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Strain Criterion For Initiation Of Liquefaction Using Shake Table Test

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Abstract

Earthquake induced liquefaction is one of the most natural disastrous calamities. In order to mitigate the hazards produced by liquefaction, the crucial steps used to evaluate the initiation of liquefaction are offered by most commonly used approaches namely cyclic stress and cyclic strain approach. An attempt is made herein to evaluate the strain criterion for initiation of liquefaction in clean sand using shake table test. Total eight tests are conducted on the laboratory shake table apparatus on clean sand at different acceleration levels and relative densities. All the samples in the present investigation have been subjected to a predetermined quantity of surcharge to simulate the actual field condition. The test results show that the threshold shear strain (γ_t) is of the order of 10^{-4} % to 10^{-3} % whereas shear strain at initiation of liquefaction (γ_L) is around 0.01% in all the tests. Lesser values of γ_t are observed for lower input acceleration while increase in γ_t is noticed for higher level of acceleration. In short, the present research strongly emphasizes on the development of a strain based criterion for initiation of liquefaction using shake table test leading to evaluation of threshold shear strain (γ_t) and magnitude of shear strain at point of initiation of liquefaction (γ_L) for clean sand.

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1. Introduction

Earthquake induced liquefaction is one of the most devastating calamities that occurs as a natural consequence of ground motion. In order to prevent or to minimize its devastating effects, the most important step is to find the initiation of liquefaction. The most commonly used approaches have been the cyclic stress approach and the cyclic strain approach. However, the cyclic stress approach is very sensitive to the parameters like preshaking [3], fabric effect [9], over consolidation ratio [4], and aging effect etc. thus making laboratory evaluation of initiation of liquefaction exceedingly difficult. Hence, the focus of the present study is to actually employ the strain approach chiefly due to its following desirable attributes: (i) Strain approach is fairly robust to the variations in the parameters mentioned above [14]. (ii) Pore water pressure generation is a hallmark of all liquefaction phenomena and is caused by the rearrangement of soil particles. This rearrangement is best characterized by shear strain rather than shear stress. [12, 14]. (iii) Finally, the strain based approach allows one to investigate the threshold shear strain (γ_t) required for generation of pore water, which is one of the measures of onset of liquefaction.

Several researchers have studied pore pressure generation and initiation of liquefaction using strain controlled cyclic triaxial test [1, 11, 14, 17]. However, for reliability of the inferences drawn, it is necessary to simulate field conditions during laboratory evaluation of liquefaction. This task of field simulation can be accomplished in a meaningful manner through contemplated shake table tests [15]. Thus, current research strongly emphasizes on the development of a strain based criterion for initiation of liquefaction using shake table test leading to evaluation of threshold shear strain (γ_t) and magnitude of shear strain at point of initiation of liquefaction (γ_L) for clean sand.

2. Literature review

Several investigators using cyclic triaxial test [11, 13, 14,] have consistently suggested the existence of threshold shear strain (γ_t) of approximately 1×10^{-2} % for clean sand. For cyclic shear strains below this threshold, there is neither densification nor prestraining of dry sands, and there is no pore water pressure buildup in saturated sands. However, the literature is very scanty on the use of shake table test to evaluate initiation of liquefaction with strain criterion.

In an effort to investigate the in situ liquefaction potential by the cyclic strain approach, Hazirbaba and Rathje [10] observed that both the laboratory and in situ testing results reflect a threshold strain; below which no pore pressure generation occurs whereas beyond the threshold shear strain there is a sudden increase in pore pressure generation. The threshold strain ranged from about 0.005% to 0.01 %. Further, it is observed that excess pore pressure generation is strongly influenced by the vertical effective stress. Higher vertical effective stress results in less pore pressure generation. The literature survey thus reveals that the overburden pressure affects significantly the pore water pressure generation and magnitude of threshold shear strain. Therefore, it is decided to conduct all the tests in the proposed work by providing surcharge over the soil in the soil model to achieve the simulation between test condition and field condition as overburden pressure always exists on the field.

