# Parametric Study on Liquefaction of Aleru River Sand

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***Abstract*—The nature and distribution of earthquake damage is strongly influenced by the response of soils to dynamic loading. For the problems those dominated by wave propagation effects, only low levels of strain are induced in the soil. However for the problems those involving the stability of masses of soil, large strains are induced in the soil. The behavior of soils subjected to dynamic loading is essentially governed by the dynamic soil properties such as shear modulus (G) and the damping (D). Further, liquefaction triggered by earthquakes cause major damage to structures resting on loose to medium dense sandy soils. An experimental study is conducted to evaluate the dynamic properties as well as the liquefaction behavior of soil samples collected from the site close to Aleru river belt, Nalgonda District, Andhra Pradesh. From the grain size distribution of the collected soil sample, it is clear that the material contains a substantial amount of medium to fine sand with size ranging from 0.425mm to 1mm indicating poor drainage. Physical modeling is carried out using PC controlled Cyclic/Static Triaxial testing system to evaluate the dynamic properties as well as liquefaction behavior.**

## I. Introduction

The design of geotechnical engineering problems that involve dynamic loading of soils and soil-structure interaction systems are becoming a challenging task for a geotechnical engineer. This design criterion requires the determination of two important parameters, such as the shear modulus and the damping of the soils. The recent developments in the numerical analyses for the nonlinear dynamic responses of grounds due to strong earthquake motions have increased. The most common cause of ground failure during earthquakes is the liquefaction phenomenon which has produced severe damage all over the world. Investigations are carried out to evaluate the liquefaction potential of soil deposits during earthquakes have been the subject of much attention in recent years. In this present paper work, the parameters that affect the dynamic properties and the liquefaction potential of a saturated sand are highlighted.

**Definitions**

***Soil liquefaction*** is a phenomenon that results when saturated sand is subjected to ground vibrations or continued deformation at a constant stress. These vibrations tend to compact the soil and thereby causes a decrease in the volume. The tendency to decrease in volume results in an increase in pore water pressure when an un-drained condition prevails. If the pore water pressures builds up to the point at which it is equal to the overburden pressure, then the effective stress becomes zero, and the sand loses its strength completely, and it develops a liquefied state which can be either due to static or cyclic stress applications.

More technically, liquefaction is imminent when the porewater pressure (u) equals the total overburden stress ($\llcorner_{vo}$). This creates an effective stress state equal to zero ($\llcorner_{vo}' = [\llcorner_{vo} - u] = 0$).

***Initial liquefaction:*** The point which denotes the condition when the residual pore water pressure becomes equal to the applied confining pressure on completion of any full stress cycle during the course of cyclic stress applications is termed as Initial liquefaction. The development of initial liquefaction has no implications concerning to the magnitude of the deformations which the soil undergoes subsequently, but however, it defines a condition which is a useful basis for assessing various possible forms of subsequent soil behavior.

***Complete Liquefaction:*** The condition when the soil mass undergoes complete deformation and loses its strength completely, further giving no chance of neither deformation or development of strains anymore. This condition is known as Complete Liquefaction.

***Cyclic mobility or Cyclic Liquefaction:*** During the cyclic stress applications, when a condition of initial liquefaction develops and further subsequent cyclic stress applications cause limited strains to develop either because of the remaining resistance of the soil to deformation or because the soil dilates, at this point the pore water pressure drops, and the soil stabilizes under the applied loads, this condition denotes Cyclic mobility or Cyclic liquefaction.

**Measurement of dynamic soil properties**

Dynamic analyses are to evaluate the response of the earth structures to dynamic stress applications, such as those produced by earthquakes, blasting, wind loading or machine vibrations, and are found to have increased applications in civil engineering practice. Precise measurement of dynamic soil properties is somewhat a difficult task in the solution of geotechnical earthquake engineering problems. Several laboratory and field oriented techniques are available to measure the dynamic properties and choice of a particular technique depends on the specific problem to be solved.

**Methods to evaluate dynamic properties of soil (*Laboratory testing*)**:

Many experimental methods have been developed from time to time. The laboratory methods have been determined with small samples and the level of displacement being different. However, they have the advantages of controlled testing and being economical.

These tests are Low-strain tests; High-strain tests; Field testing; Low-strain field tests; High-strain field tests.

**Factors controlling liquefaction**

Many factors that govern the liquefaction for in situ soil are intensity of earthquake and its duration, location of ground water table, soil type, soil relative density, particle size gradation, particle shape, depositional environment of soil, soil drainage conditions, confining pressures, aging and cementation of the soil deposits, historical environment of the soil deposit and building/additional loads on these deposits. In summary, the site conditions and the soil type which are most susceptible to liquefaction are:

*Site conditions*: The site that is close to the epicenter of fault rupture of a major earthquake. And the site that has ground water table close to ground surface.

