Physical Modelling of Liquefiable Soils in Micro Shaking Table

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ABSTRACT

The dynamic models are widely used to estimate the soil behavior during earthquake. These models are useful to validate numerical data or to understand failure mechanisms induced in that kind of events. During the quake are generated additional stress and in some cases the extra load can reduce the shear strength of the material. Besides, in saturated sandy soils is generated an excess pore pressure which could induce a failure by liquefaction. The liquefaction can be evaluated through several laboratory methods, but no all techniques represent totally the phenomenon. This research work addresses the design, construction and implementation of a flexible container capable to simulate the liquefaction phenomena by shaking table at 1 g. The equipment has a dimensions of 80 mm x 60 mm x 60 mm, and ensures an undrained condition. Additionally, it was instrumented with a one pore pressure sensor, one displacement transducer, two accelerometers and one piezoelectric actuator. An experimental program was developed, which incorporated three different size particles of Guamo sand. The soil was tested in the device with the purpose to evaluate the liquefaction susceptibility. Moreover, the experimental program also included the application of three fundamental frequencies from representative earthquakes in Colombia at three different amplitudes for a scaling of 1, 50 and 80 g. The experimental results show that the liquefaction susceptibility is higher in the fine sands and the parameter which triggered the phenomena is the system acceleration.

1 INTRODUCTION

Earthquakes is one of the greatest hazard to infrastructure and human life. After to Alaska and Niigata earthquakes, the liquefaction phenomenon is widely studied in geotechnical engineering. Liquefaction is one of the most devastating instabilities in saturated granular materials. Besides, the phenomena may trigger by seepage, rise of water level, traffic action, machine vibration and any cyclic load.

Loose sands are more susceptible to liquefaction. This kind of soil tends to compact under cyclic loading. he decrease in volume causes an increase in pore water pressure which cannot dissipate under undrained conditions (Knappett & Craig 2012). Usually, the liquefaction initiation is measured by the pore pressure excess parameter, r_u. This value is the ratio of increment of pore pressure, Δu, and initial effective stress, p'_0.

Liquefaction resistance can be assessed using field techniques or laboratory procedures. Seed & Idriss (1971) proposed a methodology from SPT results; whereas, Seed et al. (1983) with CPT tests and Zeghal & Elgamal (1994) through Down Hole data. Likewise, liquefaction susceptibility can be evaluated via Simple Shear, Triaxial and Torsional apparatus in laboratory. Some of representative investigations about this topic was done by Been & Jefferies (1985); Alarcon-Guzman et al. (1988); Andrus & Stokoe II (2000) in laboratory.

However, the field and laboratory tests must be complemented to estimate the liquefaction susceptibility. An excellent alternative to complement these tests is the physical modelling. Physical modelling can be performed in centrifuge or shaking table devices. Schofield (1980); Hushmand et al. (1988); Madabhushi & Schofield (1993); Meymand et al. (2000); Brennan et al. (2006); Turan et al. (2009); and Lee et al. (2013) were evaluated the soil dynamic behavior by means of scale models in centrifuge and shaking table apparatus.

This research works is addressed in the design and construction of a flexible container, to evaluate of soil behavior during cyclic loads. The container was build with black silicone and was instrumented to measure displacements, accelerations and pore pressure build up. Furthermore, an experimental program was executed to calibrate and implement the device; while it was estimated the liquefaction potential in sandy soils.

In addition, liquefaction phenomena were assessed. Thereby, was proposed a liquefaction resistance model to estimate the liquefaction resistance in three different size particles of Guamo sand. This model includes the pore pressure measure and the relation between system acceleration with a resistance parameter. In this research work it was found that the apparatus is capable to replicate dynamic conditions, trigger the phenomena and measure the liquefaction initiation.

