Geomechanics and Engineering, *Vol. 16, No. 5 (2018) 547-558*
DOI: https://doi.org/10.12989/gae.2018.16.5.547

1g shaking table tests on residual soils in Malaysia through different model setups

Jun X. Lim[1a], Min L. Lee[*2] and Yasuo Tanaka[1b]

[1]Department of Civil Engineering, Lee Kong Chian Faculty of Engineering and Science, Universiti Tunku Abdul Rahman, Selangor, Malaysia
[2]Department of Civil Engineering, Faculty of Engineering, University of Nottingham Malaysia, Selangor, Malaysia

(Received July 4, 2017, Revised October 4, 2018, Accepted October 11, 2018)

Abstract. Studies of soil dynamic properties in Malaysia are still very limited. This study aims to investigate the dynamic properties of two selected tropical residual soils (i.e., Sandy Clay and Sandy Silt) and a sand mining trail (Silty Sand) in Peninsular Malaysia using 1g shaking table test. The use of 1g shaking table test for soil dynamic testing is often constrained to large strain level and small confining pressure only. Three new experimental setups, namely large laminar shear box test (LLSBT), small chamber test with positive air pressure (SCT), and small sample test with suction (SSTS) are attempted with the aims of these experimental setups are capable of evaluating the dynamic properties of soils covering a wider range of shear strain and confining pressure. The details of each experimental setup are described explicitly in this paper. Experimental results show that the combined use of the LLSBT and SCT is capable of rendering soil dynamic properties covering a strain range of 0.017%-1.48% under confining pressures of 5-100 kPa. The studied tropical residual soils in Malaysia behaved neither as pure sand nor clay, but show a relatively good agreement with the dynamic properties of residual soils in Singapore. Effects of confining pressure and plasticity index on the studied tropical residual soils are found to be insignificant in this particular study.

Keywords: residual soil; 1g shaking table test; secant shear modulus; damping ratio; confining pressure

1. Introduction

Peninsular Malaysia is seismically affected by local earthquakes from internal fault zones and far-field tremors from neighbouring countries. The 2004 Aceh earthquake and 2015 Sabah earthquake are among the impactful earthquake incidents occurred in the country in recent years (Balendra and Li 2008, Cheng 2016). Most of the local studies pertaining to the impact of earthquake focused on the structural stability of buildings by means of experimental testing and numerical simulation (Adnan and Suradi 2008, Nazri and Alexander 2012). Studies on soil dynamics and geotechnical earthquake engineering are still very limited in Malaysia (Sooria *et al*. 2012, Tanaka and Lee 2016). More than three-quarters of the land areas in Peninsular Malaysia are covered by residual soils (Taha *et al*. 2000). The dynamic properties of these residual soils are expected to be more complicated than those pure sandy or clayey materials. Therefore, the researches in the realm of soil dynamics have to be carried out progressively to enrich the database of dynamic properties of soils in Malaysia.

In laboratory testing of dynamic soil properties, secant shear modulus (G) and damping ratio (D) are widely regarded as the most important parameters (Hardin and Drnevich 1972). Eqs. (1) and (2) can be used to evaluate the secant shear modulus and damping ratio from a hysteresis loop, respectively. Dynamic properties of soils can be influenced by several factors, including effective confining stress, plasticity index, shear strain amplitude, void ratio, degree of saturation, etc (Hardin and Black 1968).

$$G = \frac{\Delta\tau}{\Delta\gamma} \tag{1}$$

where G = Secant shear modulus, kPa, $\Delta\tau$ = Shear stress, kPa, $\Delta\gamma$ = Shear strain amplitude

$$D = \frac{1}{2\pi} \times \frac{\int \tau \, d\gamma}{(0.25 \times \Delta\tau \times \Delta\gamma)} \tag{2}$$

where D = Damping ratio, $\Delta\tau$ = Shear stress, kPa, $\Delta\gamma$ = Shear strain amplitude, $\int \tau d\gamma$ = Area of hysteresis loop, kPa.

There are a wide variety of laboratory tests that can be used to investigate the dynamic behaviours and properties of soils including resonant column test, cyclic triaxial test, cyclic direct simple shear test, cyclic torsional shear test, and centrifuge/1g shaking table test. In general, the element tests are particularly useful for investigating the dynamic properties of small soil samples under laboratory-controlled testing conditions, while the model tests (i.e., shaking table test) are mainly used to study the effect of soil-structure interaction, response of modelled geotechnical structure, and the behaviour of large soil models (Kramer 2014). Brennan *et al*. (2005), Kazama *et al*. (1996), and Kazama and Yanagisawa (1996) employed dynamic

*Corresponding author, Associate Professor
E-mail: minlee.lee@nottingham.edu.my
[a]Ph.D. Student
E-mail: limjx3@1utar.my
[b]Adjunct Professor
E-mail: yasuo@utar.edu.my

http://www.techno-press.org/?journal=gae&subpage=7
ISSN: 2005-307X (Print), 2092-6219 (Online)

Table 1 Confining pressures and strain ranges reported in previous 1g shaking table tests

Literature	Confining Pressure	Strain Range (%)
Araei and Towhata (2014)	<16 kPa	0.014-1.200
Dietz and Muir Wood (2007)	8.4 kPa	0.010-0.600
Tsai and *et al*. (2016)	<30 kPa	0.010-0.100
Tanaka and Lee (2016)	<10 kPa	0.092– 1.257

centrifuge shaking table tests to examine the dynamic behaviours of geomaterials through a series of cyclic motions under various gravitational conditions. There are also several studies reported on the findings of using the 1g shaking table test to investigate the deformation behaviours and dynamic properties of different types of soils (Araei and Towhata 2014, Dietz and Muir Wood 2007, Tsai *et al*. 2016, Tanaka and Lee 2016, Liu *et al*. 2018).

The 1g shaking table test is capable of reproducing simple shear deformation in a soil model through the propagation of shear waves induced by the cyclic movements of the shaking table platform (Kazama *et al*. 1996). The mechanism, which produces mechanical energy from the base towards the soil model, can facilitate the understanding of soil deformation behaviour under a close-to-actual seismic action. Dynamic properties covering a specific range of shear strain amplitudes can be obtained from the 1g shaking table tests, but it was limited by the low confining pressure exerted on the soil model. Table 1 summarizes the confining pressure and shear strain ranges that have been successfully achieved by using the 1g shaking table test from the previous studies (Araei and Towhata 2014, Dietz and Muir Wood 2007, Tsai *et al*. 2016, Tanaka and Lee 2016). The confining pressures are generally limited to below 30 kPa and the ranges of shear strain amplitudes lay in between 0.01% and 1%. Applications of higher confining pressures on the large soil model are normally restricted by the practicality and safety concerns (Koo *et al*. 2017). The shortcomings of applying low confining pressures on the soil model as in the conventional 1g shaking table test can be avoided by conducting the conventional element tests (e.g., cyclic triaxial test, cyclic simple shear test, and etc.) in which a smaller soil sample is tested in the experiment. In the element test, dynamic properties covering a wide strain range (i.e., from 0.0001% to >1%) can be obtained under confining pressures as high as 100-200 kPa (Leong *et al*. 2003, Lanzo *et al*. 1997, Tanaka and Lee 2016). Based on the concept of conventional soil element test, it is intuitively believed that the dynamic properties of soil, which involves a wide range of shear strain amplitudes and confining pressures, can be obtained by combining the results of both large soil model (i.e., laminar shear box test) and small soil sample (as similar to that of cyclic simple shear test) tested on a 1g shaking table. It is also worth mentioning that small specimen tests were significantly affected by the effect of boundary conditions. The cubical soil specimens tested in the Cambridge's simple shear apparatus were found to be affected by the non-uniformity of stress/ strain and effect of stress concentration (Budhu 1983) owing to the influence of side walls. Therefore, it is important to take into account of the boundary effect when developing a new small specimen test apparatus operated with simple shear mechanism.

