Behaviour of MH silts with varying plasticity indices

Steven T. Y. Wong BEng(Hons)

Research Scholar, Research Centre for Sustainable Technologies, Faculty of Engineering, Science and Computing, Swinburne University of Technology Sarawak Campus, Kuching, Malaysia

Dominic E. L. Ong BE(Hons), PhD, PEng, MIEM CPEng, MIEAust

Associate Professor and Director, Research Centre for Sustainable Technologies, Faculty of Engineering, Science and Computing, Swinburne University of Technology Sarawak Campus, Kuching, Malaysia
(corresponding author: elong@swinburne.edu.my)
(Orcid:0000-0001-8604-8176)

Retnamony G. Robinson BEng(Hons), MSc, PhD

Professor, Civil Engineering Department, Indian Institute of Technology Madras, Chennai, India

It is important for geotechnical engineers to understand the intrinsic and mechanical behaviour of silt as it is presently recognised that there exist gaps in understanding its fundamental behaviour. The behaviour of kaolin samples with varying clay and silt contents was investigated in the present study. This study characterised the samples by their corresponding ranges of plasticity index (P_I) – namely, $P_I \leq 13\%$ and $P_I > 13\%$. This outcome is achieved by interpreting the results of Atterberg limit tests, particle size analysis, isotropically consolidated undrained triaxial tests and oedometer tests carried out on the kaolin samples with P_I between 7 and 16%. The results obtained in this study show good reliability when compared to 65 sets of past significant experimental studies derived from 33 established past research papers.

Notation

A_c	activity
e	void ratio
$C_L\%$	clay content
CL	low plasticity clay
CH	high plasticity clay
L_L	liquid limit
M	slope of the critical-state line (CSL)
MH	high-plasticity silt
ML	low-plasticity silt
O_{CR}	overconsolidation ratios
p'	effective normal pressure or mean effective normal stress
p'_c	effective confining pressure
P_I	plasticity index
P_L	plastic limit
q	deviator stress
q_f	deviator stress at failure
u_f	pore pressure at failure
v	specific volume
$\Delta\sigma_1$	change in total vertical stress
$\Delta\sigma'_1$	change in the effective vertical stress
$\Delta\sigma_3$	change in the total confining pressure
$\Delta\sigma'_3$	change in the effective confining stress
Δu	excess pore water pressure
ε_f	axial strain at failure
κ	gradient of swelling line
λ	gradient of compression line
λ_{CSL}	gradient of compression lines of CSL
λ_{NCL}	gradient of compression lines of the normally consolidated line
$\lambda_{NCL}/\lambda_{NCL}$	ratios of the gradients of compression lines
σ_1	total vertical stress

σ'_1	effective vertical stress
σ'_1/σ'_3	maximum principal stress ratio
σ_3	total confining pressure
σ'_3	effective confining stress
ϕ'	effective angle of shearing resistance friction

Introduction

Based on particle size and plasticity characteristics, soils are classified as sand, silt and clay. The behaviours of sand and clay are unique. However, the behaviour of silts lies in between the behaviours of clays and sands. Some silts behave more like sand, and some others more like clay depending on their basic characteristics. It is essential to identify silts behaving as sand-like or clay-like. Boulanger and Idriss (2006) found that sand-like (behaving more fundamentally as sand) and clay-like soils (behaving more fundamentally as clay) have some fundamental differences in terms of stress–strain behaviour, compressibility and the slope of the critical-state line (CSL) in the $e-\ln p'$ space against the slope of the normally consolidated line (NCL). The major observations of Boulanger and Idriss (2004) include the following.

- Sand-like materials exhibit different slopes for the CSL and NCL, while clay-like materials show a similar slope for the CSL and NCL in the $e-\ln p'$ space.
- The effective stress paths of sand-like materials in undrained monotonic shearing show an initially contractive response (positive pore pressure increments) followed by a transition to an incrementally dilative response (decreases in pore pressure).
- Sand-like materials have little compressibility such that their void ratio does not change significantly as the effective consolidation stress is increased, while clay-like materials are relatively more compressible.

In a subsequent study, Boulanger and Idriss (2006) proposed that materials with P_L less than 7% would have a sand-like behaviour, while materials with P_L greater than or equal to 7% would exhibit a clay-like behaviour. In addition, it was also described that fine-grained soils with P_L values between 3 and 6% may exhibit a transitional behaviour. As a result, an appropriate procedure for examining the liquefaction resistance of soils was developed based on the behaviour of the materials. Boulanger and Idriss (2006) stated that the Atterberg limits tests are more reliable than particle size analysis, particularly for clays (i.e. $<2\mu\text{m}$), in order to correlate to the stress-strain characteristics of soils and also to differentiate between clay-like and sand-like behaviours.