2 LIQUEFACTION PHYSICAL MODELLING

The geotechnical engineers have made many investigations and analysis to recognize the causes (soil behavior) and the effects (infrastructure damage) of liquefaction phenomenon. In this way, the main progresses are in laboratory tests. However, to understand completely the phenomena is required carry out field procedures. Nonetheless, the field tests need a widespread area and at this moment is not developed an assessment which replicates totally earthquake and liquefaction phenomena in-situ.

Therefore, the constructions of scale models are useful to represent the initial conditions of liquefaction. For earthquake engineering purposes, scale models are usually well instrumented and then tested using shaking tables or centrifuge devices (Kramer & Elgamal 2001). In complex projects a comparison between centrifuge tests with finite elements is the best option to validate the both results.

2.1 Centrifuge modelling (ng)

According to engineering design, it is usual to evaluate geotechnical structures by construction of scaled models. These models can be build and, after, tested in centrifuge equipment to represents the soil behaviour of prototype. The main idea of modelling geotechnical problems in a centrifuge is that it allows the use of a reduced-scale model in which a 1/N scale for length is used together with an acceleration field that is N times earth's gravity (Caicedo et al. 2015). Then, the centrifuge model subjected to an inertial acceleration field of N times Earth's gravity. The vertical stress at depth hm will be identical to that in the corresponding prototype at depth hp where hp=Nhm (Taylor 1995).

Wood (2004) synthesizes the location and develops of geotechnical centrifuge devices around the world. A centrifuge model would represent better (more approximately) most be constructed with the same soil. The gravity is increased by the same geometric factor N relative to the normal earth's gravity field, that is 1g (Madabhushi 2014). Figure 1 describes the scale factor in centrifuge modelling.

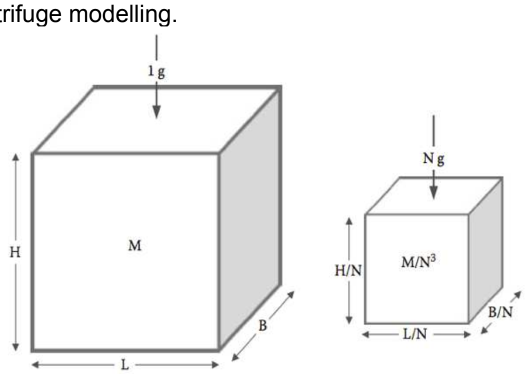

Figure 1. Principle of centrifuge modelling (from Madabhushi 2014).

In principle, the change of Earth's gravity produces an increment of stress state of soil. This increment affects the behavior of the model. To compare the results of centrifuge test model with prototype behavior is necessary taken to account the scaling factors (Table 1).

Table 1. Scaling factors for centrifuge modelling (from Kutter & James 1989).

Type of event	Parameter	Scaling law model/prototype	Units
All events	Stress	1	Nm^{-2}
	Strain	1	-
	Length	1/N	m
	Time (diffuse)	1/N	s
Dynamic	Frequency	N	s^{-1}
	Velocity	1	ms^{-1}
	Acceleration	N	ms^{-2}
	Time	$1/N^2$	s
	Displacement	1/N	m

2.2 Shaking table modelling (1g)

The shaking tables are others apparatus employed to evaluate the soil behaviour through the construction of physical models. This equipment offers the advantage of testing larger soil models (Kramer & Elgamal 2001). The tests made with these devices are often conducted in 1g gravitational field. At present many laboratories, that study damages of earthquakes, have shaking tables capable to produce displacements in 1, 2 or 3 dimensions. The movements are generated by programmable actuators. These actuators can be mechanical, hydraulic, pneumatic, electro-magnetic or piezo-electric; according to the magnitude of stress or displacement required to cause the phenomenon.

In order to simulate the prototype in the model test, the law of similitude must be applied. Initially, the similitude factor "λ" was defined by Iai (1989) and after validated in the simulation of the seismic pile behavior in saturated clay by Meymand (1998). However, to consult the mathematical procedure to compute the similitude factors in the 1g model testing, please, refers to (Lin & Wang 2006). Table 2 presents the similitude factors to compare model and prototype with the same weight density at 1g in shaking table.