It has been well agreed that the shear modulus of soil attenuates with the increase of shear strain amplitude. Based on a large number of experimental data, statistical analyses were carried out to form the relationship between normalized secant shear modulus and shear strain amplitude (Oztoprak and Bolton 2013, Vardanega and Bolton 2013, Darendeli 2001). Eqs. (3) and (4) can be employed to formulate the degradation curves of sandy soils and fine-grained soils, respectively (Oztoprak and Bolton 2013, Vardanega and Bolton 2013). In addition, the damping ratio curve for a wide variety of soils can be established using Eq. (5) (Ishibashi and Zhang 1993).

Equation of normalized shear modulus for sandy soil (Oztoprak and Bolton 2013)

$$\frac{G}{G_{\max}} = \frac{1}{\left[1+\left(\frac{\gamma-\gamma_e}{\gamma_r}\right)^a\right]} \tag{3}$$

where G = Secant shear modulus, MPa, G_{max} = Maximum shear modulus, MPa, γ = Shear strain amplitude, %, Lower bound curve, $\gamma_e = 0$; $\gamma_r = 0.02\%$; a = 0.88, Mean curve: $\gamma_e = 0.0007\%$; $\gamma_r = 0.044\%$; a = 0.88, Upper bound curve: $\gamma_{ref} = 0.003\%$; $\gamma_r = 0.10\%$; a = 0.88

Equation of normalized shear modulus for clayey soil (Vardanega and Bolton 2013)

$$\frac{G}{G_{\max}} = \frac{1}{\left[1+\left(\frac{\gamma}{\gamma_{ref}}\right)^{0.94}\right]} \tag{4}$$

where G = Secant shear modulus, MPa, G_{max}= Maximum shear modulus, MPa, γ = Shear strain amplitude, γ_{ref} = 3.7 (PI / 1000); PI = Plasticity Index, %

Equation of damping ratio for soils (Ishibashi and Zhang 1993)

$$D = \frac{0.333(1+e^{-0.0145PI^{1.3}})}{2}\left\{0.586\left(\frac{G}{G_{\max}}\right)^2 - 1.547\left(\frac{G}{G_{\max}}\right)+1\right\} \tag{5}$$

where D = Damping ratio, %, e = Void ratio, PI = Plasticity index, %, G = Secant shear modulus, MPa, G_{max} = Maximum shear modulus, MPa

The present study aims to investigate the dynamic properties of selected residual soils in Peninsular Malaysia by using different models of experimental setups on a 1g shaking table. Three experimental setups, i.e., one conventional large shear box, and two newly developed small soil models are attempted to evaluate their performance in evaluating the dynamic properties of soil under various strain ranges and confining pressures. Despite of the fact that many types of soil dynamic testing apparatus can be employed, reasonable and consistent results with respect to an identical deformation mode can only be obtained by using different models of experimental setups tested on a 1g shaking table. The developed experimental setups can facilitate cyclic simple shear deformation and examine the dynamic properties of soils. The present study intends to focus on dynamic properties over a relative large strain strange under simple shear deformations. The authors have previously conducted on the investigation of soil properties covering small strain range (Tanaka and Lee 2016). The results obtained are then compared with the established degradation curves reported from previous studies.

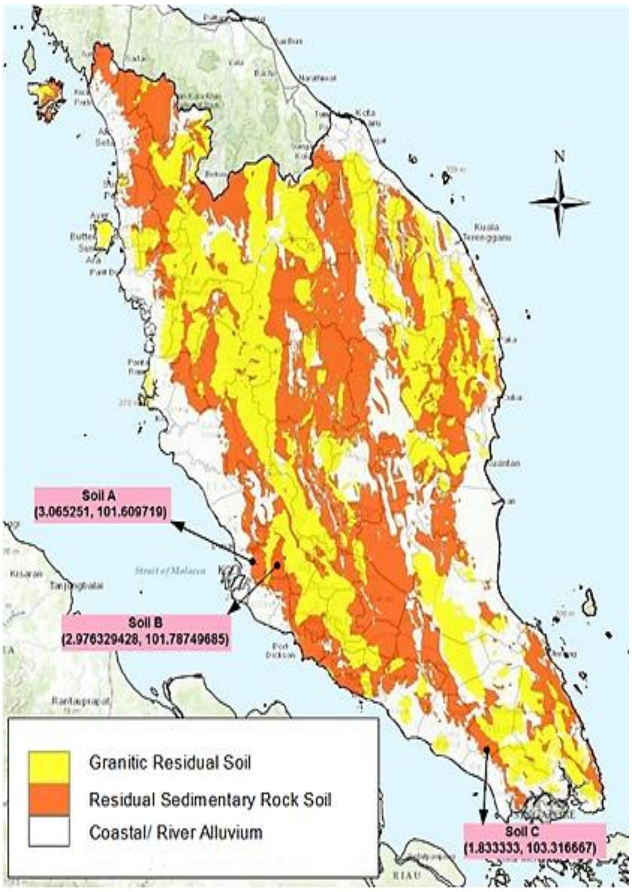


Fig. 1 Locations of soil sampling sites

Table 2 Physical properties of soils

Properties	Soil A	Soil B	Soil C
Composition			
Gravel	0%	12%	13%
Sand	46%	30%	57%
Fine Content	54%	58%	30%
Plastic Limit	22	27.5	19.9
Liquid Limit	68	45.5	24.5
Plasticity Index (PI)	46	18	5
Soil Classification	Sandy Clay	Sandy Silt	Silty Sand
Maximum Dry Density (kg/m^3)	1570	1640	1970
Optimum Moisture Content	23%	20.8%	11.8%
Void Ratio (compacted soil)	0.688	0.616	0.345
Degree of Saturation (compacted soil)	88.59%	89.48%	90.63%

2. Testing materials and methods

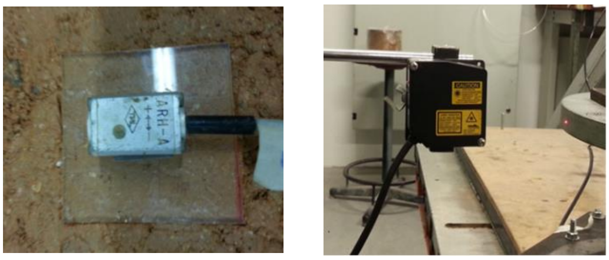
2.1 Soil sampling and physical properties

In this study, three types of soil were sampled from the superficial layer (i.e., at about 2 m below ground surface) at the selected sites in Peninsular Malaysia. The soil samples consisted of a sand mining trail and two typical types of tropical residual soil in Malaysia. The tropical residual soils were sampled from a site at Kajang area (Soil A) and a site at Simpang Renggam area (Soil B), while the sand mining trail was sampled from a site at Sunway area (Soil C). The specific locations of these soil sampling sites are presented in Fig. 1. The sand mining trail (Soil C) is not a residual soil but it was studied for evaluating the performance of the newly developed models in the present study by comparing the experimental results with the dynamic properties of similar sandy materials reported from previous literature (Hardin and Black 1968, Oztoprak and Bolton 2013). Based on the locations of the sampling sites as shown in the Fig. 1, the two selected tropical residual soils (i.e., Soil A and Soil B) are originated from the sedimentary rock. The soil deposit of Kajang formation (Soil A) belongs to a metasedimentary rock formation which consists of schist and phyllite (Gue and Wong 2009). The soil in Simpang Renggam area (Soil B) is originated from a clastic sedimentary rock formation which consists of shale material (Tan and Azwari 2001). In tropical countries, schist and shale would produce mostly silty materials or soils with illitic clay minerals as a result of the physical and chemical weathering process (Huat *et al*. 2012). In terms of the geological age of rock, the Kajang schist is within the Silurian and Ordovician periods, while the rock of Simpang Renggam is in the middle to late Permian period.

Table 2 summarizes the results of soil physical tests conducted in compliance with the British Standard, BS 1377: Part 2 (BSI, 1990). Based on the British Soil Classification System, Soil A was classified as Sandy Clay, Soil B was Sandy Silt, and Soil C was Silty Sand.