*Soil type most susceptible to liquefaction for given site conditions*: Sand which has uniform gradation and rounded particles, very loose density state, recently deposited with no cementation between soil grains, and no prior preloading or seismic shaking.

**Methods to evaluate liquefaction potential of soil**

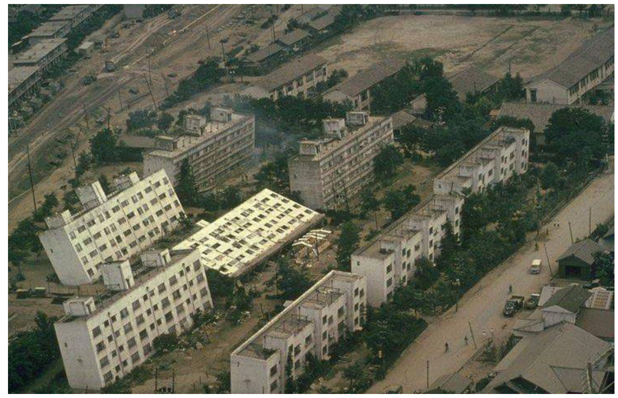
To evaluate the potential for liquefaction, many approaches are developed. The commonly employed methods are cyclic stress approach and cyclic strain approach to characterize the liquefaction resistance of soils both by laboratory and field tests. The cyclic stress approach in evaluating liquefactioen potential characterizes both earthquake loading and the soil liquefaction resistance in terms of cyclic stresses. But, in the cyclic strain approach, earthquake loading and liquefaction resistance are characterized by cyclic strains. The common laboratory tests are cyclic triaxial test, cyclic simple shear test and cyclic torsional shear test. Further, some of the in situ tests to characterize the liquefaction resistance are Standard Penetration test, Cone Penetration test, Shear wave velocity method and Dilatometer test.

The cyclic stress and cyclic strain approaches are most widely used in the field of geotechnical earthquake engineering, and some other approaches such as energy dissipation, effective stress based response analysis and the probabilistic approaches were also developed.

## II. LITERATURE REVIEW

**General**

Many experimental investigations were carried out earlier in the past and still being continued to study the characteristics and behavior of saturated sands when they are subjected to cyclic loads both by laboratory tests as well as field tests. Many failures of earth structures, slopes and foundations on saturated sands all over the world were attributed in the literature to liquefaction of the sands. The well known cases of foundation failures due to liquefaction are those that occurred during the 1964 earthquake in Nigata, Japan (Kishida, 1966). Classical examples of liquefaction are the flow slides that have occurred in the



Nigata, Japan, 1964

*province of Zealand in Holland (Geuze, 1948; Koppejan, et al. 1948) and in the point bar deposits along the Mississippi river (Waterways experiment station, 1967). The failures of Fort Peck Dam in Montana in 1938 (Casagrande, 1965; Corps of Engineers, 1939; Middlebrooks, 1942), the Calaveras Dam in California in 1920 (Hazen, 1920) and the Lower Lan Norman Dam during the 1971 San Fernando Earthquake (Seed et al., 1975) in California provide typical examples of liquefaction failures of hydraulic-fill dams.* Liquefaction often appears in the form of sand fountains, and a large number of such fountains have been observed during Dhubri Earthquake in Assam in 1930 and Bihar Earthquake in 1934 (Housner, 1958: Dunn et al., 1939). When soil fails in this manner, a structure resting on it will simply sinks into it. The most recent Koyna earthquake of 1995 is an illustration of liquefaction phenomenon causing catastrophic damages to structures and resulting in loss of life and property.

Lee and Seed, 1967; Seed and Lee, 1966; had conducted a laboratory undrained cyclic tests (triaxial, direct simple shear and gyratory shear) on saturated sands, cyclic mobility has been observed to develop and to result in large strains. It is controversial whether cyclic mobility occurs in dilative sands in situ during earthquakes to the same extreme degree as has been observed in the laboratory. A simple means for understanding the difference between liquefaction and cyclic mobility as observed in the laboratory is through the use of the state diagram, which is shown in Fig.1 (Castro and Poulos, 1976). The axes are voids ratio and effective minor principal stress. The steady state line represents the locus of states in which a soil can flow at constant effective minor principal stress $\sigma_3$ and constant shear stress. The void ratio at the steady state is the same as the critical void ratio.