It is also vital to note that Ferreira and Bica (2006) reported that transitional soils with particle size distributions between those of clean sands and plastic clays develop their own respective unique NCL and CSL and are not in accordance with the critical-state framework described for either sands or clays. In a research study, Nocilla *et al.* (2006) recommended identifying a new framework to describe the behaviour of silts in the transitional form. Such a framework has yet to be identified or developed due to the behaviour of silt being perhaps more complex than those of sand and clay.

Much research has been reported on low-plasticity silt (ML), with a liquid limit (L_L) lower than 50% – namely, by Wang and Luna (2012), Boulanger and Idriss (2006), Nocilla *et al.* (2006) and Hyde *et al.* (2006). They dealt with the characterisation and/or mechanical properties of ML. However, it was observed that very limited studies have been carried out to understand the behaviour of high-plasticity silts (MH) with a liquid limit higher than 50%. In this research, MH silts with L_L that ranges from 52 to 64% were tested, and the results are reported.

Nocilla *et al.* (2006) investigated the behaviour of Italian silt with a plasticity index (P_L) of 12% and found that as the clay contents reduce, the behaviour of silt changes from a clay-like behaviour to a transitional form demonstrating behaviours between those of clays and clean sands. For this situation, the silt in the transitional form demonstrates neither a unique NCL behaviour nor any unique CSL behaviour. Hyde *et al.* (2006) reported that the mechanical properties of low-plasticity silt with P_L of 6% were sand-like. Furthermore, Wang and Luna (2012) characterised the low-plasticity Mississippi River Valley (MRV) silt with P_L of 6% by using triaxial compression tests with different overconsolidation ratios (O_{CR}) of 1, 2 and 8 and reported that no unique critical state could be observed. The CSL in the void ratio (e) against the

logarithm of mean effective normal stress (p') space was not parallel to the normal consolidation curve, thus possibly indicating a sand-like behaviour. Ladd *et al.* (1977) as cited by Wang and Luna (2012) stated that deviator stress of some clays can be normalised by the effective consolidation pressure. The deviator stresses of the MRV silts with different O_{CR} values were normalised by the effective consolidation pressures, after which the results indicated that the MRV silts had a unique clay-like behaviour. The method identified by Boulanger and Idriss (2006) for distinguishing between sand-like and clay-like behaviours has worked well on both low-plasticity (ML) silts presented by Nocilla *et al.* (2006) and Wang and Luna (2012). Nonetheless, Wang and Luna (2012) likewise reported that the work of Boulanger and Idriss (2006) did not capture the effect of O_{CR} on silt behaviour.

As such, it is envisaged that the work of Boulanger and Idriss (2006) can be further refined to accommodate a larger variety of soils that include high-plasticity silts (MH). In the present study, the authors carried out a series of experiments on four types of kaolin samples with different particle size distributions and plasticity values. The results are analysed along with the data presented in the literature so as to identify the clay-like and sand-like behaviours.

Laboratory testing programme

Material description

Four types of commercially available kaolin powder with varying clay contents – namely KM20, KM25, KM35 and KM55 – were

Figure 1. Particle size distribution of kaolinite samples

Table 1. Gradation, L_L , P_L and P_I values of the kaolin samples

Sample	Sand fraction: %	Silt fraction: %	Clay fraction: %	L_L : %	P_L : %	P_I : %	Soil classification	Activity = $P_I/\text{clay content}\%$ (Skempton, 1953)
KM20	4.44	84.61	10.95	52	44	7	MH	0.66
KM25	0.88	79.62	19.50	59	48	11	MH	0.55
KM35	0.88	78.00	21.12	62	46	16	MH	0.77
KM55	0.88	74.75	24.38	64	49	15	MH	0.62

Figure 2. Linear relationship between clay contents and L_L , P_L and P_I of kaolin samples

utilised to set up the reconstituted kaolin samples. The kaolin was mined at depths between 4.5 and 6.0 m below the ground. Aluminium silicate is the predominant chemical constituent of kaolin. Reconstituted kaolin samples were found to be ideal in this study on the grounds that kaolin (a) has appropriate ranges of particle sizes (silt and clay fractions), (b) is relatively less expensive than obtaining undisturbed soil samples from the field and (c) tests can be consistently repeated with confidence.

Soil properties

British standard BS 1377-2:1990 (BSI, 1990a) was adopted to determine the index properties such as L_L by using the Casagrande method as well as plastic limit (P_L) and particle size distribution. Both sieve and hydrometer analyses were carried out to determine the percentages of sand, silt and clay.

The particle size distribution of kaolin samples based on the sieve and hydrometer analyses is shown in Figure 1. The clay contents (<2 μ m) of KM20, KM25, KM35 and KM55 were found to be

10.95, 19.50, 21.12 and 24.38%, respectively, while the silt contents (2–60 μ m) of KM20, KM25, KM35 and KM55 were 84.61, 79.62, 78.00 and 74.75%, respectively.