2.2 Instrumentation

Accelerometers (as shown in Fig. 2(a)) were used to monitor the changes of acceleration with time during the shaking table test. Three units of TML accelerometers (model: ARH-20A) and five units of KYOWA accelerometers (model: ASW-2A) were used in the experiment. The TML accelerometer has an acceleration measuring range from 10 m/s^2 to 500 m/s^2, while the KYOWA accelerometer has an acceleration measuring range from 9.807 m/s^2 to 196.1 m/s^2. Both of the accelerometers can withstand a water pressure up to 500 kPa (5 bars). It is worth noting that an acrylic plate of 2 mm thickness was screwed onto the surface of the accelerometers in order to provide a uniform contact surface with soil when subjected to vibration. For measurements on a metal surface, the accelerometers were attached directly on the surface by using adhesive-tape mounting method. This mounting method has an advantage of providing electrical insulation between accelerometer and the metal contact surface (Piersol and Paez 2010).



(a) Accelerometer (b) Laser displacement sensor

Fig. 2 Testing instrumentation

Laser displacement sensors were utilized to monitor the changes of linear displacement with time accurately. The model of the laser displacement sensors used in the present study was OPTEX FA CD5-85, as shown in Fig. 2(b). It has a measuring range of 85 ± 20 mm with a measuring resolution of 1 μm and a minimum sampling interval as low as 100 μs. Calibrations were also done on accelerometers where all accelerometers were subjected to identical shaking motions. The laser displacement sensors were calibrated by the instrument manufacturer as they were highly sensitive sensors that required special calibration tools.

2.3 Experimental setups

2.3.1 Large laminar shear box test

A 1g shaking table was used to investigate the dynamic behaviours of soils using three different experimental setups. The 1g shaking table was capable of providing one-dimensional cyclic motions with varying loading frequency and displacement under single-gravitational condition. The details of 1g shaking table system have been reported by Koo *et al*. (2017). In specific, the 1g shaking table apparatus was calibrated in terms of the actual linear displacement, frequency of movement, and the maximum acceleration being generated (Fig. 3).

Figs. 3 (a) and 3(b) show the schematic diagram and the photograph of the large laminar shear box test (LLSBT) used in the present study. The LLSBT setup consisted of an aluminium laminar shear box of 1.5 m (length) × 0.7 m (width) × 0.21 m (height), and a surcharge loading applied on top of the soil model to produce an overburden pressure. The soil model was instrumented with accelerometers to monitor the acceleration during the shaking table test. It was anticipated that the large sample of LLSBT could give a better replication of the in-situ soil condition as compared to small laboratory samples (Koo *et al.* 2017, Kazama and Yanagisawa 1996). There were three layers of aluminium shear stacks used to confine the soil model, with the height of each stack was 0.07 m. The base of the aluminium shear stacks was rigidly clamped on the shaking table platform, and the shear stacks were allowed to move freely relative to each other.

For the sample preparation, the soil was compacted to 95 % of the maximum dry density. A wood tamper, which was coated with latex, was fabricated to compact the soil sample into six successive layers. The interface between the soil layers was scratched to minimize heterogeneity between the compacted layers. A plywood panel was placed on top of the compacted soil model. Nails were protruded approximately 3 mm into the soil sample in order to reproduce shear stress induced by inertia force from the surcharge loading. The surcharge loading (i.e., 5 kPa and 10 kPa) was formed by a timber box containing sandbags and steel plates. A surcharge loading weighed 1000 kg could reproduce an overburden pressure of 10 kPa.

Seven accelerometers were embedded at the centre of the soil model from the base to the top surface at a height interval of 3.5 cm in order to evaluate the complete displacement profile along the sample height. In addition, an accelerometer was attached on the surcharge loading container to measure the acceleration induced by the surcharge loading when the soil model was subjected to a shaking motion. By knowing the acceleration trace of the surcharge loading, the inertia shear force or shear stress applied on the soil can be computed. A laser displacement sensor was used to measure the linear displacement of the shaking table platform. The measurement can be compared with the displacement derived from the accelerometer attached on the base of the shaking table for verification purposes.

2.3.2 Small chamber test with positive air pressure

The surcharge loading that could be applied in the LLSBT was limited by the practical constraints. For an example, a surcharge loading as high as 1000 kg was required to produce an overburden pressure of merely 10 kPa. The application of higher overburden pressures to replicate the in-situ soil at a deeper depth was restricted by safety concern and limited space in the laboratory. A new model, namely small chamber test with positive air pressure (SCT), was developed to overcome this limitation. Fig. 4 shows the schematic diagram and photograph of the SCT. The small soil sample was subjected to a positive air confining pressure isotropically inside the chamber to enable a higher confining pressure than the LLSBT. It was also anticipated that a smaller shear strain amplitude than that of LLSBT can be achieved in the SCT in order to provide insights into the deformation behaviour of soil over different strain ranges. The small soil sample was radially confined by confining air pressures of 50 kPa and 100 kPa. In the experiment, the conventional boundary condition problem induced from the side walls of apparatus can be overcome by either applying a positive confining pressure through air pressure (SCT) or a negative air suction pressure (SSTS) on the small soil specimens. Besides, the small soil specimen was radially confined by a sleeve of flexible rubber membrane (thickness of 0.3 mm) in which pneumatic air pressure was applied on. The cylindrical soil specimen was subjected to a confining pressure as similar to the specimen in standard triaxial test. Through this kind of boundary condition, the interaction effect as imposed by the side walls can favourably be avoided.

The small soil specimen as adopted in this study was 150 mm (diameter) × 100 mm (height) whereas the maximum particle size was limited to 5 mm only. It follows that the ratio between the smallest length of soil specimen and the largest soil particle size is twenty. Budhu (1983) conducted a simple shear test on cuboidal soil specimen (10 cm × 10 cm × 2 cm) for a Leighton Buzzard sand, which has an average grain size of 1 mm. Therefore, the dimension of soil specimen being adopted in the present study can be justified with that being reported by Budhu (1983) in which the ratio of specimen dimension and particle size is identical under the feature of simple shear deformation. The soil compaction was done in four successive layers with the interfaces of soil layers were scratched. A piece of filter paper was placed at the bottom of soil sample to allow for drainage when the confining pressure was applied. The compacted soil sample was sandwiched by a base pedestal and a top platen. Numerous pins were attached on the surface of the base and top plates to facilitate a uniform shear stress distribution between the plates and soil. The soil sample was confined by a sleeve of pre-fabricated rubber membrane and tightened by a pair of O-rings at the top and bottom of the sample. The soil model was then fixed into a PVC cylindrical chamber with an acrylic plate cover. Two accelerometers were attached on the top platen and the base pedestal, respectively. A laser displacement sensor was also used to measure the linear displacement of the shaking table platform for verification purposes. Herein, the compacted soil specimens were allowed to consolidate for 15 minutes, which was based on the previous experience from carrying out the monotonic triaxial test. However, the volume change of the compacted soil specimens was not measured during the consolidation stage because the compacted soil specimens were inherently unsaturated and therefore complicated the measurement of volume change during consolidation. Despite of the fact that the volume change was not measured in the experiment, the obtained dynamic properties of compacted soil specimen were beneficial considering the specimens had been consolidated at certain confining pressure, within a reasonable duration.

2.3.3 Small chamber test with negative air pressure

In the SCT, the top and bottom displacements were derived from the measurement of accelerometers. The derived displacement was, however, subjected to uncertainties caused by the methods of signal processing. The laser displacement sensor used was only capable of providing verification on the movement of the shaking table platform, but not the direct measurement on the soil sample. A new small sample test with suction (SSTS) was developed to further improve this experimental limitation by measuring the displacements of soil specimen directly. This apparatus can avoid the interference of uncertainties arisen from data processing by using laser displacement sensor. Fig. 5 shows the schematic diagram and the photograph of SSTS. Instead of supplying a positive air confining pressure into the soil chamber, the pressure surrounding the soil sample in the SSTS was maintained at atmospheric pressure while air suction (negative pressure) was applied directly to the bottom of the soil sample to create an effective stress condition in the soil. The exclusion of the outer chamber in the SSTS enabled a direct measurement of laser displacement sensor on the soil body. Two laser displacement sensor heads were required in order to measure the changes of shear strain profile with time. The suction (i.e., 50 kPa and 80 kPa) was supplied from a convum which was capable of converting a positive air