3. Experimental investigation

3.1 Selection of soil sample

The soil sample selected for the present research work is uniformly graded clean sand which has been observed to fall well within the boundaries for most liquefiable soil as given by Iwasaki [16] reproducing Tsuchida's [2] work. The index properties of the clean sand used in the present research work are as summarized in Table 1.

Table 1. Properties of clean sand

Properties of sand	Value	Properties of sand	Value
γ_{max}	16.51 kN/m^3	D_{30}	0.18 mm
γ_{min}	13.81 kN/m^3	D_{50}	0.25 mm
G	2.63	D_{60}	0.28 mm
e_{max}	0.868	C_u	2.15
e_{min}	0.563	C_c	0.89
D_{10}	0.13mm	--	--

3.2 Soil model

All the tests in the present work are conducted by employing a square soil model of size 400 × 400 × 400 mm. (Fig. 1)

3.3 Surcharge model

For the application of surcharge, a specially designed and fabricated surcharge model [Fig. 2(a)] is carefully lowered into soil model resting on the top layer of soil. Fig. 2.1. (b) shows openings provided in the surcharge model through which instruments like accelerometer and strain transducer are inserted within actual soil matrix. Also, during the test, care has been taken that water should not come out through openings or gap between soil model and surcharge model. The desired amount of surcharge is applied by filling calculated weight of dry sand in the surcharge model.

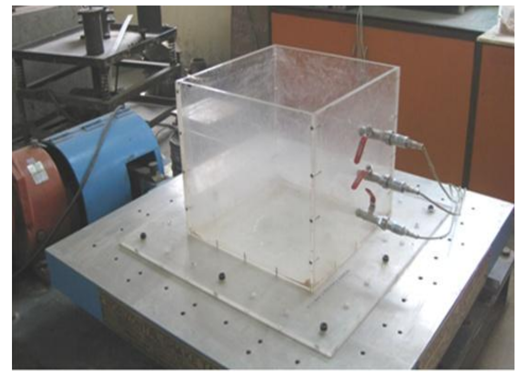

Figure 1. Soil Model (400 X 400 X 400 mm)

(a)

(b)

Figure 2. Surcharge Model (375 X 375 X 200 mm)

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3.4 Instrumentation

3.4.1 Pore pressure transducers. As generation of pore water pressure is a signature of the liquefaction phenomenon, these transducers play a major role. Three pore pressure transducers are connected along the direction of shaking at 0.1m, 0.2m and 0.3m height from the bottom of soil.

3.4.2 Linear variable displacement transducer (LVDT). LVDT is connected along the direction of shaking for determination of displacement of base plate of shake table.

3.4.3 Accelerometers. In order to measure actual acceleration transferred to the soil from the base of shake table, waterproof accelerometer is inserted at a height of 0.1m from the base.

3.4.4 Strain Transducer. Strain transducer is inserted in soil at a height of 0.1m from the bottom in a direction perpendicular to the direction of shaking.

3.5 Data acquisition system

The proposed work mainly comprises of study of liquefaction phenomenon which occurs within a very short span of time. Study of such a phenomenon requires adequate data which are collected with the help of data acquisition system at a sampling rate of 50 saps (i.e. 50 samples per second). Eight channel data acquisition system is installed for current research work, out of which three pore pressure transducers are connected to first three channels, accelerometers are connected to the fifth and sixth channel and LVDT and strain gauge are connected to seventh and eighth channel respectively. (Fig. 3)

Figure 3. Data acquisition system

3.6 Testing program

A total of eight tests are conducted on clean sand; exhibiting properties given in Table 1 with relative densities 30%, 40%, 50% and 60% representing a sufficiently wide range of loose to dense state. For each of the states, the sample is subjected to two acceleration levels viz. 0.21g and 0.45g.