The L_L , P_L and P_I of the kaolin samples are summarised in Table 1. They are found to be directly proportional to the clay contents of the kaolin samples as shown in Figure 2. The reconstituted kaolin samples are classified as high-plasticity silt (MH) in accordance with ASTM D 2487 (ASTM, 2000).

The activity (A_c) of fine-grained soils, the ratio of P_I to the percentage clay content, is the amount of water that is attracted to the surfaces of the soil particles. The amount of water attracted is largely influenced by the amount of clay that is present in the soil (Lambe and Whitman, 1969; Skempton, 1953). Skempton (1953) stated that A_c provides a convenient value for assessing the particular minerals found in clay. In other words, before X-ray diffraction tests are done on the samples, soils with different minerals, such as kaolin, can be differentiated by their respective A_c values since different minerals are characterised by their unique A_c values. Table 1 shows that the A_c of the reconstituted kaolin samples are 0.66, 0.55, 0.77 and 0.62 for KM20, KM25, KM35 and KM55, respectively, which are considered reasonable as they are generally close to the A_c value given by Skempton (1953) ($A_c = 0.46$) and Ferreira and Bica (2006) ($A_c = 0.66$).

Testing procedures

The one-dimensional (1D) consolidation (oedometer) tests were also conducted in accordance with BS 1377-5:1990 (BSI, 1990b) and ASTM D 2435 (ASTM, 2011) to examine the compressibility characteristics. Consolidated isotropic undrained (CIU) triaxial tests were performed based on BS 1377-8:1990 (BSI, 1990c) and Head (1998) in order to obtain the shear strength characteristics.

Strips of filter paper were attached on the sides of the specimens for radial drainage as per specifications by Head (1998) so as to accelerate the consolidation during the consolidation phase. The specimens were saturated by applying back pressures until Skempton's pore pressure parameter (B) value of at least 0.98 was achieved. The CIU tests were carried out at different initial effective

Table 2. Summary of the conducted tests

Test	KM20	KM25	KM35	KM55
L_L , P_L and P_I	✓	✓	✓	✓
Sieve and hydrometer	✓	✓	✓	✓
Oedometer	✓	✓	✓	✓
CIU	✓	✓	✓	✓
Effective confining pressure, p'_c : kPa				
100	KM20-100	KM25-100	KM35-100	KM55-100
150	KM20-150	KM25-150	KM35-150	KM55-150
200	KM20-200	KM25-200	KM35-200	KM55-200
250	KM20-250	KM25-250	KM35-250	KM55-250
300	KM20-300	KM25-300	KM35-300	KM55-300
400	KM20-400	KM25-400	KM35-400	KM55-400

confining pressures of 100, 150, 200, 250, 300 and 400 kPa. As the time taken for consolidation was shorter than 2 h for all specimens, a minimum of 2 h was considered in order to calculate the required shearing rates as suggested by Head (1998). As such, a shearing rate of 0.07 mm/min was adopted. A similar shearing rate was also reported by Pillai *et al.* (2011) for tests on kaolin.

During the consolidation phase of the triaxial test, the volume change of the sample was continuously monitored with time. The post-consolidation dimensions were determined using equations 6.3(b) and 6.3(c) of BS 1377-8:1990 (BSI, 1990c). Once the consolidation was over, the sample was sheared under undrained conditions. The axial load, axial deformation and the pore pressures were recorded using a load cell, a displacement transducer and a pore pressure transducer, respectively. The tests were continued until axial strain of 20% was reached. The corrected length was used for computing the strains during the shearing stage.

CIU tests at six different initial effective confining pressures of 100, 150, 200, 250, 300 and 400 kPa were carried out for each type of kaolin sample with varying clay-silt contents (KM20, KM25, KM35 and KM55). Therefore, a total of 24 CIU tests were performed. The samples were labelled based on the type of kaolin and the confining pressure in the triaxial test. For example, KM20-100 means 100 kPa confining pressure was applied on the sample KM20. The conducted tests are summarised in Table 2.

Sample preparation

Reconstituted and undisturbed samples

Burland (1990) demonstrated that the reconstituted clay should be mixed at higher water contents from 1.00 to 1.50 times its liquid limit (L_L) and preferably consolidated one-dimensionally to obtain the intrinsic properties of the soils. As there is no proper guideline for reconstituting kaolin silt, the mixing ratio as proposed by Burland (1990) has been adopted. A comparative mixing ratio was likewise embraced in the work of Pillai *et al.* (2011) for kaolin to prepare normally consolidated reconstituted samples.