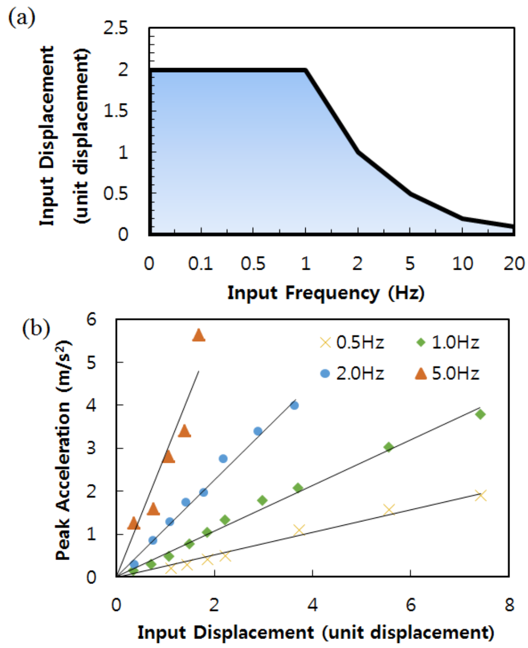


Fig. 3 (a) Feasible input motions for shaking table apparatus and (b) Relationship between peak acceleration and input displacement

Table 3 Testing parameters

Testing set-up	Input motion	Variables		
		Confining pressure (kPa)	Strain	Soil type
LLSBT	Table 4(a)	0, 5, 10	Large	A, B, C
SCT	Table 4(b)	0, 50, 100	Medium	A, B, C
SSTS	Table 4(b)	0, 50, 80	Medium	A, B, C

Table 4 Input motions (a) LLSBT

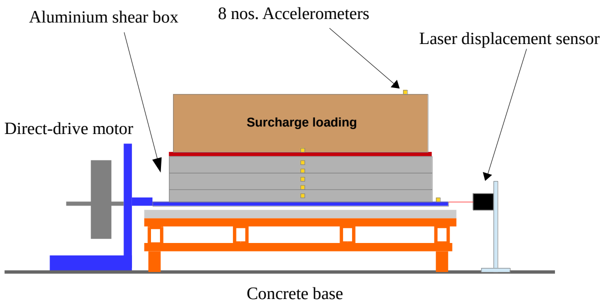
No	Input Motion	
	Frequency (Hz)	Displacement (mm)
1	0.1	1
2	0.1	5
3	0.5	7
4	1	4
5	1	7
6	2	2
7	2	4
8	5	1
9	5	2
10	20	0.1

Table 4 (b) SCT and SSTS

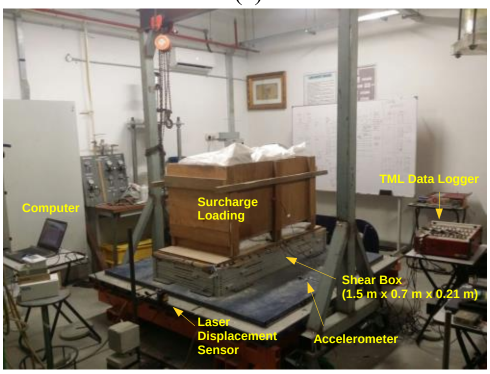
No	Input motion	
	Frequency (Hz)	Displacement (mm)
1	0.5	0.4
2	0.5	0.7
3	2	0.8

Table 4 (b) Continued

No	Input motion	
	Frequency (Hz)	Displacement (mm)
4	2	2
5	4	0.8
6	4	2
7	6	0.8
8	6	2



(a)



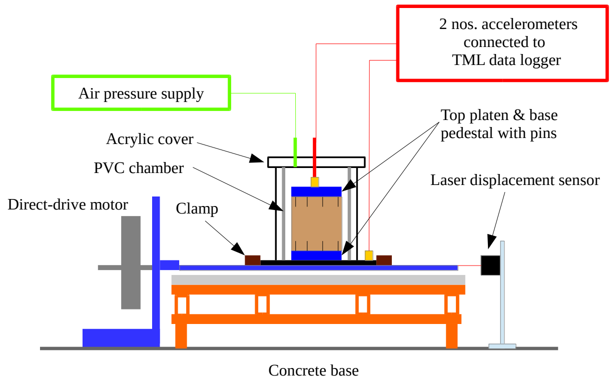
(b)

Fig. 4 (a) Schematic diagram and (b) photograph of LLSBT

pressure to a negative air pressure. It should be noted that the maximum suction that can be produced was 80 kPa only as limited by the capacity of the convum.

The small sample tests (SCT and SSTS) could effectively facilitate simple shear deformation whereby 1D wave propagation mechanism could be observed. The shear wave energy transmitted from the base towards the top surface of the soil specimen. The dimension of small specimen being adopted in the present study corresponded to the soil height that simple shear deformation was able to be observed in the LLSBT. In the LLSBT, the phenomenon of simple shear was observed in the soil interval at an elevation of 10.5 cm from the base of soil model (Koo *et al.* 2017). Therefore, it can be confirmed that the occurrence of simple shear deformation and identical wave propagation mechanism as in the LLSBT could effectively be reproduced in the small sample tests.

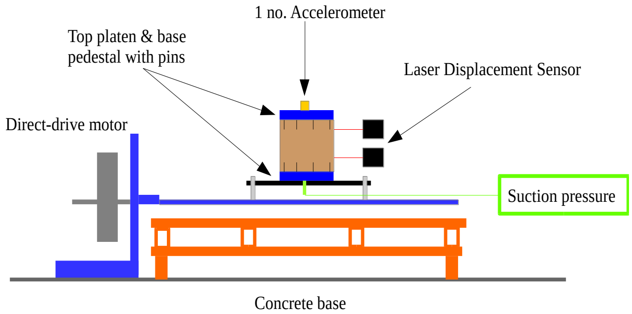
From the foregoing, the three developed experimental setups on the 1g shaking table can be used to investigate the dynamic properties of soils with different features of apparatus. Cyclic simple shear deformation can be anticipated in the three setups tested on the 1g shaking table.



(a)

(b)

Fig. 5 (a) Schematic diagram and (b) photograph of SCT



(a)

(b)

Fig. 6 (a) Schematic diagram and (b) photograph of SSTS

2.4 Experimental parameters

Table 3 shows the testing variables involved in the present study, while Tables 4(a) and 4(b) present the input motions (in terms of frequency and displacement) tested in the LLSBT, SCT, and SSTS.

3. Signal processing

The measured acceleration data were processed by removing the baseline drift and unwanted noises from the actual signal before they were used for subsequent analysis. Simple quadratic baseline correction, Osaki (1995) baseline correction, and 4th order Butterworth bandpass filtering methods were attempted to process the raw acceleration data and compared with the actual displacement measurements from laser displacement sensors. From the comparison (Fig. 6), it was decided to adopt the 4th order Butterworth bandpass filtering method as the signal processing method in the present study since the displacement profile obtained upon performing the filtering method showed the best agreement with the direct displacement measurements. A detailed discussion on the influence of noises and systematic approaches in selecting the most suitable signal processing method can be referred to the work done by Lim *et al*. (2017).

4. Result and discussion

A series of hysteresis loops can be obtained after obtaining the stress and strain profiles. The enclosed loop area represents the work done in the system and the representative slope indicates the shear modulus of soil (Brennan *et al*. 2005). The mathematical expressions for computing the shear modulus and damping ratio can be referred to Eq. (1) and Eq. (2), respectively. Brennan *et al.* (2005) suggested that the most reliable method to obtain the representative shear modulus is by computing the ratio between the difference in maximum and minimum shear stresses, and the difference in maximum and minimum shear strain amplitudes (refer to Eq. (6)).

$$G = (\tau_{max} - \tau_{min}) / (\gamma_{max} - \gamma_{min}) \tag{6}$$

The damping ratio as computed based on the stress-strain relationship was also claimed to be affected by varying degrees of high-frequency signal that could cause the loop crossing effect. Fig. 7 shows the samples of single-cycle hysteresis loop to represent the response of cyclic movement at the selected soil interval in LLSBT, SCT and SSTS. It is apparent that the SSTS was unable to render a reasonable and consistent hysteresis loop due to the problem of data synchronization and the effect of noise caused by the supply of suction pressure at the bottom of soil sample. In the SSTS, the difference of displacements and the corresponding shear strain were measured directly by using a pair of laser displacement sensor heads. The acceleration data was measured by using an accelerometer mounted on the top platen of the soil sample. However, the acceleration and displacement records in the SSTS were not electronically synchronized with time because of using two instrumentation devices with different data loggers.