3.7 Testing program

One of the indispensable aspects of test procedure is the method of sample preparation; with the stringent requirement of obtaining a homogenous sample with specific uniform density. In view of this requirement, the water sedimentation method is adopted as it ensures complete saturation. The soil model is filled in six equal layers by pouring sand from specific height using the raining technique, to achieve the desired density.

Amplitude on shake table is set with the help of a crank – shaft arrangement and the frequency on the control panel is set in accordance with the maximum acceleration required before filling the model. Subsequently, the LVDT and pore pressure transducers are then connected to the data acquisition system. Strain transducer and accelerometer are inserted at their designated positions while pouring the soil. All these instruments are further connected to the data acquisition system to record the data in real time. The surcharge model is then carefully placed on the top surface of the soil layer in the soil model. It is then filled with clean sand so that a maximum possible surcharge of 2.5 kPa is applied to the soil sample in the soil model. Fig. 4 shows the soil specimen along with surcharge of 2.5 kPa ready to undergo the test.

Figure 4. Soil Specimen in soil model with surcharge

Now the shake table is switched on, which starts shaking with the predefined acceleration. The knobs of pore pressure transducers are opened precisely at the

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instant of start of shaking. The shake table is accelerated till pore pressure is decreased or showed a constant value after attaining peak value with respect to initial value. This stage is considered as initiation of liquefaction and the test is continued till complete dissipation of pore water pressure occurs.

4. Results

The parametric study of the data recorded during the all the tests is carried out. The details about input acceleration time history are stated in the next section. Also, typical variation of excess pore water pressure, pore pressure ratio and shear strain is discussed in subsequent sections.

4.1. Input acceleration time history

In order to provide the predefined acceleration to the base of shake table, i.e. 0.21g and 0.45g, the frequency selected was 2 Hz and 3 Hz respectively with 15 mm of amplitude. Fig. 5 shows a typical a-t history for 0.21g; which also confirms that the acceleration as per the testing program is provided to the shake table base.

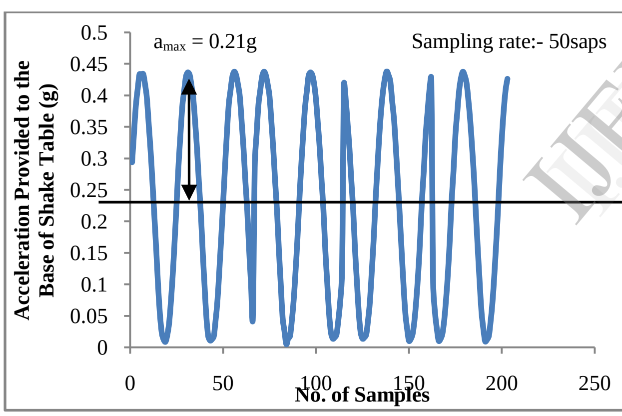
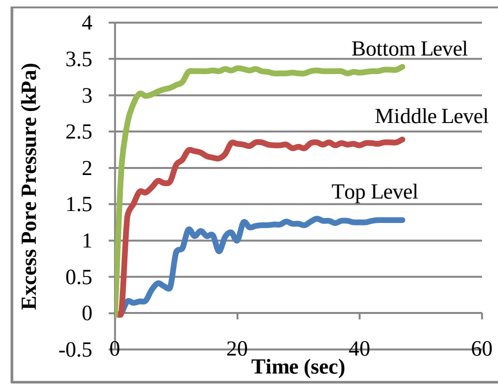


Figure 5. Acceleration provided to the base of shake table for a_{max} = 0.21g

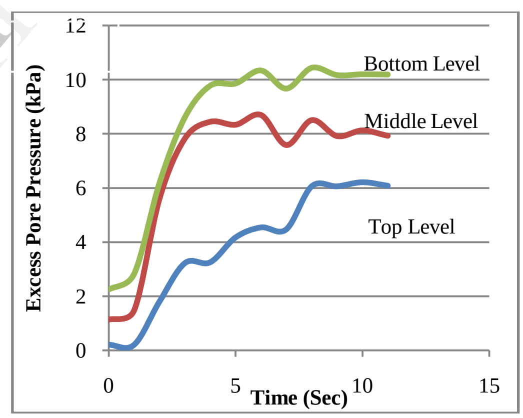
4.2 Typical variation of excess pore pressure vs. time

Figures 6 (a) and (b) show the variation of excess pore pressure with time for all the three levels (i.e. at top, middle and bottom level) typically for soil sample of 30% relative density tested at 0.21g [Fig. 6 (a)] and 0.45g [Fig. 6 (b)].