Table 2. Similitude factors for shaking table modelling (from Pitilakis et al. 2008)

Parameter	Similitude law model/prototype	Units
Length	$1/\lambda$	m
Strain	1	-
Stress	1	Nm^{-2}
Acceleration	1	ms^{-2}
Time	$1/\lambda^{1/2}$	s
Frequency	$\lambda^{1/2}$	s^{-1}

Thereby, by means of shaking table models it can simulate the application of dynamic loads during earthquakes. Moreover, the implementation of this kind of equipment on centrifuge devices is an excellent option to evaluate the liquefaction phenomena. Taylor (1995) and Madabhushi (2014) describe the procedures and techniques to evaluate dynamics event in soil deposits through the combination of both apparatus.

3 METHODOLOGY

3.1 Equipment

At the physical models laboratory of University of los Andes Serrato (2012) and Correa (2015) designed and built a flexible container to evaluate the soil behaviour on 1g and ng tests. Nevertheless, the first laminar box model had some friction between the laminae and the vertical supports and its performance at low frequencies was low.

However, it was built a container flexible with dimensions of 80 mm x 60 mm x 60 mm. A new flexible container type Equivalent Shear Beam (ESB) was fabricated with black silicone (Figure 2). The design is completely frictionless and has the advantage to permits the free movement of soil inside, due to the low rigidity of the material. Additionally, the material of recipient ensures that at the junctions no water leaks occur. Further, the box has a hole of 10 mm in the bottom covered with a mesh of 0.075 mm (sieve 200) to permit the pore pressure measurement.

Figure 2. ESB silicone container.

Also, the device is capable to generate a cyclic loads and measure the soil response. The apparatus is combination of the flexible container with a dynamic system. Besides, it works as 1D Shaking Table. The movement of system is generated by a Piezo-Electric actuator (CEDRAT APA ML120), through an input signal which is produced by a function generator (PROTEK B8003FD) and amplified by an amplifier (CEDRAT LA75).

Moreover, the shaking table was instrumented with a sensor of 0.5μm type Eddy-Current (με eddy-NCDT 3010) to measure displacement at the bottom of container. The soil response during the dynamic test was registered by two accelerometers (ACC104A OMEGA). The accelerometers were located at the base and the middle of container. Finally, a pore pressure transducer (Honey Well 40PC0156) was connected directly to the bottom hole of the box. This transducer measures the soil pore pressure increment. The data acquisition was done with a Lab VIEW code through the NI 9234 National Instruments card. Figure 15 shows the system instrumentation.

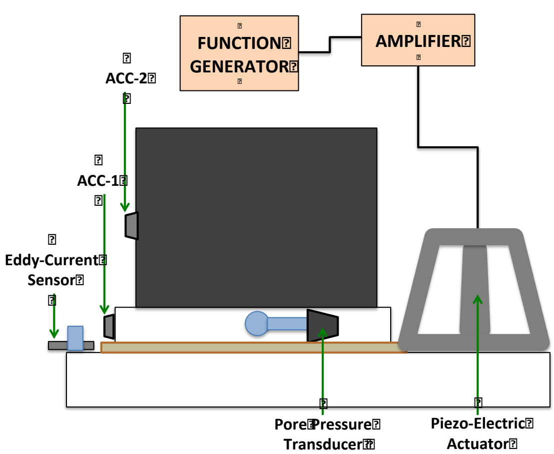

Figure 3. Schematic view of the shaking table.

3.2 Material

The soil chosen to this research work was Guamo Sand. Geologically the Guamo Sand was product of the sequence of volcanic deposits coming to Machín volcano. This soil is from Luisa River located at Tolima department (Colombia). Since the liquefaction hazard depends of geological, compositional and state criteria (Kramer 1996), the material was selected in order to guarantee that soil was cohesionless. Also, the selection was done on basis of the particle size and evaluate by standard procedure ASTM D422 (ASTM International 2007). Three different particle size was taken: coarse (pass T_{10} retain T_{20}), medium (pass T_{20} retain T_{40}) and fine (pass T_{40} retain T_{100}). Figure 3 displays the granulometric curve of the three sizes of sand used for this work.