Secant shear moduli and damping ratios were evaluated from a series of representative hysteresis loops. The results of SSTS were discarded for discussion in this particular study because of unfavourable and inconsistent hysteresis loops obtained from the SSTS. Although numerous shaking magnitudes were attempted for the three selected soils, only

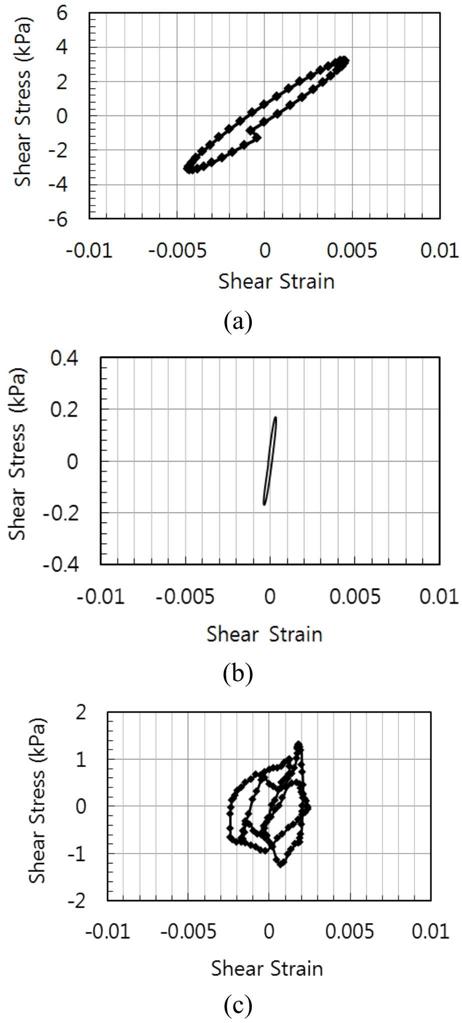


Fig. 7 Hysteresis loops for (a) LLSBT, (b) SCT, and (c) SSTS

the reliable and favourable results were included for discussion purpose. The unfavourable results showed a crossing configuration in the hysteresis loops. Under a series of uniform cyclic movements, a hyperbolic stress-strain relationship was expected in a hysteresis loop (Hardin and Drnevich 1972). This provokes a need for further improvement on the setup of SSTS in the future. The maximum shear moduli were computed in accordance with the formula suggested by Hardin and Black (1968) and ranged from 10.6 MPa to 23.7 MPa depending on the soil type.

Figs. 8(a)-8(c) show the relationships between normalized secant shear modulus and shear strain amplitude for the Soils A, B and C, respectively as compared with the established degradation curves. The established degradation curves for sandy and clayey soils were based on the studies reported by Oztoprak and Bolton (2013) and Vardanega and Bolton (2013), respectively. The degradation curves for clayey soil were established in accordance with the plasticity indexes of the studied soils. In addition, the experimental data were compared with a degradation curve for a Singapore tropical residual soil reported by Leong *et al.* (2003). It is worth mentioning that the degradation curve

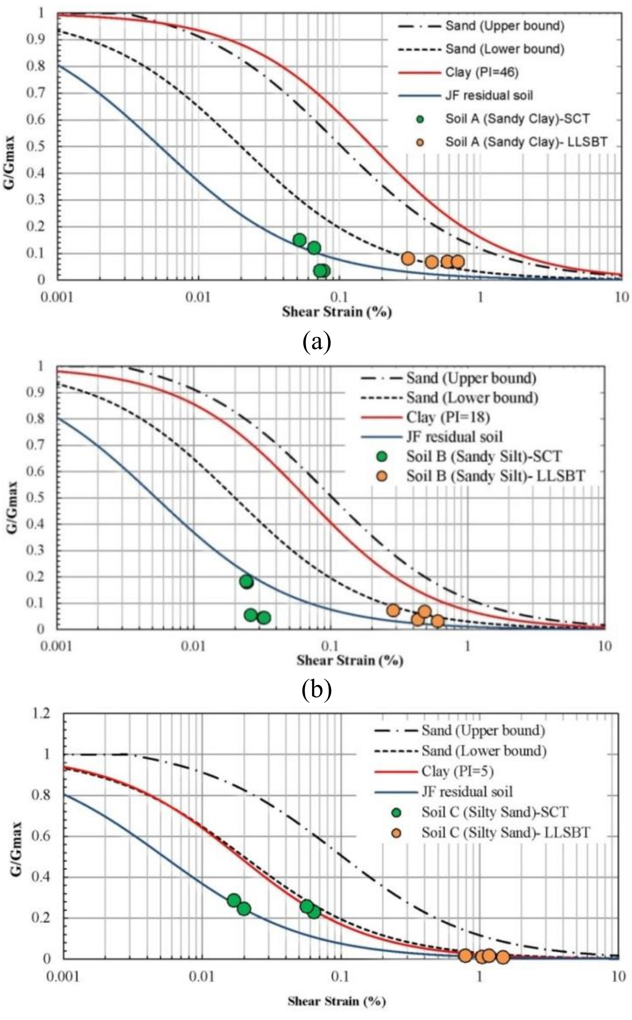


Fig. 8 Degradation curves for tested soils: (a) Soil A (Sandy Clay), (b) Soil B (Sandy Silt), and (c) Soil C (Silty Sand)

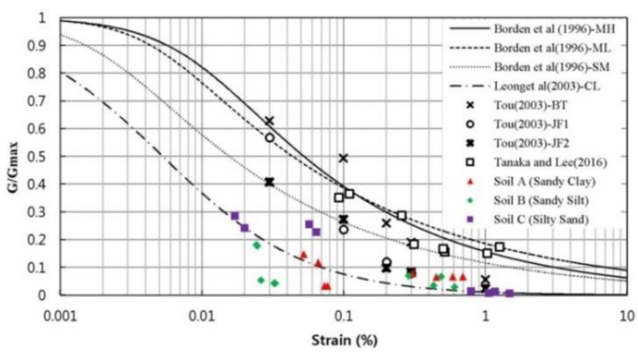


Fig. 9 Comparisons of shear modulus for different residual soils

was based on a cyclic triaxial testing result of a Singapore Jurong Formation (JF) residual soil, which comprised of low plasticity clayey materials with a fine content of 67%.

Experiments for the LLSBT and SCT were conducted under identical confining pressures (5 kPa and 10 kPa) and then compared to each other in an attempt to investigate the compatibility of experimental data. The comparison of experimental data between the LLSBT and SCT were realised to be difficult because LLSBT and SCT could facilitate wildly different shear strain ranges. Besides, a sand mining trail (Silty Sand) was selected to be tested in the three experimental setups and then compared with the experimental findings as reported by previous researchers. It was found that the experimental results of sand mining trail matched well with the dynamic behaviours of sandy material. It can thus be inferred that the developed experimental setups can give rise to reasonable results and therefore can be used to investigate the dynamic properties of soils. It was also found from the Fig. 8 that the experimental data points obtained from the SCT and LLSBT distributed tendentiously with respect to the established degradation curves for pure sand and clay.

From Fig. 8(a), the experimental data of Soil A (Sandy Clay) was plotted below the lower bound of established curve for sand and the established curve for clay. The established curves for sand and clay were developed from testing results of pure sand and clay, respectively. It was anticipated that the mixture of sand and clay in the natural tropical residual soil has altered their dynamic properties significantly compared to the pure materials. For comparison with residual soil, the experimental data of Soil A showed relatively good agreement with the degradation curve of Singapore JF residual soil reported by Leong *et al*. (2003).

Similar trend as Soil A was also observed in Soil B (Sandy Silt) as presented in Fig. 8(b). From the comparison between the experimental data and established degradation curves, it can be inferred that the two selected tropical residual soils (Soil A and Soil B) in Malaysia were unique and behaved neither as sand nor clay. This observation was probably caused by the composition of sand-to-fine content mixture, and the characteristics of parent rock formation of the residual soils. This finding provoked further investigation on the dynamic properties of tropical residual soils.