It is seen from the above figures (Figs. 6 (a) and (b)) that generation of pore pressure is higher at the bottom level as compared to the one at middle and top level, which can be readily traced back to the major fact that the amount of surcharge at bottom level is higher than that at other two levels. Similar trend is observed for test specimens with D_r = 40%, 50% and 60% subjected to acceleration levels of 0.21g and 0.45g. Thus, (for all the test results) it is discernible that pore water pressure first develops at the top of the sample and then proceeds to middle and eventually to the bottom. Thus, liquefaction proceeds in the downward direction for the laboratory shake table apparatus which agrees well with earlier work [5].



(a)



(b)

Figure 6. Variation of excess pore pressure vs. time for D_r = 30% (a) a_{max} = 0.21g (b) a_{max} = 0.45g

4.3 Typical variation of excess pore pressure vs. time for varying acceleration level

Figures 7 (a), (b) and (c) present the variation of excess pore pressure vs. time at top, middle and bottom level respectively typically for 30% relative density tested at 0.21g and 0.45g.

It is clear from the figures [Fig. 7 (a), (b) & (c)] that as the acceleration level increases from 0.21g to the more vigorous value of 0.45g, peak pore pressure also increases significantly from 1.15 kPa to 6.21 kPa at top level, 2.24 kPa to 8.51 kPa at middle and 3.37 kPa to 10.44 kPa at bottom level transducer. Further,

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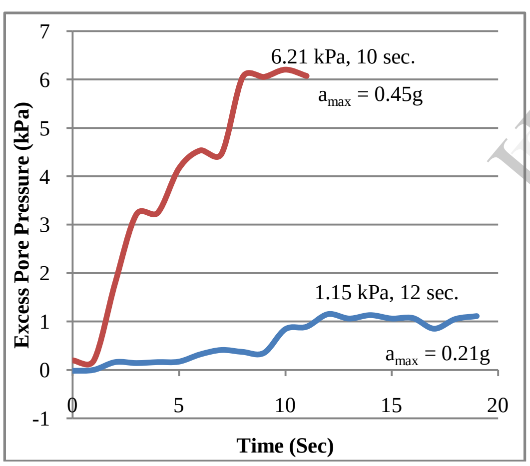
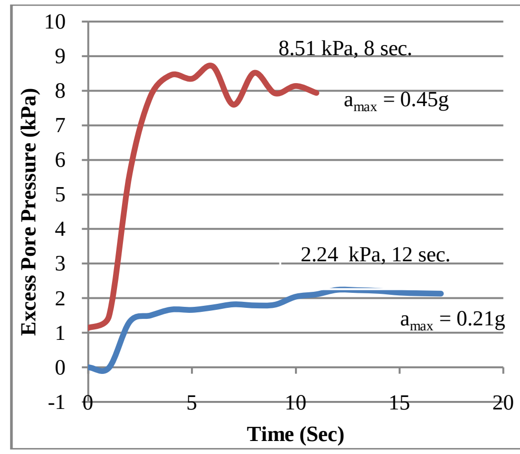
the rate of generation of excess pore pressure also increases by 5 kPa, 6 kPa and 7 kPa for the three respective levels with the same increase in acceleration.