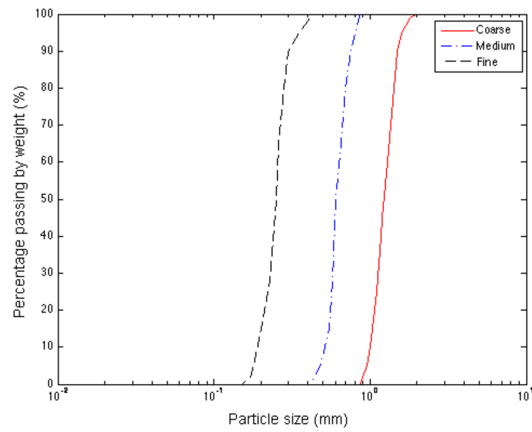

Figure 4. Grain size curve.

Furthermore, physical sand properties were computed. Specific gravity of soils for particles was estimated through the method of ASTM D854 (ASTM International 2014). The maximum and minimum void ratio of the particles sizes were assessed according to the stablished in the INV-E 136 standard (INVIAS 2013). Also, was calculated the D_{50} characteristic of each size particle curve. In Table 3 is presented the physical properties results.

Table 3. Physical parameters of soil.

Parameter	Unit	Coarse sand	Medium sand	Fine sand
Gs	-	2.70	2.70	2.70
e_{max}	-	0.96	0.84	0.99
e_{min}	-	0.85	0.70	0.75
γd_{max}	kN/m^3	14.32	15.60	15.11
γd_{min}	kN/m^3	13.54	14.42	13.34
D_{50}	mm	1.25	0.60	0.25

3.3 Experimental program

The liquefaction susceptibility was evaluated by the development of an experimental program in the shaking table device. Thereby, three representative earthquakes, from Colombia, was selected. These earthquakes were: Armenia (1999), Quetame (2008) and Tumaco (2013). The accelerogram signals were transformed by FFT and taken to frequency domain (see Figure 5). According to Moczo et al. (2014) the fundamental frequency of each earthquake was applied in sinusoidal signals towards the Piezo-Electric actuator and perform the tests.