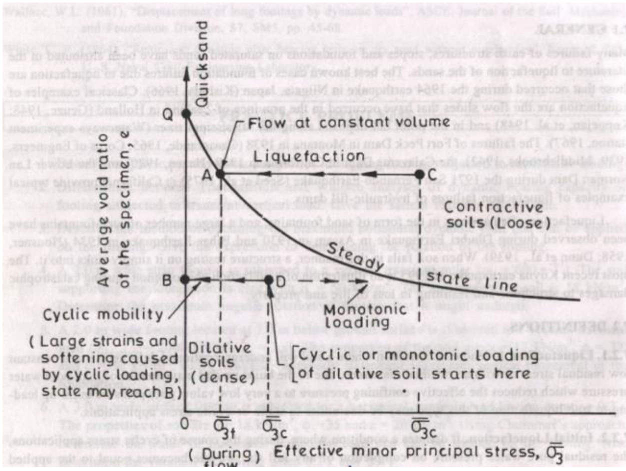



Fig.1: Undrained tests on fully saturated sands depicted on state diagram (Castro and Poulos, 1976)

Liquefaction is the result of undrained failure of a fully saturated, highly contractive (loose) sand, for example starting at Point C and ending with steady state flow at constant volume and constant $\sigma_3'$ at Point A. During undrained flow, the soil remains at Point A in the state diagram.

The quicksand condition that is so familiar through the use of quicksand devices for instruction in soil mechanics is depicted by points on the zero effective stress axes at void ratios above Q. In this state, sand has zero strength and is also neither dilative nor contractive. At void ratios above Q the sand grains are not in close contact at all times.

The mechanics of cyclic mobility also can be illustrated with the aid of the above (Fig.1). Consider first the behavior in (Fig.1) when a fully saturated dilative sand starting, for example, at Point S is loaded monotonically (statically) in the undrained condition. In that case the point on the state diagram may move slightly to the left of Point D but then it will move horizontally towards the steady state line as load is applied. If one now starts a new test at Point D, but this time applies cyclic loading, one can follow the behavior by plotting the average void ratio and the effective stress each time the applied cyclic load passes through zero. In this case the state point moves horizontally to the left, because the average void ratio is held constant and the pore water pressure rises due to cyclic loading.

The magnitude of pore pressure build up in the cyclic test will depend on the magnitude of the cyclic load, the number of cycles, the type of test, and the soil type, to name a few variables. In particular, it has been observed in the laboratory that in triaxial tests for which the hydrostatic stress condition is passed during cycling, and if a large enough number of cycles of sufficient size are applied, the state point for the average conditions in the specimen eventually reaches zero effective stress at Point B each time the hydrostatic stress state is reached. Subsequent application of undrained monotonic loading moves the state point to the right toward the steady state line, and the resistance of the specimen increases.

During cycling in the test described above, strains develop and the specimen becomes softer. If these strains are large enough, one can say that the specimen has developed cyclic mobility. Adequate evidence has been presented to show that most of the strains measured in cyclic load tests in the laboratory are due to internal redistribution of void ratio in the laboratory specimens. For example, at the completion of such tests the void ratio at the top of the specimen is much higher than at the bottom (Castro, 1969). Thus the horizontal line D-B in (Fig.1) is fictitious in the sense that it represents average conditions. Near the top of the specimen, the void ratio increases, and near the bottom the void ratio decreases. The pore pressures that build up and the strains measured in the laboratory are due to the formation of such loose zones (Castro and Poulos, 1976).

In summary, specimens that lie above the steady state line on (Fig.1) can liquefy if the load applied is large enough. Such liquefaction can be triggered by monotonic or cyclic undrained loading. Further to the right of the steady state line that the starting point is, the greater will be the deformation associated with the liquefaction. If the initial point is above Q, the strength after liquefaction will be zero. If the starting point is below Q, the strength after liquefaction will be small but finite. Saturated sands starting at points on or below the steady state line will be dilative during undrained monotonic loading in the triaxial cell and the state point will move to the right. If cyclically loaded the state points will shift to the left as strains occur and the specimen softens. If enough cycles are applied, if they are large enough, and if the hydrostatic stress condition are passed during each cycle, then the zero effective stress condition (i.e. initial liquefaction) can ultimately be reached in the laboratory.

**Mechanism of Liquefaction**

The strength of sand is primarily due to its internal friction. In saturated state it can be expressed as (Fig.2)

$$S = \bar{\sigma}_n \tan\phi \qquad \text{……….} (1)$$

where, S = Shear strength of sand

$\bar{\sigma}_n$ = Effective normal pressure on any plane xx at depth

Z = $\gamma$ $h_w$ + $\gamma_{sub}$ (Z-$h_w$)

$\phi$ = Angle of internal friction

$\gamma$ = Unit weight of soil above water table

$\gamma_{sub}$ = Submerged unit weight of soil

When a saturated sand is subjected to ground vibrations, it usually tends to compact and decrease in volume, if drainage is restrained the tendency to decrease in volume results in an increase in pore pressure. The strength of sand is expressed as,

$$S_{dyn} = (\bar{\sigma}_n - u_{dyn}) \tan\phi_{dyn} \qquad \text{...….. (2)}$$

where,

$S_{dyn}$ = Shear strength of soil under vibrations

$u_{dyn}$ = Excess pore water pressure due to ground vibrations

$\phi_{dyn}$ = Angle of internal friction under dynamic conditions

It is observed that with the development of additional positive pore pressure, the strength of sand is reduced. In sands, $\phi_{dyn}$ is almost equal to $\phi$, i.e. angle of internal friction in static conditions.