Also, it is noticed that this increase in peak pore pressure at higher acceleration occurs within a very short duration as compared with that at lower acceleration for the same state of soil at the same level of pore pressure transducer. Thus, the peak value of excess pore pressure 3.37 kPa reached in 20 sec. when sample of relative density 30% is accelerated at 0.21 g while that of 10.44. kPa reached within 8 sec. for the same sample after subjecting it to shaking at 0.45g [cf.Fig. 7(c)].

Similar results as that for 30% relative density are obtained for tests performed on samples with other relative densities varying from 40% to 60% and tested at both the acceleration levels of 0.21g and 0.45g. Therefore, it is inferred that as the acceleration level increases, the peak pore pressure also increases significantly for all relative densities tested in the present work. Moreover, this increase in peak pore pressure occurs within a very short duration (4 to 5sec) for high level of acceleration as against that (20 to 24 sec) at low acceleration level.



(a)



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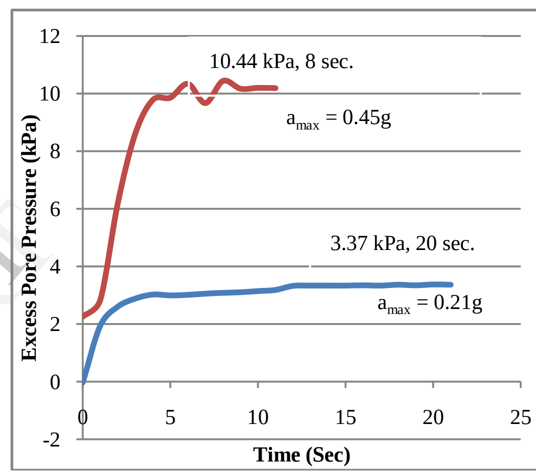

(c)

Figure 7. Variation of excess pore pressure vs. time for varying acceleration (a_{max} = 0.21g, 0.45g) & at D_r = 30% (a) Top level (b) Middle level (c) Bottom level

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4.4 Variation of pore pressure ratio vs. shear strain

In order to find the threshold shear strain (γ_t) and strain at initiation of liquefaction (γ_L), test results for the pore pressure ratio vs. shear strain are plotted for each relative density (D_r varying from 30% to 60%) and for both the acceleration levels (a_{max} = 0.21g and 0.45g). It is observed from these results that there is a sudden increase in pore pressure ratio and thus the corresponding strains are designated as threshold shear strain (γ_t) [14]. The magnitude of γ_t for each test is specifically mentioned in the Table 2. It is interesting to note that magnitudes of γ_t for all the tests are within a range of 10^{-4} % to 10^{-3} % of shear strain.

Table 2. Magnitude of threshold shear strain and strain at initiation of liquefaction for different relative densities and acceleration levels

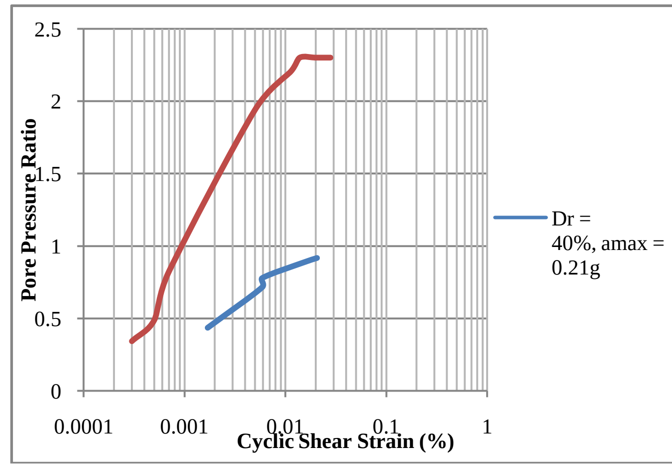
Sr. No.	Relative Density (D_r)	a_{max} (g)	Threshold Shear Strain (γ_t)	Shear Strain at Initiation of Liquefaction (γ_L)
1	30%	0.21	0.0001%	0.01%
2	40 %		0.0017%	0.0206%
3	50%		0.0026%	0.0119 %
4	60%		0.0001%	0.0173%
5	30%	0.45	0.005%	0.0245%
6	40 %		0.0003%	0.0139%
7	50%		0.0013 %	0.0145%
8	60%		0.0025%	0.0161%