In a different work by Hyodo *et al.* (1994), triaxial compression and extension tests were conducted on both normally consolidated undisturbed and reconstituted marine clays with a liquid limit of 124.2%, a plastic limit of 51.4% and a plasticity index of 72.8%. The monotonic axial load was applied at an axial strain rate of 0.1%/min. The test outcomes demonstrated that undisturbed and reconstituted samples behaved similarly in terms of deviator stresses and stress paths. Hyodo *et al.* (1994) added that the ageing effects such as chemical bonding or secondary compression for undisturbed samples were eliminated during the application of effective confining stress to the normally consolidated state during the consolidation stage. Therefore, the work of Hyodo *et al.* (1994) guaranteed that the behaviour of normally consolidated undisturbed samples can be decently depicted by reconstituted samples.

Sample preparation

The use of reconstituted kaolin samples prepared by consolidation of the soil-water mixture in the form of slurry state is a common approach in the laboratory testing of soils (Pillai *et al.*, 2011; Wang and Luna, 2012). Firstly, the kaolin powder was mixed with distilled water at a water content of 1.50 times its liquid limit. The water-soil mixture was mixed thoroughly in an automatic soil mixer to form the slurry. For triaxial tests, 38 mm dia. samples were used. The slurry was directly poured into the 38 mm dia. sampling tube after applying silicone grease on the inner surfaces of the tubes to reduce side friction. Subsequently, the sample was gradually loaded to a vertical consolidation pressure of 100 kPa. The vertical consolidation pressure was achieved by the application of a dead weight of 11.5 kg on the sample through a guide rod.

After the consolidation stage, the kaolin samples were extruded using a universal extruder. The reconstituted kaolin samples were then trimmed to 38 mm diameter and 76 mm height.

For the 1D consolidation tests, the slurry was carefully poured into a 60 mm dia. and 20 mm high 1D consolidation ring along with the collar so that the thickness of the sample was 30 mm. The slurry was initially consolidated to a consolidation pressure of 6.25 kPa. Once the consolidation under 6.25 kPa was complete, the sample was carefully trimmed to a thickness of 20 mm. The sample was then subjected to a 1D consolidation test with a load increment ratio of 1.0.

Results and discussions

CIU tests

Figures 3(a), 3(b) and 3(c) show the stress-strain behaviour, stress paths and pore pressure-strain behaviour of the reconstituted kaolin samples, respectively. For clarity, separate plots of these three parameters for the KM20, KM25, KM35 and KM55 samples can be found in Figures 4, 5, 6 and 7, respectively.

When the clay contents decreased (from KM55 to KM20), the deviator stresses at failure (q_f) increased as shown in Figure 3(a). The mean effective normal stress (p') and deviator stress (q) can be expressed as Equations 1 and 2a. Equation 2a can then be written in incremental form as shown in Equation 2b. For the CIU tests, the change in the total confining pressure ($\Delta\sigma_3$) equals zero. Hence, Equation 2b can be rewritten as Equations 2c and 2d as the changes in deviator stress (Δq) are defined as negative and positive for strain softening and hardening, respectively.

From Equations 2c and 2d, it is obvious that samples are expected to exhibit strain softening only if (a) the change in the total vertical stress ($\Delta\sigma_1$) or (b) the summation of the changes in the effective vertical stress ($\Delta\sigma'_1$) and excess pore water pressure (Δu) is negative. In other words, the kaolin samples will strain soften, due to the negative value of Δu (dilation) and/or negative value of $\Delta\sigma'_1$ (axial unloading).

Figure 3. (a) Stress–strain, (b) stress paths and (c) pore pressure–strain behaviours of kaolinite samples subjected to CIU tests

Similarly, the kaolin samples will exhibit strain hardening if (a) the change in $\Delta\sigma_1$ or (b) the summation of $\Delta\sigma'_1$ and excess pore water pressure (Δu) is positive. The kaolin samples will strain harden due to the positive values in Δu (contraction) and/or positive value in $\Delta\sigma'_1$ (compression)

$$1. \quad p' = \frac{\sigma'_1 + 2\sigma'_3}{3}$$

$$2a. \quad q = \sigma'_1 - \sigma'_3$$

$$2b. \quad \Delta q = \Delta\sigma'_1 - \Delta\sigma'_3 = \Delta\sigma_1 - \Delta\sigma_3$$

For softening

$$2c. \quad \Delta q = \Delta\sigma_1 = \Delta\sigma'_1 + \Delta u < 0$$

For hardening

$$2d. \quad \Delta q = \Delta\sigma_1 = \Delta\sigma'_1 + \Delta u > 0$$

The effective stress paths in Figure 3(b) were plotted in the Cambridge stress space with Equation 1 in the abscissa and Equation 2a in the ordinate. A similar approach was also adopted by Wang and Luna (2012) to understand the stress paths of silts. The effective stress paths of KM20 and KM25 with the lower P_I

Figure 4. (a) Stress–strain, (b) stress paths and (c) pore pressure–strain behaviours of KM20 kaolin samples subjected to CIU tests

values of 7% and 11%, respectively, curved away from the origin of mean effective normal stress (p'), but the effective stress paths of KM35 and KM55 with higher P_1 values of 16 and 15%, respectively, curved towards the origin of p' .