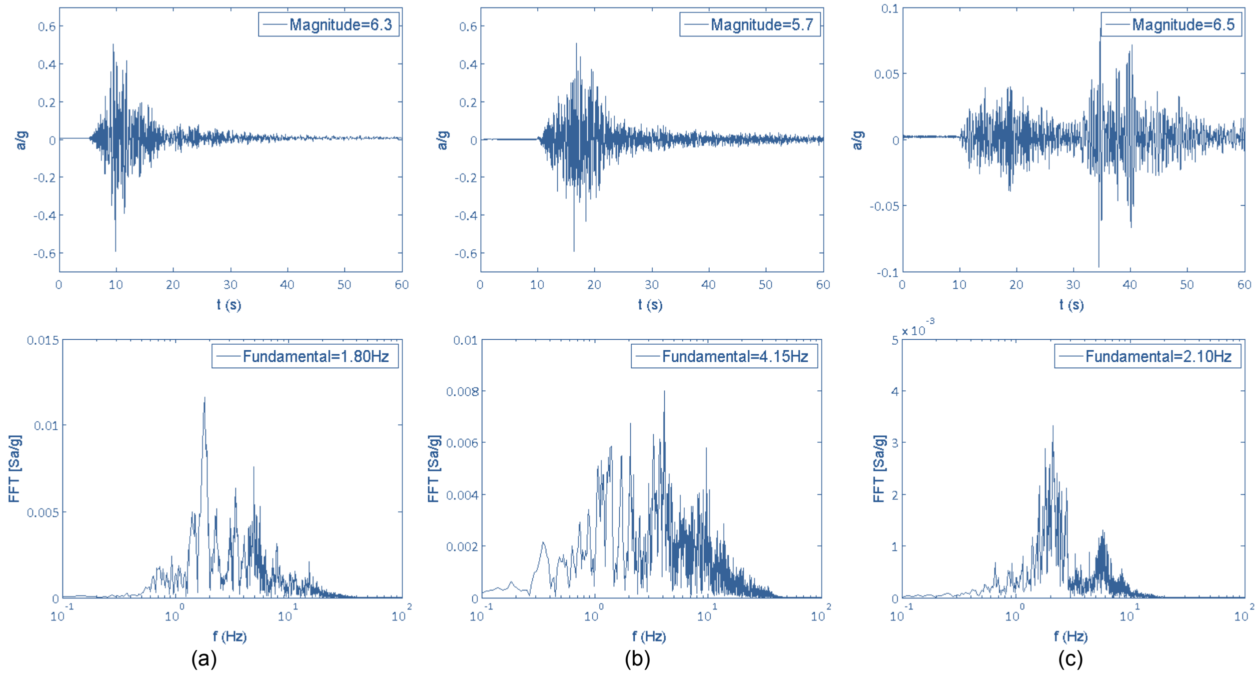

Figure 5. Earthquake signal and Spectra response: (a) Armenia signal; (b) Quetame signal; (c) Tumaco signal

In addition, the input signal was amplified by three different voltages. Nevertheless, to compare the results through shaking table centrifuge tests in future research, the fundamentals frequencies of each earthquake were scaled according to Tables 1 and 2 at 1g, 50g and 80g. Experimental program includes a total of 81 tests in shaking table. In Table 4 presented the experimental program parameters to evaluate the liquefaction phenomena in three sandy soils.

Table 4. Physical parameters of soil.

Parameter	Unit	Values		
		1	2	3
Amplitude	mV	200	500	800
Frequency (1g)	Hz	1.34	1.45	2.04
Frequency (50g)	Hz	90	105	207.5
Frequency (80g)	Hz	144	168	332

4 ANALYSIS AND RESULTS

4.1 Shaking table test

Several test without soil were executed to assess the flexible container displacement. Outcomes registered by the Eddy-Current sensor indicate that the device displacement is not dependent of the frequency applied. However, the frequency controls the system acceleration. According to frequencies values were measured the container response acceleration without soil.

However, it was found that to assess the soil displacement, is necessary to combine of the Eddy-Current sensor and accelerometers measures. The above, due to the position of the sensors. Thereby, the tests were performed with soil to estimate the precision of the instrumentation. Moreover, soil displacements were computed through the Duhamel's Integral (equation 1). This equation describes the response of a linear system, like the 1D shaking table apparatus.

$$u(t) = \frac{1}{m\omega_d} \int_0^t Q(\tau) e^{-\xi\omega(t-\tau)} \sin \omega_d (t-\tau) d\tau \qquad [1]$$

The Duhamel's Integral is very difficult to solve analytically, but it can be integrated numerically. Figure 6 presents the approximation procedure to evaluate the convolution integral by numerical methods. Since the displacement is defined as the second integral of acceleration, equations 2 and 3 explain the computation used to obtain the soil response by the accelerometers in a time instant Δt.

$$\dot{u}: \Delta\dot{u}(\tau) = \ddot{u}(\tau)\Delta t + \Delta\ddot{u}(\tau)\frac{\Delta t}{2} \qquad [2]$$

$$u: \Delta\dot{u}(\tau) = \dot{u}(\tau)\Delta t + \Delta\ddot{u}(\tau)\frac{\Delta t^2}{2} + \Delta\ddot{u}(\tau)\frac{\Delta t^2}{6} \qquad [3]$$

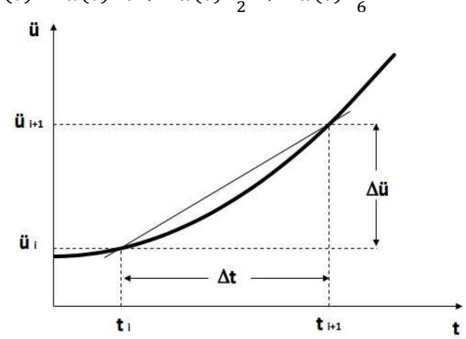

Figure 6. Numerical integration method.