Further, it is found from all the results that irrespective of acceleration and relative density; at around 10^{-2} % of shear strain, pore pressure ratio has reached its peak value (which is the indication of initiation of liquefaction) [5]. Therefore, the corresponding shear strain value (i.e. around 10^{-2} %) has been defined in the present work as a shear strain at initiation of liquefaction (γ_L). The magnitudes of γ_L for all the tests are also exclusively mentioned in Table 2.

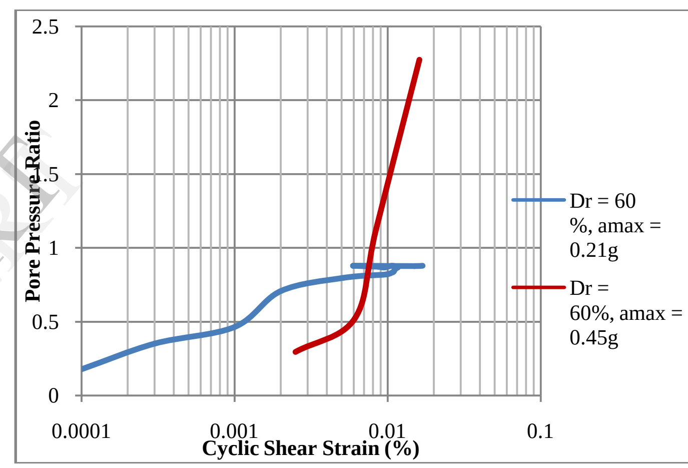
Figures (Fig. 8 (a) and (b)) show a typical variation of pore pressure ratio vs. shear strain for relative densities 40% and 60% indicative of loose and dense state of soil respectively. The pore pressure ratio is calculated as

$$\frac{Excess\ Pore\ Pressure\ generated}{Vertical\ Efeetive\ Stress} = \frac{\Delta U}{\sigma'} \quad (1)$$

It can be seen from the test results that for pore pressure ratio less than unity, initiation of liquefaction is observed for the lower level of acceleration (i.e. 0.21g) whereas for the higher level of acceleration (i.e. 0.45g) liquefaction initiates when pore pressure ratio exceeds unity. [cf.Fig. 8(a) and (b)]. This is in agreement with a similar observation made by Singh et al. [5]. The trend observed (i.e. Pore pressure ratio exceeds the magnitude of one) is mainly due to the higher level of shaking.



(a)



(b)

Figure 8. Variation of excess pore pressure ratio vs. shear strain for varying acceleration (a_{max} = 0.21g and a_{max} = 0.45g (a) D_r = 40% (b) D_r = 60 %

5. Comparison with previous research work

In accord with the previous investigation cited above, the vertical effective stress strongly influences the value of threshold shear strain. The same is carried out in the present research work by employing shake table test and compared with the results obtained earlier, as shown in Fig.9.

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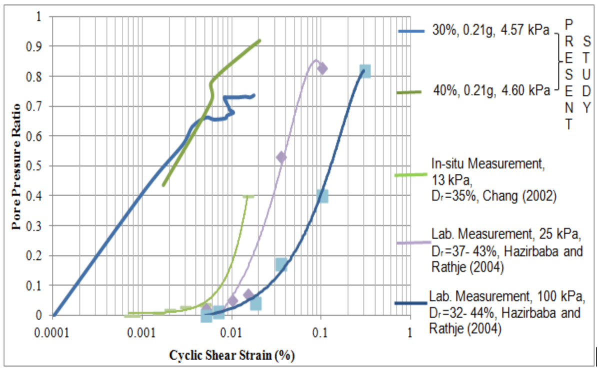