Figure 3(c) shows the pore pressure behaviour of the kaolin samples. Table 3 summarises the changes in effective vertical stresses and pore pressures. As shown in Figure 3(a), KM20 exhibited strain softening at σ'_3 values of 100, 150 and 200 kPa due to the development of negative values of Δu (dilation). This is because of having negative values in both Δu and $\Delta \sigma'_1$ as well as the summation of Δu and $\Delta \sigma'_1$ being similarly negative, resulting in the change in deviator stress (Δq) also being a negative value (see Equation 2c). Theoretically, KM20 is expected to exhibit strain softening at the aforementioned confining pressures, as observed in Figure 3(a) or Figure 4 for better clarity. However, on the contrary, with KM20 tested under confining pressures σ'_3 of

250, 300 and 400 kPa, the values of $\Delta \sigma'_1$ become positive (compression). As such, KM20 is expected to show strain hardening with positive Δq values as per Equation 2d and as observed in Figure 3(a) or Figure 4 for better clarity.

For KM25, dilation can be observed when the samples are tested with confining pressures of σ'_3 of 100, 200, 250, 300 and 400 kPa, except 150 kPa, with no change in Δu . $\Delta \sigma'_1$ at all effective confining pressures are negative values (denoting axial unloading behaviour), except $\sigma'_3 = 100$ kPa, which showed a small positive value (slight compression). Hence, with Equation 2c, KM25 is expected to show strain softening behaviour at all the tested σ'_3 values due to the summation of Δu and $\Delta \sigma'_1$ or Δq being all negative values. By comparing with Figure 5 for better clarity, this observation holds true.

For KM35, the samples have shown slight dilation at σ'_3 of 100 and 400 kPa, and no dilation or slight contraction at σ'_3 of 150,

Figure 5. (a) Stress-strain, (b) stress paths and (c) pore pressure-strain behaviours of KM25 kaolin samples subjected to CIU tests

200, 250 and 300 kPa as shown in Table 3. For KM55, the samples show slight dilation at σ'_3 of 100, 200, 250, 300 and 400 kPa; while at 150 kPa it shows no dilation. It can be seen that the strain softening and hardening of KM35 and KM55 samples are governed by their more significant and dominant values of $\Delta\sigma'_1$.

For example, KM35 has shown strain hardening at σ'_3 of 100 and 150 kPa due to the positive values of $\Delta\sigma'_1$ (compression) greater than Δu ; hence, the samples have shown positive values in Δq based on Equation 2d. For other σ'_3 values of KM35 and all σ'_3 values of KM55, the negative values of $\Delta\sigma'_1$ (axial unloading) are greater than Δu , so the samples have shown negative values (strain softening) in Δq of Equation 2c. By observing Figure 3(a), or for better clarity Figures 6 and 7, the behaviours of strain softening and strain hardening are very obvious.

Ishihara *et al.* (1975) defined the phase transformation point as the point at which the stress path turns its direction in p' - q space. They also observed that for cyclic triaxial tests, it is necessary for a sample to go at least once through this critical value in order to be taken to a completely liquefied state. In this sense, the phase transformation point may be considered as a threshold at which the behaviour of sand as a solid is lost and transformed into that of a liquefied state.

As shown in Figures 4(b), 5(b), 6(b) and 7(b), all samples exhibited the initial contraction before the phase transformation points. As shown in Table 3, all samples exhibit post-peak dilation except for KM25-150, KM35-150, KM35-200, KM35-250 and KM55-150. Except for KM20-250, KM20-300, KM20-400, KM35-100 and KM35-150, all the samples exhibited post-peak strain softening.

Figure 6. (a) Stress–strain, (b) stress paths and (c) pore pressure–strain behaviours of KM35 kaolin samples subjected to CIU tests

Critical state

The deviator stresses at failure (q_f) can be identified when either of the following conditions are achieved: (a) maximum principal stress ratio (σ'_1/σ'_3) and (b) ‘critical-state’ condition – that is, constant deviator stress and pore pressure. The corresponding values of strain and pore pressure are axial strain at failure (ϵ_f) and pore pressure at failure (u_f), respectively (Head, 1998). The work of Wang and Luna (2012) showed that the criterion of maximum principal stress ratio always gives a consistent estimation of the effective angle of shearing resistance. Hence, in the present study, the maximum principal stress ratio method has been used to identify q_f .