From Fig. 8(c), the experimental data of soil C (Silty Sand extracted from sand mining trail) showed a relatively better fit to the lower bound of sand compared with Soil A and Soil B. This was because Soil C consists of predominantly sandy material (i.e., content of coarse-grained material = 70%). However, the fine content in Soil C (i.e., 30%) was believed to have played a significant role on its dynamic properties. This was evidenced by some of the mismatched data points below the lower bound of the established curve for pure sand. In addition, Soil C was found to have higher shear moduli compared with Soil A and Soil B.

Fig. 9 shows the comparison of experimental results in the present study (Soil A and Soil B only) with the shear modulus of residual soils reported from different parts of the world (Borden *et al*. 1996, Tou 2003, Leong *et al*. 2003, Tanaka and Lee 2016). The LLSBT and SCT were able to facilitate soil movements from medium to large shear strain amplitudes (i.e., 0.017% to 1.48%). SCT was able to facilitate soil movements with smaller shear strain amplitudes (i.e., 0.017%-0.077%) as compared to the LLSBT.

Borden *et al*. (1996) investigated the dynamic properties of Piedmont residual soil in North Carolina, United States. The compositions of Piedmont residual soils ranged from silt to sand with different plasticity indexes. In their study,

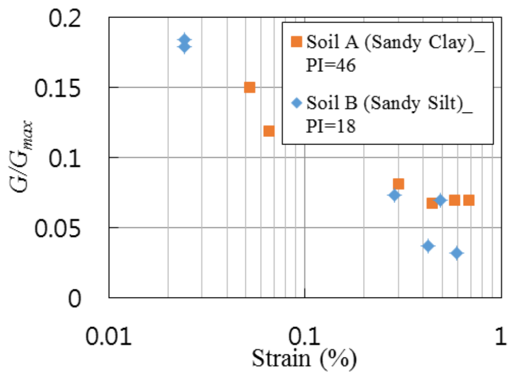
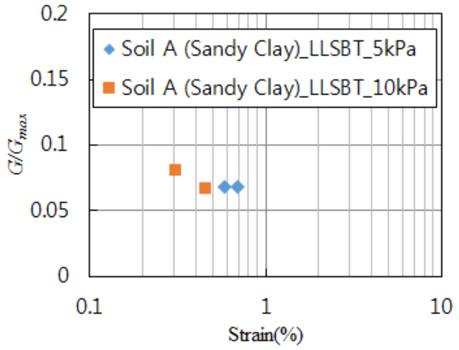
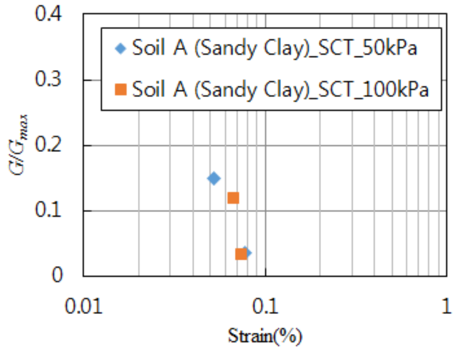


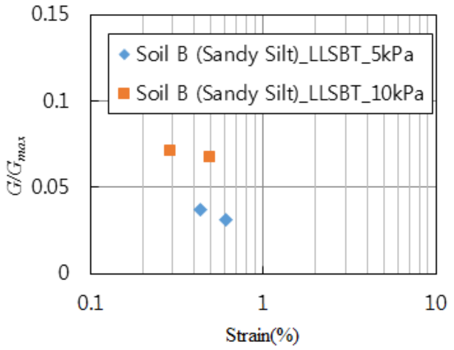
Fig. 10 Effect of plasticity index on shear modulus for soil A and soil B



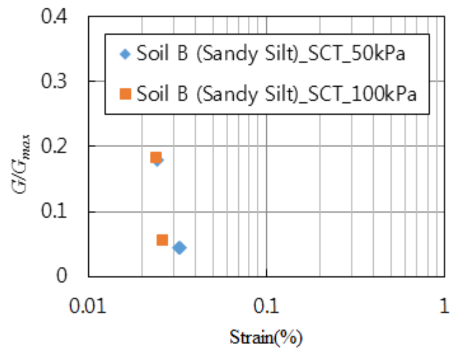
(a) LLSBT (Soil A)



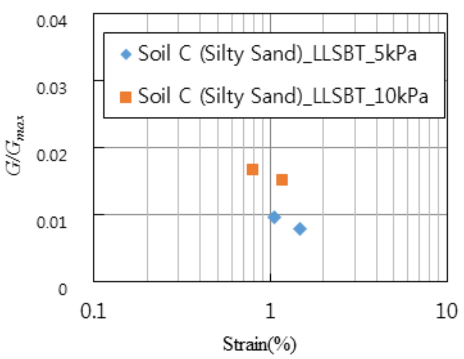
(b) SCT (Soil A)



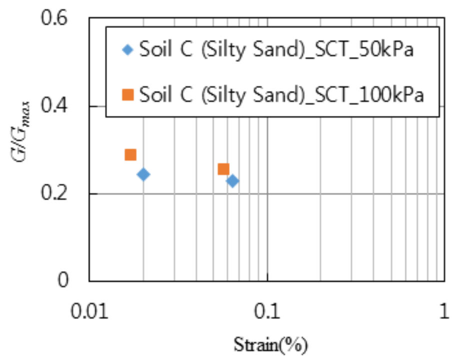
(c) LLSBT (Soil B)



(d) LLSBT (Soil B)



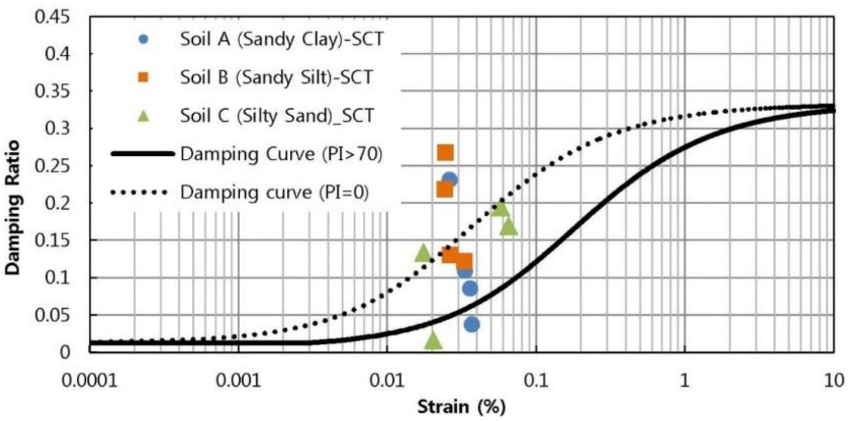
(e) LLSBT (Soil C)



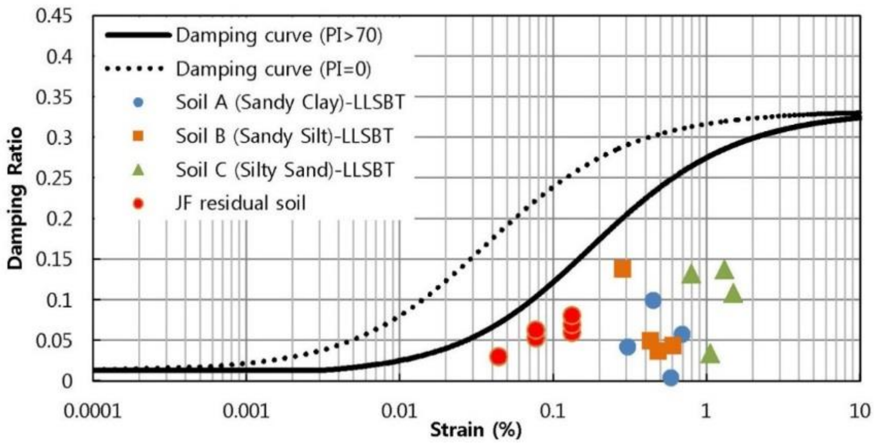
(f) SCT (Soil C)