Figure 9. Comparison of test results obtained from present study (at a_{max} = 0.21g) with previous research work [Hazirbaba K. and Rathje E.M. (2004)]

It is manifest that though the present work results are for a similar relative density range, the trend is different than that obtained by the earlier researchers. This could be due to the following features of distinction: nature of soil specimen (Present research work – clean sand with D_{50} = 0.25 mm, Previous investigation – uniformly graded aggregate sand with D_{50} = 0.7 mm), effect of type of test (Present research work – Shake table test, Previous research work – Cyclic simple shear test), effect of surcharge (Discussed in detail in next paragraph). However, the relationship between pore pressure ratio and shear strain for clean sand is independent of relative density, grain size distribution, effect of fabric and method of testing [14]. Hence, two of the above reasons (i.e. nature of soil specimen and effect of type of test) do not have any significant influence on the plot of pore pressure ratio vs. cyclic shear strain. (Fig. 9)

Now, as far as effect of vertical effective stress (surcharge) is considered, it should be noted that the magnitude of effective stress in the present research work is set to 4.57 kPa to 4.68 kPa (for D_r varying from 30% to 60% respectively) as against 13 kPa, 25 kPa and 100 kPa used in the previous research work. Hence, it may be inferred that the difference in test results obtained in the present research work and previous investigation (Fig. 9) are mainly due to the significant difference in the vertical effective stresses. Thus, higher pore pressures are generated at lower values of threshold shear strain [10]. This effect of vertical effective stress on the magnitude of threshold shear strain is presented through Table 3.

Table 3. Magnitude of threshold shear strain for different effective vertical stresses

Sr. No.	Vertical effective stress	(γ_t)
1	4.57 kPa to 4.68 kPa (Present Study)	10^{-4} to 10^{-3} %
2	13 kPa (In –situ measurement by Chang (2002)	0.005%
3	25 kPa (Lab. Measurement [Hazirbaba and Rathje (2004)	≈ 0.01%
4	100 kPa (Laboratory Measurement [Hazirbaba and Rathje (2004)	0.01%

It is clear from the above table that effect of vertical effective stress is significant as far as the strain criterion is considered. To sum up, the effect of different parameters such as relative density, acceleration level on initiation of liquefaction using shake table adopting strain approach is studied in detail in the present research work. The conclusions which are drawn from the above discussion are highlighted in the next section.

6. Conclusions

Based on the experimental work carried out in the present study, the following salient conclusions could be drawn:

1. For all the tests conducted in this work, the pore water pressure initially increases with the time and then either remains constant or decreases after attaining a peak. This stage represents initiation of liquefaction.

2. From variation of pore pressure with time at all the three transducers, it is observed for all the tests that pore water pressure first develops at the top of the sample and then proceeds to middle and eventually to the bottom. Thus, liquefaction progresses in the downward direction in the laboratory shake table apparatus also which agrees well with earlier work carried out with other laboratory tests.

3. As the acceleration level increases, the peak pore pressure also increases significantly. Further, this increase in peak pore pressure occurs within a very short duration for high level of acceleration.

4. Existence of threshold shear strain is observed in the present study below which there is no generation of pore pressure.

5. The magnitudes of threshold shear strain (γ_t) obtained from the present investigation are of the order of 10^{-4} % to 10^{-3} % for all the relative densities (varying from 30% to 60%). These values of γ_t are lower than the earlier reported values which is the effect of vertical effective stress.

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6. It is observed in the present study that the magnitude of shear strain at initiation of liquefaction (γ_L) is around 0.01% for all the relative densities (varying from 30% to 60%) and acceleration levels (varying from 0.21g to 0.45g).

7. It could also be deduced from the test results that for pore pressure ratio less than one, initiation of liquefaction is observed for lower level of acceleration whereas for higher level of acceleration liquefaction initiates when the pore pressure ratio exceeds one.

8. As values of threshold shear strain (γ_t) and shear strain at initiation of liquefaction (γ_L) do not affect significantly by the parameter of relative density, from the present study it can be concluded that strain approach for finding initiation of liquefaction is preferable for any state of soil.