The outcome of projecting the effective stress paths in Figure 3(b) onto the semilogarithmic plot of $v\text{-}\ln p'$ is shown in Figure 8. As drainage was not allowed during the shearing stage of the sample, the effective stress paths in $v\text{-}\ln p'$ were expected only to move

horizontally since undrained shearing occurred at constant specific volume. It is observed that the CSLs and the NCLs are not parallel as shown in Figure 8 for the kaolin with lower clay contents (KM20 and KM25) due to the P_1 values of KM20 and KM25 being lower at 7 and 11%, respectively, compared to other samples, which have P_1 values greater than 13% (KM35 ($P_1 = 16\%$) and KM55 ($P_1 = 15\%$)). Therefore, it is defined that KM20 and KM25 behave as sand-like materials. A similar observation of non-parallel CSL and NCL curves was also reported by Wang and Luna (2012) in their study of triaxial compression test on the MRV silt with P_1 of 6%. Furthermore, it is interesting to observe that as the P_1 values of the kaolin samples increased starting from KM20 ($P_1 = 7\%$) and KM25 ($P_1 = 11\%$) to KM35 ($P_1 = 16\%$) and KM55 ($P_1 = 15\%$), the gradients of the respective CSLs seem to be getting more and more parallel to the gradients of their corresponding NCLs, indicating that the clay-like behaviour starts to be more prominent

Figure 7. (a) Stress–strain, (b) stress paths and (c) pore pressure–strain behaviours of KM55 kaolin samples subjected to CIU tests

progressively. Therefore, for KM20 and KM25 with the relatively lesser P_1 values, the non-parallel NCLs and CSLs seem to suggest that the sample demonstrated a more dominant sand-like behaviour instead, when compared to those KM samples with greater P_1 values.

As shown in Figure 9 for better clarity, the normalised deviator stress of KM20 and KM25 cannot be effectively bound by the effective consolidation pressures (p'_c) into a narrower single band, particularly at 100 kPa for p'_c , which indicates that they are not clay-like materials. Comparatively, KM35 and KM55 can be better normalised by p'_c into a single band and have shown a unique behaviour as clay-like materials.

Based on this important observation, since KM35 and KM55 have higher P_1 values and their NCLs and CSLs are much more parallel

(evident in Figure 8), it is thus postulated that KM35 and KM55 demonstrate clay-like behaviours. In view of this, a P_1 value of equal to or greater than 13% can be postulated as the boundary when clay-like behaviour types of soils would start to dominate. Similar results were also reported by Wang and Luna (2012) and Boulanger and Idriss (2006) in their works, which showed sand-like materials exhibiting non-parallel NCL and CSL behaviours.

Table 4 summarises the results of the CIU tests performed on the reconstituted kaolin soils. Based on the mean effective normal stress at failure (p'_f) and q_f from Table 4, the CSLs of reconstituted kaolin soil samples are then determined, as shown in Figure 10. The slope of the CSL increased when the clay contents of the reconstituted kaolin soils reduced. The slope of the CSL is M , and the effective angle of shearing resistance friction (ϕ') can be back-calculated using the following equation.

Table 3. Changes in effective vertical stresses and pore pressures

Sample	Axial strains: %	$\Delta\sigma'_1$	Δu	$\Delta\sigma'_1 + \Delta u$
KM20-100	19.0–20.0	–2.95	–3.95 (d)	–6.89 (s)
KM20-150	18.0–20.0	–1.82	–1.00 (d)	–2.82 (s)
KM20-200	15.5–20.0	–22.45	–5.00 (d)	–27.45 (s)
KM20-250	12.5–20.0	41.49	–6.00 (d)	35.49 (h)
KM20-300	16.0–20.0	7.43	–2.00 (d)	5.43 (h)
KM20-400	12.0–20.0	74.82	–12.00 (d)	62.82 (h)
KM25-100	7.0–20.0	0.67	–10.00 (d)	–9.33 (s)
KM25-150	18.5–20.0	–1.95	0.00 (—)	–1.95 (s)
KM25-200	11.0–20.0	–30.82	–3.00 (d)	–33.82 (s)
KM25-250	17.5–20.0	–15.19	–2.00 (d)	–17.19 (s)
KM25-300	12.0–20.0	–93.66	–5.00 (d)	–98.66 (s)
KM25-400	19.0–20.0	–7.71	–1.00 (d)	–8.71 (s)
KM35-100	10.5–20.0	23.86	–4.00 (d)	19.86 (h)
KM35-150	12.5–20.0	13.43	0.00 (—)	13.43 (h)
KM35-200	16.5–20.0	–47.85	0.00 (—)	–47.85 (s)
KM35-250	11.5–20.0	–30.75	0.00 (—)	–30.75 (s)
KM35-300	13.0–20.0	–83.37	10.00 (—)	–73.37 (s)
KM35-400	14.5–20.0	–59.88	–5.00 (d)	–64.88 (s)
KM55-100	12.5–20.0	–2.79	–1.00 (d)	–3.79 (s)
KM55-150	19.0–20.0	–1.95	0.00 (—)	–1.95 (s)
KM55-200	12.0–20.0	–36.16	–1.00 (d)	–37.16 (s)
KM55-250	12.0–20.0	–61.47	–1.00 (d)	–62.47 (s)
KM55-300	12.0–20.0	–56.27	–3.00 (d)	–59.27 (s)
KM55-400	16.5–20.0	–27.77	–1.00 (d)	–28.77 (s)

d, dilation; c, contraction; s, softening; h, hardening

$$3. \quad \sin \phi' = \frac{6M}{6 + M}$$

A similar approach was also used by Wang and Luna (2012) to find the ϕ' value. The M and ϕ' values of the kaolin soils are summarised in Table 4.