From the accelerometers measurement were computed the shear strain and shear stress of soil. The shear strain was obtained by the comparison between soil displacement at the centre and the container length. Furthermore, the shear stress was found through the motion equation (see equation 4). Turan et al. (2009) affirm that the motion equation integral can be solved using linear interpolation of acceleration. Due to the acceleration was measured in two points of the shaking table device, is possible estimate the shear stress of soil applying equation 5.

$$\tau(z,t) = \int \rho \ddot{u}\, dz \qquad [4]$$

$$\tau_i(t) = \sum_{k=0}^{i-1} \rho \frac{\ddot{u}_k + \ddot{u}_{k+1}}{2} \Delta z_k \qquad [5]$$

Results shows that the displacements obtained via numerical integration are widely close with the registered by the Eddy-Current sensor. In addition, the shear stress plots evidence the soil stiffness variation in the time before the liquefaction occurrence. Figures 7 and 8 present the results of system displacement and shear stress of the soil derivative by the accelerometers.

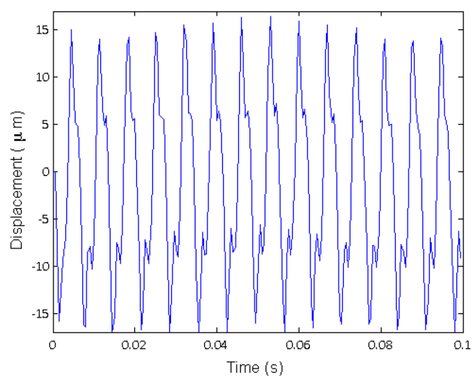

Figure 7. Base displacements.

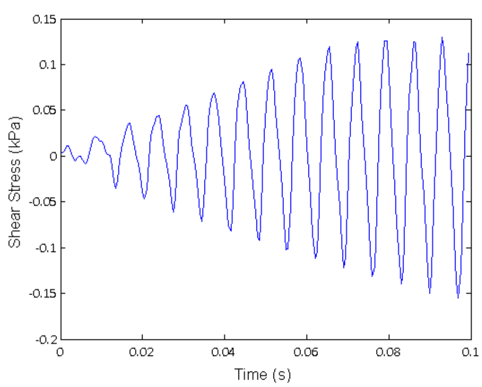

Figure 8. Shear stress.

4.2 Liquefaction test

Specimens of Guamo Sand were prepared by the water deposition method. This method was used to obtain homogeneous samples with the minimum void ratio possible. Likewise, water deposition method was selected to make certain that the sand will be saturated. The preparation procedure is described by Tatsuoka et al. (1986) and Ishihara (1996).

During the tests performing was observed water flow toward the surface and soil settlements. This effect is caused by soil stiffness loss and rearrangement of particles. In Addition, during each test it was detected that the soil experiments a densification process. The liquefaction process is shown in Figure 9.