9. It is noticed from the present investigation that vertical effective stresses significantly affect the pore pressure build-up and threshold shear strain. Hence, it can be said that strain approach is mainly influenced by the vertical effective stress. Thus, the liquefaction potential at a site can be effectively assessed using strain criterion by simulating field conditions in laboratory shake table test.

7. References

[1] D. Erten, M.H. Maher, "Cyclic undrained behaviour of silty sand", *Soil Dynamics and Earthquake Engineering*, (1995b), 14:115-123.
[2] H. Tsuchida, "Prediction & countermeasure against the liquefaction in sand deposits", Abstract of the seminar in the port and harbour research institute, (1970), 3.1-3.33.
[3] H.B. Seed, K. Mori and C.K. Chan, "Influence of seismic history on the liquefaction characteristics of sands", Earthquake Engineering Research Center, University of California, Berkeley, (1975 b), Report No. EERC 75-25
[4] H.B. Seed, "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes", *Journal of the Geotechnical Engineering Division (ASCE)*, (1979), 105:201-255.
[5] H.P. Singh, B.K. Maheshwari and S. Saran, "Liquefaction behaviour of Solani sand using small shake table", The 12th International Conference of International Association for Computer Methods and Advances in Geomechanics (IACMAG), (2008), 2797-2803.
[6] IS 2720 (Part III/section 1): 1980, Indian Standard, Methods of Test For Soils – Determination of Specific Gravity (First Revision), Bureau of Indian Standards, New Delhi, 1980.
[7] IS 2720 (Part IV): 1985, Indian Standard, Methods of Test For Soils – Grain Size Analysis (Second Revision), Bureau of Indian Standards, New Delhi, 1986.
[8] IS 2720 (Part XIV): 1983, Indian Standard, Methods of Test For Soils – Determination of Density Index (Relative Density) of Cohesionless Soils (First Revision), Bureau of Indian Standards, New Delhi, 1984.
[9] J.P. Mulilis, H.B. Seed, C.K. Chan, J.K. Mitchell, K. Arulanadan, "Effects of sample preparation on sand liquefaction", *Journal of the Geotechnical Engineering Division (ASCE)*, (1977), 103:91-108.
[10] K. Hazirbaba, E.M. Rathje, "A comparison between in situ and laboratory measurements of Pore water pressure generation", 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, (2004), Paper No. 1220.
[11] M.L. Silver, T.K. Park, "Liquefaction potential evaluation from strain controlled properties tests on sands", *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, (1976), 16:51- 65.
[12] M. Derakhshandi, E.M. Rathje, K. Hazirbaba, S.M. Mirhosseini, "The effect of plastic fines on the pore pressure generation characteristics of saturated sands", *Soil Dynamics and Earthquake Engineering*, (2008), 28:376–386.
[13] R.S. Ladd, "Specimen preparation and cyclic stability of sands", *Journal of the Geotechnical Engineering Division (ASCE)*, (1977), 103:535-547.
[14] R.S. Ladd, R. Dobry, F.Y. Yokel, R.M. Chung, "Pore water pressure build-up in clean sands because of cyclic straining", *American Society for Testing of Materials*, (1989), 12:77-86.
[15] S.R. Pathak, R.S. Dalvi, A.D. Katdare, "Earthquake induced liquefaction using shake table test", 5th International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics, San Diego, California, (2010), Paper No. 4.16 a.
[16] T. Iwasaki, "Soil liquefaction studies in Japan: State of the Art", *Soil Dynamics and Earthquake Engineering*, (1986), 5:2-68.
[17] T.G. Sitharam, L. Govindaraju, "Pore pressure generation in silty sands during cyclic loading", *Geomechanics and Geoengineering*: An International Journal, (2007), 2:295-306.
[18] W.J. Chang, "Development of an in situ dynamic liquefaction test", Dissertation, University of Texas, (2002).

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