The relationship between the clay contents and effective angles of shearing resistance is indirectly proportional as shown in Figure 11. The correlation between clay contents ($C_L\%$) and effective angle of shearing resistance (ϕ') for the kaolin is

$$4. \quad \phi' = 34.6422 - 0.3323 \times C_L\%$$

with coefficient of determination (R^2) equal to 0.931. Similarly, Nocilla and Coop (2008) conducted CIU triaxial tests on alluvial sediments from the floodplain of Po River in Italy with $C_L\%$ ranging from 16 to 25% and P_1 ranging from 13 to 16%. It was also found that the back-calculated ϕ' showed a similar inverse correlation with $C_L\%$. However, the observed scatters in their data set could be due to the presence and the variation in silt and sand contents.

One-dimensional consolidation tests

The compressibility of the reconstituted kaolin was determined using 1D consolidation tests. The semilogarithmic $e-\log \sigma'_v$ (effective vertical stress) relationships obtained from the 1D consolidation testing are plotted in Figure 12. The initial void ratio is directly proportional to the L_L of the soils. If the L_L is higher, the initial void ratio will also tend to be higher due to the higher water content present within the void of the samples. The results of the 1D consolidation tests are summarised in Table 5. Furthermore, the gradient of compression line (λ) and the gradient of swelling line (κ) show a directly proportional relationship with the $C_L\%$ of the samples, as shown in Figure 13.

Figure 8. CSLs and isotropically NCLs of kaolin samples derived from CIU tests

Figure 9. (a) Normalised deviator stress–strain plots for KM20; (b) normalised deviator stress–strain plots for KM25; (c) normalised deviator stress–strain plots for KM35; (d) normalised deviator stress–strain plots for KM55

Gradient of compression lines of NCL and CSL from CIU tests

The gradients of the compression lines of the reconstituted kaolins were back-calculated from the NCLs and CSLs of the CIU tests as shown in Figure 8. Both the gradients of the compression lines of NCLs and CSLs are directly proportional to the clay contents as shown in Table 6. Furthermore, the gradient of the compression lines of NCL (λ_{NCL}) converges to the gradient of the compression

lines of CSL (λ_{CSL}) when the clay content of kaolin increases. This also indicates that the NCLs and CSLs of KM55 and KM35 are relatively more parallel than the NCLs and CSLs of KM20 and KM25. As shown in Figure 14 and Table 6, the ratios of the gradients of compression lines ($\lambda_{NCL}/\lambda_{CSL}$) of KM20, KM25, KM35 and KM55 are 0.702, 0.806, 0.846 and 0.948, respectively. A value of $\lambda_{NCL}/\lambda_{CSL}$ closer to 1.000 indicates that the NCL and CSL are more parallel. The R^2 of the $\lambda_{NCL}/\lambda_{CSL}$ and

Table 4. Results of CIU tests, slopes of CSLs and effective angles of shearing resistance

Sample	p'_c : kPa	p'_f : kPa	q_f : kPa	u_f : kPa	ϵ_{af} : %	M	ϕ : °
KM20	100	134.16	189.47	23	18.0	1.2496	30.73
	150	169.40	211.20	51	17.5		
	200	199.01	258.02	83	13.0		
	250	265.09	309.28	88	20.0		
	300	290.10	360.31	124	17.0		
	400	367.76	464.29	187	20.0		
KM25	100	106.27	141.82	39	12.5	1.1448	28.83
	150	156.22	153.67	45	18.0		
	200	173.88	206.65	95	10.5		
	250	219.54	250.63	114	17.0		
	300	257.86	278.57	134	11.5		
	400	337.70	401.11	196	19.0		
KM35	100	87.92	113.75	47	18.5	1.0996	27.75
	150	115.77	134.30	79	20.0		
	200	195.44	211.33	75	16.5		
	250	211.59	226.77	114	16.0		
	300	222.80	239.41	154	10.0		
	400	317.19	351.56	200	14.5		
KM55	100	78.69	89.07	47	12.5	1.0300	26.02
	150	98.61	112.82	89	19.0		
	200	159.46	160.38	94	9.5		
	250	198.24	207.72	121	14.5		
	300	223.05	231.16	149	10.5		
	400	307.13	309.40	196	16.5		

clay contents ($C_L\%$) for the kaolin samples is rather strong at 0.931 and the proposed correlation equation is