When complete strength is lost then $S_{dyn}$ is zero.

Thus, $\overline{\sigma}_n - u_{dyn} = 0$

or $\overline{\sigma}_n = u_{dyn}$

or $u_{dyn} / \overline{\sigma}_n = 1$ ……….(3)

Expressing $u_{dyn}$ in terms of rise in water head, $h_w$ and $\gamma_{sub}$ as $(G-1/1+e)\ \gamma_w$, the Eq. (3) is expressed as $(\gamma_w\ .\ h_w) / (G-1/1+e)\ \gamma_w\ .\ Z = 1$

or $h_w / Z = (G-1/1+e) = i_{cr}$ ………(4)

where, G = Specific gravity of soil particles

e = void ratio

$i_{cr}$ = Critical hydraulic gradient.

It can be noted that, as the pore water pressure increases the effective stress reduces, thus resulting in loss of strength. Thus, if this transfer is complete, then there is complete loss of strength, resulting in what is known as complete liquefaction.

**Case Study Referred**

Seed and Lee (1966) had reported the first set of comprehensive data on liquefaction characteristics of sand performed by dynamic triaxial test. They have performed several undrained triaxial tests on Sacramento River sand ($e_{max}$ = 1.03, $e_{min}$ = 0.61). The grain size ranged between 0.149 mm and 0.297 mm. The results of a typical test in loose sand (e = 0.87, $D_R$ = 38%) are shown in Fig.3. In this test the initial all-around pressure and the initial pore water pressure were 196kN/m$^2$ and 98kN/m$^2$ respectively, thus giving the value of effective confining pressure as 98kN/m$^2$. The cyclic deviator stress $\sigma_d$ of magnitude 38.2 kN/m$^2$ was applied with a frequency of 2 cps. The test data in Fig.2 shows the variation of load, deformation and pore-water pressure with time.

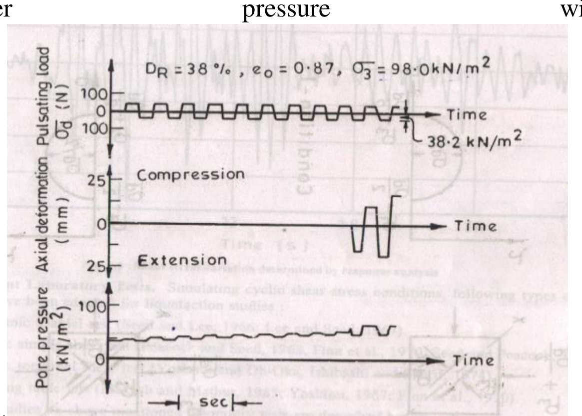



Fig.2: Typical pulsating load test on loose saturated Sacramento River sand (Seed and Lee, 1966)

Another case study which is being referred in this present work was performed by Prof. T.G.Sitharam, Professor, Department of Civil Engineering, Indian Institute of Science, Bangalore. An experimental program was carried out to evaluate the dynamic properties as well as the liquefaction behavior of soil samples collected exclusively from the sites close to epicenter of recent Bhuj earthquake, sites close to Sabarmati river belt in Ahmedabad and Meizoseismal region of Shillong Plateau, Assam. All these tests were performed on Cyclic Triaxial Testing Equipment.

They constituted specimens of size 50mm diameter and 100mm height of liquefied sands collected from the site and subjected to cyclic loads in undrained condition with a frequency of 1Hz (sine wave) under an effective confining pressure of 100 kPa.