Fig. 11 Effect of confining pressure on shear modulus

Borden *et al*. (1996) focused mainly on the small-strain properties of unsaturated soil samples using the resonant column and cyclic torsional shear tests with the shear strain amplitude below 0.1 %. From Fig. 9 it is apparent that the shear moduli from the Piedmont residual soil were greater than the experimental results of Soil A and Soil B. The discrepancy might be attributed to the physical properties of residual soils and the characteristics of parent



(a) SCT



(b) LLSBT

Fig. 12 Relationship between damping ratio and shear strain amplitude

rocks for different types of residual soil. The Piedmont residual soils consisted of a wide spectrum of fine contents (in terms of silt and clay) ranging from 10% to 90%, while the fine contents of Soil A and Soil B were ranging from 54% to 58%. Besides, the degree of saturation of the unsaturated Piedmont residual soils were 39%-98%, while the degree of saturation of the residual soils in the present study were 88.6% and 89.5% for Soil A and Soil B, respectively. From the literature, an unsaturated soil sample was expected to have a higher stiffness than the saturated soil sample, and hence the degree of saturation was an important parameter influencing the dynamic properties of residual soils (Kramer 2014). Soil A and Soil B were originated from the sedimentary rocks whereas the Piedmont residual soil was originated from the igneous and metamorphic rocks. In addition, the Piedmont soils are of sub-tropic residual soils while the residual soils in the present study were weathered under the tropical climate which was believed to have finer particles under the intense weathering effect. From the foregoing, it can be summarized that the dynamic properties of residual soil may be affected by numerous factors including fine content, degree of saturation, characteristics of parent rock, and weathering condition.

The effect of confining pressure on shear modulus was significant in sandy soil, while the effect of plasticity index prevailed on clayey soils (Oztoprak and Bolton 2013, Vardanega and Bolton 2013). At specific shear strain amplitude, the shear modulus increased with the increasing confining pressure. Fig. 10 shows the experimental shear modulus obtained in the present study for Soil A (Sandy Clay) and Soil B (Sandy Silt) with different plasticity indexes. The experimental data were obtained from the LLSBT and SCT, respectively. Apparently, the effect of plasticity index on the shear modulus of soil was undistinguishable. The shear moduli for both soils were plotted almost along an identical degradation curve despite of the fact that the plasticity index of Soil A (PI = 46) was considerably higher than Soil B (PI = 18). It can thus be implied that the studied residual soils in the present study did not behave as the pure clayey soil even though the fine contents were dominant in these soils.

As mentioned earlier, the effect of confining pressure was significant in sandy material. The effect of confining pressure on Soil C (70% of granular material) which was formed by sand mining trail was investigated to confirm the statement. As shown in Figs. 11(e) and 11(f), Soil C (Silty Sand) showed a good agreement with the characteristics of sandy material when subjected to different levels of confining pressures (i.e., 5 kPa, 10 kPa, 50 kPa and 100 kPa). At specific shear strain amplitude, the shear moduli for 100 kPa confining pressure were higher than those of 50 kPa in SCT, and likewise for LLSBT with confining pressures of 10 kPa and 5 kPa.

As for the tropical residual soils in the present study (Soil A and Soil B), the effect of confining pressure on the shear modulus was less distinguishable as shown in Figs. 11(a)-11(d). The results implied that the residual soils did not exhibit a similar dynamic behaviour as the sandy material. This was largely attributed to the presence of fine contents in the studied residual soils (Soil A = 54% and Soil B = 58% of fine contents).

Figs. 12(a) and 12(b) show the relationships between damping ratio and shear strain amplitude obtained from the SCT and LLSBT, respectively. The experimental damping ratio data were compared with the established damping ratio curves reported by Ishibashi and Zhang (1993), which was obtained from statistical analysis on non-plastic sand and plastic clay. From the experimental results, it was found that the damping ratio data were not increasing with the shear strain amplitude in both LLSBT and SCT. Besides, there was no direct relationship that can be traced between the damping ratio and the type of soil. For SCT, the experimental data points generally distributed within the range of the established damping ratio curves at smaller shear strain amplitudes. It should be noted that the damping ratio data reported by Leong *et al*. (2003) and Brennan *et al*. (2005) also scattered below the established damping ratio curves. The above-mentioned experimental observations from the LLSBT and SCT were caused by the fact that lower shear stress and shear strain levels (i.e., range of shear stress = 0.1-2.5 kPa and range of shear strain = 0.017%-0.077%) were obtained in the SCT compared with those of LLSBT (i.e., range of shear stress = 2-6 kPa and range of shear strain = 0.077%-1.48%). This was because the computation of damping ratio was defined by the area of hysteresis loop divided by the multiplication of the shear stress range and shear strain range. In this case, lower shear stress and shear strain ranges could give rise to a greater damping ratio using the SCT despite of the fact that the loop areas of the SCT were smaller than those of LLSBT. The low magnitude of inertia shear stress generated in the SCT was caused by the smaller loading mass applied on the top surface of soil sample compared with the higher overburden loading on top of the soil model in the LLSBT.

From the foregoing interpretation on the experimental results, it is suspected that the studied topical residual soils in Malaysia are unique and behave neither as pure sand nor clay. This finding provokes the need of further investigation on the dynamic properties of tropical residual soils in Malaysia.

5. Conclusions

Three experimental setups, namely large laminar shear box test (LLSBT), small chamber test with positive air pressure (SCT), and small sample test with suction (SSTS) were attempted to investigate the dynamic properties of selected soils in Malaysia using 1g shaking table test. The following conclusions can be drawn from the present study:

• Although the LLSBT setup has the advantage of replicating the in-situ soil condition with a larger size of soil model, it was impractical to apply a high overburden pressure on the soil model of LLSBT. To overcome this shortcoming, the setup of SCT was developed to apply higher confining pressures to the soil sample in which the actual stress condition of soil below the ground surface can be reproduced. The setup of SSTS was subsequently attempted to measure the deformation of soil sample directly by using a pair of laser displacement sensor. The experimental data of LLSBT and SCT were used as the main results for interpreting dynamic properties of soils, while the experimental data of SSTS was discarded because of the noise effect and the problem of data synchronization occurred in the experiment. Medium to large shear strain amplitudes of soil deformation can be obtained from the present testing setups, viz. LLSBT (i.e., 0.077%-1.48%) and SCT (i.e., 0.017%-0.077%). The combined use of the LLSBT and SCT enables the investigation of soil dynamic properties covering a wide range of shear strain amplitudes and confining pressures.

• From the experimental results of LLSBT and SCT, the shear moduli of Soil C (i.e., Silty Sand, with a fine content of 30 % only) was found to agree well with the established degradation curves of sand reported in literature. The fine content in Soil C was believed to play a significant role on the dynamic properties of soil. The experimental shear moduli of two studied tropical residual soils, namely Soil A and Soil B (i.e., Sandy Clay and Sandy Silt, with fine contents ranging from 54% to 58%) were plotted below the established degradation curves for pure sand and clay.

• The experimental results of studied tropical residual soils in Malaysia matched well with the results of a selected Singapore residual soil. Although the effects of confining pressure and plasticity index are widely regarded as influential parameters on dynamic properties of sand and clay in literature, those effects were found to be less distinguishable in the studied tropical residual soils. It is suspected that the studied topical residual soils in Malaysia are unique and behave neither as pure sand nor clay. However, it is still uncertain that the discrepancy is caused by the inconsistent results obtained from the new setup or the actual behavior of the soil. Further experimental investigations involving various types of tropical residual soils in Malaysia using more established testing setups have to be carried out to deepen the understanding on this kind of complicated geo-material.

Acknowledgements

The authors would like to acknowledge the financial supports from the Fundamental Research Grant Scheme (FRGS), Malaysia (Grant No. FRGS/2/2014/TK02/UTAR/02/1).

References

Adnan, A. and Suradi, S. (2008), “Comparison on the effect of earthquake and wind loads on the performance of reinforced concrete buildings”, *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing, China, October.

Araei, A.A. and Towhata, I. (2014), “Impact and cyclic shaking on loose sand properties in laminar box using gap sensors”, *Soil Dyn. Earthq. Eng.*, **66**, 401-414.