5.
$$\frac{\lambda_{NCL}}{\lambda_{CSL}} = 0.500 + 0.017 \times C_L\%$$

Figure 10. CSLs of tested kaolin samples

Distinguishing between clay-like and sand-like soil behaviours through plasticity index (P_I)

A sand-like behaviour is observed from the reconstituted kaolin soils of KM20 and KM25, characterised by their lower P_I values of 7 and 11%, respectively, as well as their CSL and NCL being non-parallel (see Figure 8). In the event that the method of Boulanger and Idriss (2006), which was developed for distinguishing clay-like (P_I greater

Figure 11. Relationship between clay content and effective angle of internal friction

Figure 12. Stress-void relationship during 1D consolidation tests

Table 5. Results of 1D consolidation tests

Sample	λ	κ
KM20	0.148	0.035
KM25	0.201	0.042
KM35	0.160	0.036
KM55	0.186	0.037

than or equal to 7%) and sand-like (P_I smaller than 7%) behaviours of silt based on P_I , was applied to the high-plasticity silt (MH) of KM20 and KM25, then KM20 and KM25 would be 'incorrectly' classified as having a clay-like behaviour.

To evaluate further the reliability of the original work of Boulanger and Idriss (2006) on MH, various P_I and L_L data

Figure 13. Relationship between clay content and compressibility

Table 6. Gradients of compression lines of NCLs and CSLs

Sample	λ_{NCL}	λ_{CSL}	$\lambda_{NCL}/\lambda_{CSL}$
KM20	0.09354	0.13324	0.702
KM25	0.08163	0.10117	0.807
KM35	0.10887	0.12863	0.846
KM55	0.10484	0.11055	0.948

Figure 14. Relationship between compression indices' ratio and clay content

points obtained from well-established papers (see Table 7) are subsequently used to develop Figure 15 together with the classification rules for sand-like and clay-like behaviours as reported by Boulanger and Idriss (2004). As illustrated in Figure 15, apparently, some of the well-documented sand-like soils are lying slightly above the $P_I = 7\%$ boundary suggested by Boulanger and Idriss (2006), which evidently demonstrates the conflict in trying to categorise the well-documented sand-like data to be within the $P_I = 7\%$ boundary. Therefore, it is now proposed that the boundary for describing sand-like behaviour soils be raised from $P_I = 7\%$ (Boulanger and Idriss, 2006) to $P_I = 13\%$ so that the well-documented data points (see Table 7) for soils with a sand-like behaviour can be fulfilled, including that of KM20 ($P_I = 7\%$) and KM25 ($P_I = 11\%$), whose sand-like behaviour is clearly demonstrated in Figure 8. The boundary $P_I = 13\%$ is also supported by Nocilla *et al.* (2006), who conducted reliable triaxial compression tests on Italian silts. In their work, the samples with clay contents of 4% ($L_L = 37\%$ and $P_I = 13\%$) and 8% ($L_L = 34\%$ and $P_I = 13\%$) had evidently demonstrated a sand-like behaviour as the effective stress paths showed an initially contractive response (due to increase in pore pressure) followed by a transition to an incrementally dilative response (due to decrease in pore pressure).

Finally, a threshold of P_I greater than 13% is now suggested for soils behaving in a clay-like manner. This would then include the

Table 7. Documented Atterberg limits for soils with sand-like and clay-like behaviours (continued on next page)

Soil name	Soil classification, ASTM D 2487 (ASTM, 2000)	L _L : %	P _i : %	Tests	Categories and evidence	References
Cohesive or clay-like behaviour						
1 B6 marine clay – James Bay	CL	37	13	Triaxial and direct simple shear	Cohesive, Figure 9	Ladd (1991); Boulanger and Idriss (2006)
2 Natural London Clay, 5.2 m BGL	CH	69	44	HCA stress path	C3, Figure 4	Nishimura <i>et al.</i> (2007)
3 Natural London Clay, 10.5 m BGL	CH	70	44	HCA stress path	C3, Figure 4	Nishimura <i>et al.</i> (2007)
4 Speswhite kaolin	MH	62	30	Triaxial	C3, p. 8, column 2, line 4 – p. 9, column 1, line 1	Georgiannou <i>et al.</i> (1990)
5 Aeolian silt	CL	37	18	Triaxial	C3, Figures 8–13	Cui and Delage (1996)
6 Sleaford silt (3 m depth)	CH	58	36	Triaxial and 1D consolidation	C3, Figure 3(a); C1, Figure 2	Lehane (2003)
7 Sleaford silt (6 m depth)	CH	70	48	Triaxial and 1D consolidation	C3, Figure 3(a); C1, