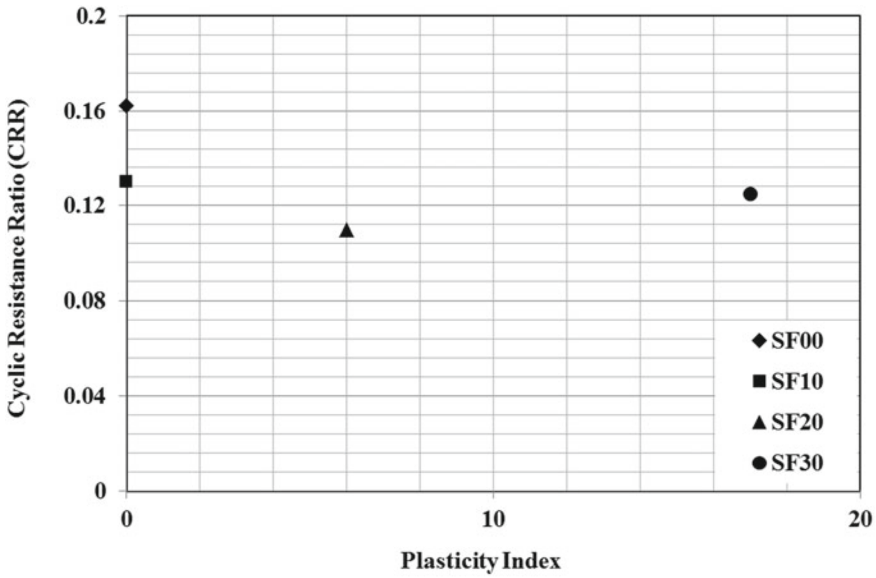


Fig. 6 Cyclic resistance ratio (CRR) versus plasticity index

in sand skeleton void ratio, also when fines content is less than limiting fines content for 10 and 20%. There is a decreasing trend with increase in sand skeleton void ratio. In general, the CRR ranges from 0.21 to 0.125 for all fines content considered in this study.

Figure 6 shows the plot of cyclic resistance ratio (CRR) versus plasticity index for the sand-fines mixtures. It is observed that clean sand has higher CRR value compared to sand-fines mixtures. Addition of fines beyond the limiting fines content, CRR increases marginally with an increase in plasticity index more studies are necessary. It is observed that when PI is less than or equal to 6, the sand-fines mixtures indicate silt like behavior characterized by flow liquefaction. When PI > 6, sand-fines mixtures indicate clay-like behavior characterized by cyclic mobility.

4 Conclusions

Based on the analysis of the results of the cyclic behavior of sand-fines mixtures, the following conclusions are drawn.

1. Addition of low plastic fines (PI = 6–17) decreases the CRR till limiting fines content of 20% beyond which it increases. The decrease of CRR due to fines content is in the range of 65–68% compared to base sand.
2. Addition of low plasticity fines beyond the limiting fines content shows a marginal increase in CRR with an increase in plasticity index.

3. Sand-fines mixtures under loose and medium dense condition ($D_r < 70\%$) fail by flow liquefaction exhibiting sand-like behavior.
4. Medium plasticity sand-fines mixtures under dense conditions ($D_r \geq 70\%$) with fines in excess of limiting fines content fail by cyclic mobility exhibiting clay-like behavior and these results are in agreement with Bayat et al. [4].

References

1. ASTM Designation: D 5311–92 (1996) Standard test method for load controlled cyclic triaxial strength of soil. In: Annual book of ASTM standards, vol 04.08
2. Balreddy MS, Dinesh SV, Sitharam T (2016) G: Influence of sand and low plasticity clay mixtures on the liquefaction and postliquefaction behavior. Jpn Geotech Soc Spec Publ 2(21):806–810
3. Balreddy MS, Dinesh SV, Sitharam TG (2019) Effect of fines on pore pressure development during cyclic loading. In: Adimoolam B, Banerjee S (eds) Soil dynamics and earthquake geotechnical engineering. Lecture Notes in Civil Engineering, vol 15. Springer, Singapore, pp 83–90
4. Bayat M, Bayat E, Aminpour H, Salarpour A (2014) Shear strength and pore-water pressure characteristics of sandy soil mixed with plastic fine. Arab J Geosci 7(3):1049–1057 (2014)
5. Boulanger RW, Idriss IM (2004) Evaluating the potential for liquefaction or cyclic failure of silts and clays. Report UCD/CGM-04/01. University of California, Davis, California
6. Gratchev IB, Sassa K, Fukuoka H (2006) How reliable is the plasticity index for estimating the liquefaction potential of clayey sands. J Geotech Geoenviron Eng ASCE 132(1):124–127
7. Ishihara K, Koseki J (1989) Discussion of cyclic shear strength of fines-containing sands. In: Earthquake geotechnical engineering, 12th international conference on soil mechanics and foundation engineering, Rio de Janeiro, pp 101–106
8. Maheshkumar G (2012) Effect of fines on the cyclic response of sandy soil. Ph.D. thesis submitted to Visvesvaraya Technological University, Belgaum
9. Tsuchida H (1970) Prediction and countermeasure against liquefaction in sand deposits. Abstract of the seminar in the Port and Harbor Research Institute, pp 3.1–3.33
10. Yasuda S, Wakamatsu K, Nagase H (1994) Liquefaction of artificially filled silty sands. Ground failure under seismic conditions. Geotechnical Special Publication No. 44, ASCE, pp 91–104