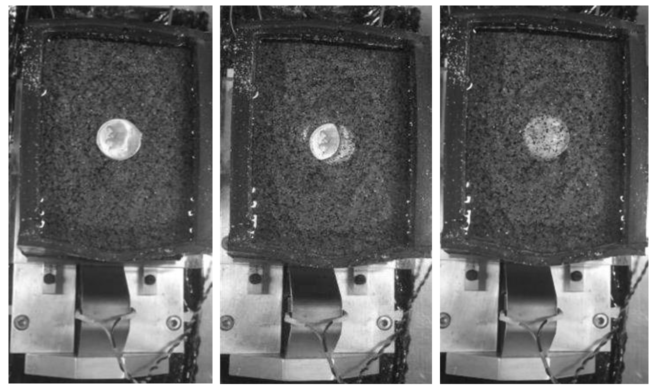
(a) (b) (c)

Figure 9. Soil behaviour during test: (a) Before liquefaction; (b) Initial liquefaction; (c) After Liquefaction.

Besides, the evaluation of liquefaction phenomena initiation was assessed by the measurement of pore pressure increment. However, not at all tests recorded values of r_u near to 1. Thereby, to estimate the instant of liquefaction initiation were compared the results of pore pressure excess with the soil acceleration. Figure 10 shows the soil response during the cyclic loading application.

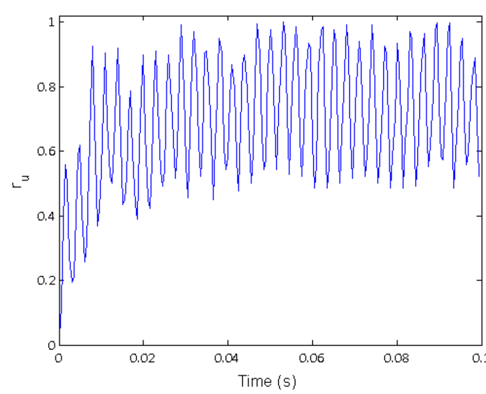

Figure 10. Pore pressure increment.

Results show that in this type of tests the input factor which establishes the liquefaction initiation is the frequency. Hence, high frequencies produce high accelerations, and at same time the accelerations generate the pore pressure increasing. Also, fast accelerations do not permit the dissipation of pore pressure excess, which is validated by Jefferies & Been (2015). Thereby, it was evidenced that tests with frequencies lower than 105 Hz not show soil phenomena. Liquefaction and no liquefaction test report is presented in Table 5.

Table 5. Physical parameters of soil.

Test input		Fine sandy soil		Medium sandy soil		Coarse sandy soil	
Amplitude (mV)	Frequency (Hz)	Dr (%)	Occurrence	Dr (%)	Occurrence	Dr (%)	Occurrence
	1.34	NP*	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
200	1.45	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	2.04	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	1.34	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
500	1.45	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	2.04	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	1.34	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
800	1.45	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	2.04	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	90	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
200	105	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	207.5	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
	90	NP	Non liquefaction	NP	Non liquefaction	NP	Non liquefaction
500	105	21.67	Non liquefaction	22.34	Non liquefaction	21.82	Non liquefaction
	207.5	22.87	Non liquefaction	20.03	Non liquefaction	23.38	Non liquefaction
	90	24.45	Liquefaction	23.45	Non liquefaction	22.45	Non liquefaction
800	105	23.63	Liquefaction	25.58	Liquefaction	21.37	Non liquefaction
	207.5	25.17	Liquefaction	26.88	Liquefaction	20.91	Liquefaction
	144	22.44	Liquefaction	23.51	Liquefaction	21.76	Liquefaction
200	168	23.57	Liquefaction	22.54	Liquefaction	22.25	Liquefaction
	332	19.68	Liquefaction	21.41	Liquefaction	21.77	Liquefaction
	144	20.54	Liquefaction	25.46	Liquefaction	26.75	Liquefaction
500	168	23.63	Liquefaction	21.75	Liquefaction	19.87	Liquefaction
	332	21.71	Liquefaction	20.11	Liquefaction	25.01	Liquefaction
	144	21.59	Liquefaction	24.13	Liquefaction	19.98	Liquefaction
800	168	24.31	Liquefaction	20.17	Liquefaction	22.63	Liquefaction
	332	22.69	Liquefaction	22.69	Liquefaction	24.82	Liquefaction

* Not performed test

Experimentally, it was recorded the time of the liquefaction trigger. Thereby, of a population of 81 tests, only 33 presented liquefaction. The findings were estimated by the pore pressure sensor and the accelerometers. Furthermore, analytically the parameter used to estimate the liquefaction susceptibility was the CSR. This parameter is the ratio of the average cyclic shear stress τ developed on horizontal surfaces as a result of the cyclic loading to the initial vertical effective stress σ'_0 acting on the soil layer (Seed et al. 1983). Liquefaction resistance work made by Been & Jefferies (1985) revealed that the liquefaction resistance is associated to the critical state of soil and it can be represented by an exponential function.

The liquefaction resistance calculated is expressed that the model which predicted the liquefaction potential approach corresponding to exposed by Seed & Idriss (1971); Kramer (1996); Andrus & Stokoe II (2000); Viana da Fonseca et al. (2011); Jefferies & Been 2015; Soares & Viana da Fonseca (2016). However, the previous authors evaluated the liquefaction phenomena with methods different to shaking table modelling. In other way, it was found that the liquefaction susceptibility is lower in coarse sands compared with fine sands. The above, is validated according to mentioned by Zienkiewicz et al. (1980); and Ishihara (1993). Since the hydraulic conductivity does not permit that the pore pressure build up in a short time. Figure 11 represents a model to evaluate the liquefaction resistance from of CSR values and measure of acceleration at the liquefaction initiation.

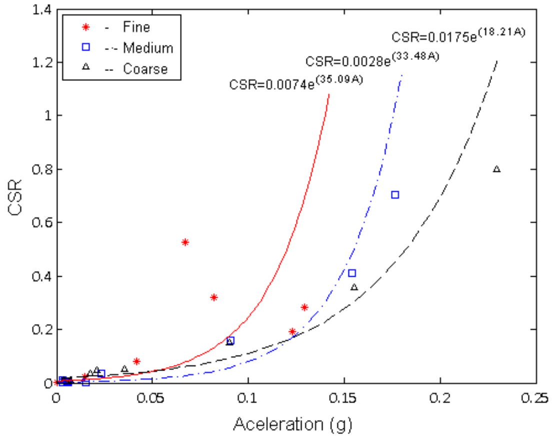

Figure 11. Liquefaction susceptibility curves

5 CONCLUSIONS

In this research work a flexible, impermeable and frictionless container was build. Also, a shaking table device was instrumented and implemented to evaluate the dynamic behaviour of soils. It is expected that this apparatus will be used in scale models of geotechnical centrifuge. Furthermore, the liquefaction susceptibility of three different size particle of Guamo sand were assed, through the variation of amplitude and frequency of the input signals. The experimental measures revealed that:

The device response can be evaluated by displacement and accelerations. It was found that the displacement is controlled by the amplitude of the input signal and the acceleration by the frequency applied.

The water deposition method is the most adequate preparation process to assess liquefaction phenomena in shaking table apparatus. Because, low relative densities are obtained with this method. Also, the density values of relative density achieved are uniform, as is showed by its normal probability distribution (μ=22.83 and σ=2.00). Also, this preparation process ensures the soil saturation, due to the particle accommodation inside of the water.

By means the comparison of pore pressure build up measurement and acceleration data is possible to evaluate the liquefaction phenomena in shaking table test. The increment of pore pressure in addition of densify process observed during the test performance established the liquefaction initiation. Further, the soil stiffness reduction, reflected the change of acceleration, allows estimate the instant which the phenomena start.

The liquefaction susceptibility assessed in shaking tables devices is controlled by the system acceleration. Since that the acceleration generates shear stresses in the soil, is it possible affirm that the liquefaction phenomena is triggered by the frequency of the signal applied. If the system acceleration is low, is necessary generate high values of CSR to trigger the liquefaction initiation. Nevertheless, at high accelerations higher shear stress are produced and the most probable is that the liquefaction initiation occur.

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