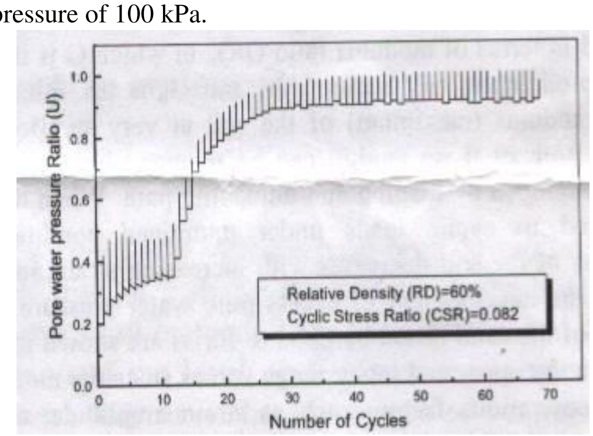



Fig.3: Response of Pore water pressure to cyclic load

From the above figure which represents the variation of sample pore water pressure due to dynamic load. When paying attention to the result, in continued applications of the cyclic load with a cyclic stress ratio (CSR = $\sigma_{dc} / 2\overline{\sigma}_c$, $\sigma_{dc}$ = cyclic stress, $\overline{\sigma}_c$ = effective confining pressure) of 0.082, the excess pore water pressure increased gradually, and it reached a value equal to the initially applied hydrostatic pressure in 45 cycles of uniform load applications (at which pore water pressure ratio U = $u_{max}/\overline{\sigma}_c$ = 1, where $u_{max}$ = excess pore water pressure during each cycle). Corresponding to this pore water pressure build up, there was a drastic reduction of effective stresses in the soil and the stage was regarded as an ultimate stage of Liquefaction.

The resistance to liquefaction is quantified in terms of cyclic stress ratio or it is referred as the cyclic strength. They found that the cyclic resistance to liquefaction tends to increase in proportion to the relative density at which the sample is prepared

The degradation of shear modulus is expressed in terms of modulus ratio $G/G_o$ in which G is the secant shear modulus at any strain level and $G_o$ is the initial secant shear modulus (maximum) of the soil at very small levels of strain. Both of these moduli were obtained by the hysteresis loop developed by loading and unloading path. When the soil was subjected to cyclic loads under undrained conditions, due to the development of excess pore water pressures, the modulus of the soil decreased with increasing strain amplitudes.

From their results, they concluded that the soils exhibit nonlinear, inelastic stress-strain behavior under dynamic loading conditions. The stiffness of a soil is greatest at low strain levels and however, at higher strain levels, the effects of nonlinearity and inelasticity increase, producing lower stiffness in the soils. The cyclic stress approach characterizes soil liquefaction resistance in terms of cyclic stresses

## III. Experimental Investigation

The primary objective of this study is to assess the affect of parameters like confining pressure, relative density, size of the particle, applied cyclic stress and number of cycles on the liquefaction potential of the saturated sands in undrained conditions when they are subjected to cyclic/dynamic loading.

Factors effecting cyclic response of soils

The cyclic strength is influenced by various factors, such as number of load cycles, axial strain cycles, and magnitude of applied cyclic shear stress; excess pore water pressure, state of effective stress density, stress history, particulate structures, age of soil deposit, specimen preparation procedure, frequency, its limit and shape of the cyclic wave. More often the influence of No. of loading cycles on strength, modulus of elasticity, Poisson's ratio, total elastic and plastic strain, pore water pressure variation, shear and back modulus and damping properties are studied in compression and extension. For the design of pavements in addition to other elastic properties modulus of resilient deformation and fracture energy of sub-base/stabilized soils may be determined to assess the remaining life of the pavement.

Use of Cyclic test data

The results of cyclic triaxial test when suitably processed and interpreted are used in solving/assessing the following problems.

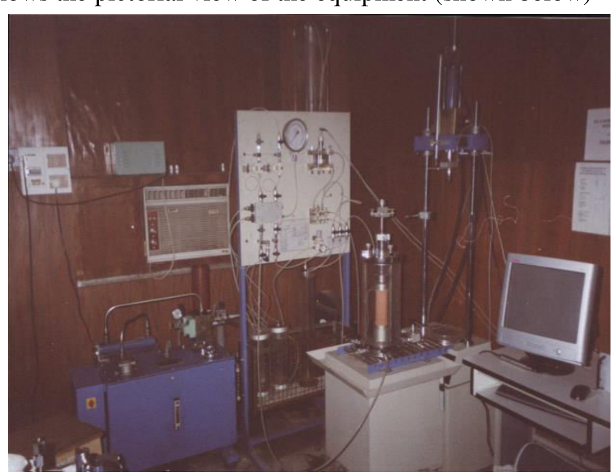
1) Liquefaction potential of foundation soils.
2) Assessment of improved ground
3) Site amplification studies
4) Seismic slope stability
5) Stability of rail track or road pavement sub-grades
6) Effects of blasting on foundation soils
7) Influence of construction vibrations
8) *Performance foundation soil of off shore structure*.

In short, the cyclic triaxial test results are relevant and directly useful in wave propagation analysis, liquefaction hazard assessment and in cyclic non-linear constitutive models applicable to soils.

In this present work, investigations are carried out on Aleru river sand. From the grain size distribution curve (Fig.5) of the selected sand, the material contains a substantial amount of medium to fine sand with appreciable quantity of fines. The soil profile indicates that the sandy layer is sandwiched between silty clay depositions indicating undrained conditions. Therefore, medium graded sand (1.0mm to 0.425mm) was selected and specimens of size 50mm diameter and 100mm height with corresponding relative densities of 30% and 70% are constituted and are subjected to cyclic loads in undrained condition with a frequency of 1 Hz (sine wave) and peak pulsating stress of 0.5 kg/cm$^2$ for single amplitude and 1.0 kg/cm$^2$ for double amplitude under an effective confining pressures of 1.0, and 2.0 kg/cm$^2$ respectively for each relative density.

Description of the Equipment

The state-of-the-art cyclic triaxial testing facility has been installed in Geotechnical Engineering division of Osmania University, Hyderabad. The equipment is completely automated and computerized. It consists of servo-controlled submersible load cell that has a capacity of 10 kN. The load frame is fitted with a hydraulic actuator with frequency range of 0.01 Hz to 10 Hz. The triaxial cell has the facility to conduct the tests on soil samples of sizes 38mm, 50mm and 75mm diameter with confining pressures up to 10 kg/cm$^2$ using pneumatic control panel. Both stress-controlled and strain-controlled tests can be performed using built in sine, triangular and square waveforms or any other desired loading waveform by means of external input. The axial deformation, lateral deformation, volume change, cell pressure, cyclic load and sample pore water pressure can be monitored using a built-in data acquisition system. Fig. 4 shows the pictorial view of the equipment (shown below)



**Testing Methodology**

To accomplish with the concerned study stated above, the following methodology is adopted:

**I.** Evaluation of Specific Gravity by Pycnometer method and Grain Size distribution using Sieve Analysis for naturally available sand sample.

**II.** Study of Density Characteristics using Sand Replacement method.

**III.** Study of Liquefaction according to ASTM 3999 and

ASTM 5311 using cyclic triaxial testing system.

**Table-1: Density Calculations**

| Type of Gradation | Type of state | Density (g/cc) | Relative Density (%) | Density corresponding to RD (g/cc) | Weight of the sample (gms) |
|---|---|---|---|---|---|
| Medium Sand | Natural | 1.512 | 30% | 1.5405 | 303.23 |
| | Loose | 1.477 | | | |
| | Dense | 1.705 | 70% | 1.6345 | 320.86 |

***Sieve Analysis***: For the above specified soil sample, sieve analysis is performed and it is shown in the Fig.3.

According to IS 456-1978 the test sample comes under Grade-1 sand.

Based on gradation curve the effective size of the particle ($D_{10}$) is 0.5mm.

Uniformity Coefficient, $C_u = 2.25$

Coefficient of Curvature, $C_c$=1.03

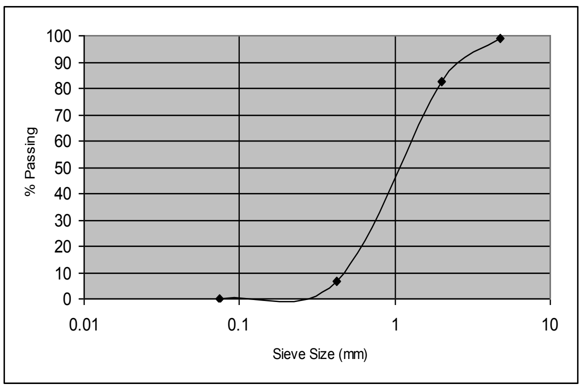


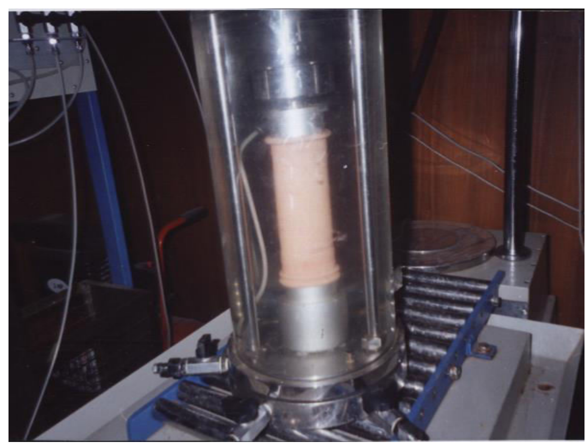

Fig.5 Particle Size Distribution Curve

**Table-2: Index Properties of soil sample**

| | |
|---|---|
| Specific Gravity, G | 2.63 |
| Dry Density, $_d$ (g/cc) | 1.66 |
| Bulk Density, (g/cc) | 2.905 |
| Uniform Coefficient, $C_u$ | 2.25 |
| Coefficient of curvature, $C_c$ | 1.03 |
| Void ratio, e | 0.58 |
| Porosity, n | 36.89 |

## Sample Preparation

Many of the water sedimentation depositional methods tend to produce non-homogeneous specimens with the coarser fraction on the bottom and the finer fraction on the top of the specimen. Cylindrical soil specimens of size 50mm diameter and 100mm height are prepared by placing the dry sand in a funnel with a tube attached to the spout. The tube is placed at the bottom of the membrane lined split mould. The tube is slowly raised along the axis of the symmetry of the specimen, so that there is no drop in height of the soil specimen. This procedure is used to achieve the loosest possible density for a specimen prepared in a dry state. While preparing the soil specimens at relatively higher densities, the mould is gently tapped in a symmetrical pattern until the desired density is achieved. Using the above technique, soil specimens with two different target initial relative densities ($D_R$) of 30% and 70% are prepared. In preparation of soil specimens of 30% $D_R$, the soil is placed in five layers and each layer being saturated simultaneously, in order to avoid the settlement of the top layer of the specimen because of its loosest density when it is saturated in complete dry state. Whereas in preparation of soil specimens of 70% $D_R$, the soil is placed in five layers and each layer being compacted in order to achieve required density and accumulation of the soil in the mould. After the preparation of the specimens, a vacuum pressure of 60 kPa is applied to the specimens in order to reduce disturbance during the removal of split mould. The specimens are then saturated with de-aired water through gravity. Saturation of the specimens is checked by measuring Skempton's pore pressure parameter B. Following the saturation, the specimens are then isotropically consolidated to the required confining pressure**.**



Pictorial view of sample after preparation and before testing

**Dynamic properties Assessment**

Stress-controlled cyclic triaxial test are carried out on isotropically consolidated soil specimens under undrained conditions to simulate essentially undrained field conditions during earthquake. Cyclic loading is applied on the specimens using hydraulic actuator. The test are conducted with a frequency of 1Hz with sinusoidal wave along with peak pulsating stress of 0.5 kg/cm$^2$ as single amplitude and 1.0 kg/cm$^2$ as double amplitude under an effective confining pressures of 1.0 kg/cm$^2$ and 2.0 kg/cm$^2$ respectively. The axial deformation, cell pressure, cyclic load and pore water pressure are monitored using a built-in data acquisition system.

Evaluation of dynamic properties of soil

Data calculation: when cyclic triaxial test are performed on soil specimen, a hysteresis loop is formed in the plot of deviator stress, $\sigma_d$, versus axial strain, ε. The slope of the secant line connecting the extreme points on the hysteresis loop is the dynamic Young's modulus, E, which is given by

$$E = \sigma_d / \varepsilon$$

Further,

$$\gamma = (1+\upsilon)\varepsilon \text{ and } G = E/2(1+\upsilon),$$

Where G is the shear modulus, γ is the shear strain and υ is the Poisson's ratio that may be taken as 0.5 for saturated undrained specimens. The damping ratio, D, is a measure of dissipated energy versus elastic strain energy, and may be computed from the equation

$$D = (1/4\pi)*(A_L/A_T),$$

Where $A_L$ = area enclosed by the hysteresis loop; and $A_T$ = area of the triangle.

**Table-3: Variation of all Parameters on specimens of different relative densities:**

| Relative density (g/cc) | **30%** | | **70%** | |
|---|---|---|---|---|
| Confining Pressure (kg/cm$^2$) | 1 | 2 | 1 | 2 |
| Excess Pore Pressure (kg/cm$^2$) | 0.11 | 0.30 | 0.16 | 0.42 |
| Cyclic Pore Pressure Ratio | 0.48 | 1.39 | 0.77 | 2.63 |
| Deviator Stress (kg/cm$^2$) | 0.94 | 0.94 | 0.94 | 0.95 |
| Time (secs) | 223.19 | 493.78 | 806.28 | 858.41 |
| Axial Strain (%) | 20.85 | 25.77 | 22.48 | 23.67 |
| Double Amplitude Axial Strain (%) | 1.44 | 1.34 | 1.72 | 0.74 |
| Cycle Number | 225 | 500 | 810 | 900 |
| Load (kg) | 18.50 | 18.40 | 18.50 | 18.60 |
| Deformation (cm) | 2.085 | 2.57 | 2.25 | 2.36 |
| Mean Effective Stress (kg/cm$^2$) | 0.49 | 0.60 | 0.48 | 0.59 |

**Table-4: Variation of number of cycles causing initial and complete liquefaction for different relative densities @ varying confining pressures:**

| Relative Density (%) | Confining pressure (kg/cm$^2$) | No. of cycles for initial liquefaction | No. of cycles for complete liquefaction |
|---|---|---|---|
| | 1.0 | 162 | 180 |
| 30 % | 1.5 | 185 | 205 |
| | 2.0 | 236 | 270 |
| | 1.0 | 324 | 390 |
| 70 % | 1.5 | 345 | 418 |
| | 2.0 | 402 | 480 |

For the considered peak pulsating stress, the number of cycles at which initial liquefaction occurred i.e.. When excess pore pressure ratio $U = u_{max}/\sigma_c = 1.0$.