Balendra, T. and Li, Z. (2008), “Seismic hazard of Singapore and Malaysia”, *EJSE Special Issue Earthq. Eng. Low Moderate Seismic Reg. South East Asia Australia*, 57-63.

Bao, X.H., Morikawa, Y., Kondo, Y., Nakamura, K. and Zhang, F. (2012), "Shaking table test on reinforcement effect of partial ground improvement for group-pile foundation and its numerical simulation", *Soil. Found.*, **52**(6), 1043-1061.

Borden, R., Shao, L. and Gupta, A. (1996), "Dynamic properties of piedmont residual soils", *J. Geotech. Eng.*, **122**(10), 813-821.

Brennan, A., Thusyanthan, N., and Madabhushi, S. (2005), "Evaluation of shear modulus and damping in dynamic centrifuge tests", *J. Geotech. Geoenviron. Eng.*, **131**(12), 1488-1497.

BSI (1990), "Methods of test for soils for civil engineering purposes: Compressibility, permeability and durability tests", BS 1377-5: 1990, BSI, U.K.

Budhu, M. (1984), "Nonuniformities imposed by simple shear apparatus", *Can. Geotech. J.*, **21**(1), 125-137.

Cheng, K.H. (2016), "Plate tectonics and seismic activities Sabah area", *Trans. Sci. Technol.*, **3**(1), 47-58.

Darandeli, M.B. (2001), "Development of a new family of normalized modulus reduction and material damping curves", Ph.D. Thesis, The University of Texas at Austin, Austin, Texas, U.S.A.

Dietz, M., and Muir Wood, D. (2007), "Shaking table evaluation of dynamic soil properties", *Proceedings of the 4th International Conference on Earthquake Geotechnical Engineering*, Thessaloniki, Greece, June.

Enomoto, T. and Sasaki, T. (2015), "Several factors affecting seismic behaviour of embankments in dynamic centrifuge model tests", *Soil. Found.*, **55**(4), 813-828.

Gue, S.S. and Wong, S.Y. (2009), "Slope engineering design and construction practice in Malaysia", *Proceedings of the CIE-IEM Joint Seminar on Geotechnical Engineering*, Yilan, Taiwan, August.

Hardin, B. and Drnevich, V. (1972), "Shear modulus and damping in soils measurement and parameter effects", *J. Soil Mech. Found. Div.*, **98**(6), 603-624.

Hardin, B.O. and Black, W.L. (1968), "Vibration modulus of normally consolidated clay", *J. Soil Mech. Found. Div.*, **94**(SM2), 353-369.

Huat, B.B.K., Toll, D.G. and Prasad, A. (2012), *Handbook of Tropical Residual Soils Engineering*, (1st Edition), CRC Press, London, Unite.

Ishibashi, I. and Zhang, X. (1993), "Unified dynamic shear moduli and damping patios of sand and clay", *Soil. Found.*, **33**(1), 182-191.

Ishikawa, A., Zhou, Y.G., Shamoto, Y., Mano, H., Chen, Y.M. and Ling, D.S. (2015), "Observation of post-liquefaction progressive failure of shallow foundation in centrifuge model tests", *Soil. Found.*, **55**(6), 1501-1511.

Kazama, M. and Yanagisawa, E. (1996), "Dynamic properties of soft clay directly obtained from centrifuge dynamic test", *Proceedings of the 11th World Conference on Earthquake Engineering*, Acapulco, Mexico, June.

Kazama, M., Toyata, H., Towhata, I. and Yanagisawa, E. (1996), "Stress strain relationship of sandy soils obtained from centrifuge shaking table tests", *J. Geotech. Eng.*, **535**, 73-82 (in Japanese).

Koo, K.K., Lim, J.X., Yang, C.L., Lee, M.L., Tanaka, Y. and Zhao, J.J. (2017), "Shaking table test on dynamic behaviours of tropical residual soils in Malaysia", *KSCE J. Civ. Eng.*, **21**(5), 1735-1746.

Kramer, S.L. (2014), *Geotechnical Earthquake Engineering*, Pearson Education Limited, U.K.

Lanzo, G., Vucetic, M. and Doroudian, M. (1997), "Reduction of shear modulus at small strains in simple shear", *J. Geotech. Geoenviron. Eng.*, **123**(11), 1035-1042.

Leong, E.C., Rahardjo, H. and Cheong, H.K. (2003), "Stiffness-strain relationship of Singapore residual soils", *Proceedings of the 2003 Pacific Conference on Earthquake Engineering*, Christchurch, New Zealand, February.

Lim, J.X., Lee, M.L. and Tanaka, Y. (2017), "Systematic approaches for signal processing of soil shaking table test", *Proceedings of the 6th International Conference of Euro Asia Civil Engineering Forum (EACEF-2017)*, Seoul, South Korea, August.

Liu, N., Huang, Q.B., Fan, W., Ma, Y.J. and Peng, J.B. (2018), "Seismic responses of a metro tunnel in a ground fissure site", *Geomech. Eng.*, **15**(2), 775-781.

Nazri, F.M. and Alexander, N.A. (2012), "Predicting the collapse potential of structures", *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisbon, Portugal, September.

Osaki, Y. (1995), *New Introduction to Spectrum Analysis of Earthquake Ground Motion*, Kajimashuppan, Tokyo, Japan (in Japanese).

Oztoprak, S. and Bolton, M.D. (2013), "Stiffness of sands through a laboratory test database", *Geotechnique*, **63**(1), 54-70.

Piersol, A. and Paez, T. (2010), *Harris' Shock and Vibration Handbook,* McGraw-Hill Education, New York, U.S.A.

Shinoda, M., Nakajima, S., Nakamura, H., Kawai, T. and Nakamura, S. (2013), "Shaking table test of large-scaled slope model subjected to horizontal and vertical seismic loading using E-Defense", *Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering*, Paris, France, September.

Sooria, S.Z., Sawada, S. and Goto, H. (2012), "Proposal for seismic resistant design in Malaysia: Assessment of possible ground motions in Peninsular Malaysia", *Ann. Disas. Prev. Res. Inst. Kyoto Univ.*, Kyoto, Japan, September.

Suzuki, H., Tokimatsu, K. and Tabata, K. (2014), "Factors affecting stress distribution of a 3×3 pile group in dry sand based on three-dimensional large shaking table tests", *Soil. Found.*, **54**(4), 699-712.

Taha, M., Hossain, M., and Mofiz, S. (2000), "Behavior and modeling of granite residual soil in direct shear test", *J. Inst. Eng. Malaysia*, **61**(2), 27 - 40.

Tan, B.K. and Azwari, H.M. (2001), "Physio-chemical properties of carboneceauos shale soils in the Yong Peng area, Johor", *Proceedings of the Geological Society of Malaysia Annual Geological Conference 2001*, Pangkor Island, Perak, June.

Tanaka, Y. and Lee, M.L. (2016), "Laboratory study on dynamic properties of compacted residual soils in Malaysia", *Geotech. Eng. J. SEAGS AGSSEA*, **47**(4), 89-96.

Tou, J.H. (2003), "Cyclic triaxial test apparatus for evaluating dynamic properties of residual soils", M.Sc. Thesis, Nanyang Technological University, Nanyang, Singapore.

Tsai, C.C., Lin, W.C. and Chiou, J.S. (2016), "Identification of dynamic properties through shaking table tests on a large saturated sand specimen in a laminar shear box", *Soil Dyn. Earthq. Eng.*, **83**, 59-68.

Vardanega, P.J. and Bolton, M.D. (2013), "Stiffness of clays and silts: Normalizing shear modulus and shear strain", *Geotech. Geoenviron. Eng.*, **139**(9), 1575-1589.

Wang, B., Zen, K., Chen, G.Q. and Kasama, K. (2012), "Effects of excess pore pressure dissipation on liquefaction-induced ground deformation in 1-g shaking table test", *Geomech. Eng.*, **4**(2), 157-164.

Zarnani, S., El-Emam, M.M. and Bathurst, R.J. (2011), "Comparison of numerical and analytical solutions for reinforced soil wall shaking table tests", *Geomech. Eng.*, **3**(4), 291-321.

CC