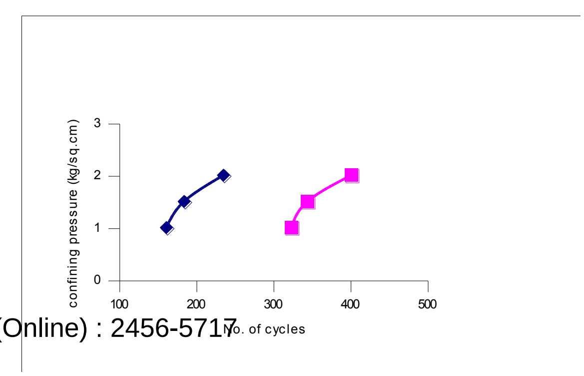



Fig. 6: Variation of no. of cycles causing initial liquefaction for different relative densities

For the considered peak pulsating stress value, the number of cycles at which axial strain =20% indicating complete liquefaction

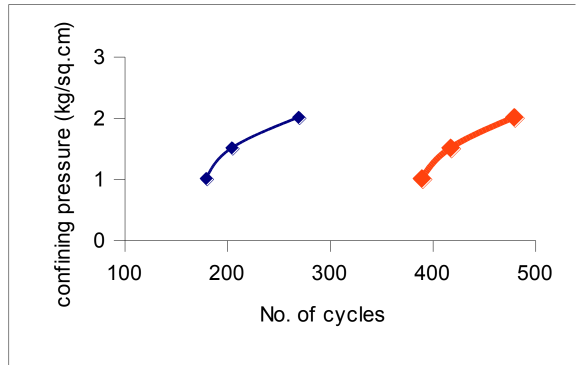
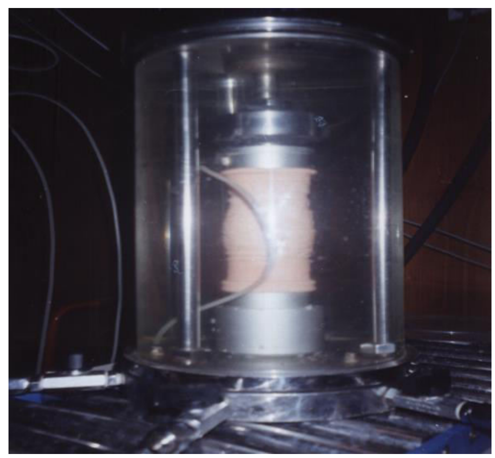



Fig. 7: Variation of no. of cycles causing complete liquefaction for different relative densities



Sample failure for loose state Dr=30%

Sample failure for higher density state Dr=70%

## IV. Conclusions & Discussions

The peak pulsating stress of 0.5 kg/cm$^2$ for single amplitude and 1.0 kg/cm$^2$ for double amplitude are considered based on seismic zone classification according to IS:1893 and in relative to the probable depth of sample collected.

1. The number of cycles required to cause initial liquefaction increased as the effective confining pressure increased for both relative densities.
2. For a given confining pressure, number of cycles required to cause initial liquefaction increased as the relative densities increased.
3. The number of cycles required to cause complete liquefaction are increased as the confining pressure is increased.
4. For confining pressure, number of cycles required to cause complete liquefaction are more for higher relative density.
5. From the sample considered in the investigation, even at higher relative density i.e. $D_R$ = 70%, cyclic mobility is not observed indicating potential for liquefaction.
6. The results re-established the fact that the occurrence of liquefaction in sand with higher relative density is less. Further this study showed that the minimum no. of cycles required to cause liquefaction of Aleru river sand is 162 which may not be the characteristic of the earthquake expected in this region.
7. Hence it was found minimum chances of occurrence of liquefaction in this location.

## *V.* References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## VI. Appendix

Many trials have been performed in the process of getting introduced to the experimental procedure of the equipment as part of post graduate research study. It took nearly three months to get trained on the system but even then many samples failed due to lack of good membranes, machine breakdown & due to voltage drop. The graphical results were found displayed but not to correct scale and possibility of default pre-programmed software also. The graphical representation were observed to be different from the actual printed copies, hence were not taken into consideration.

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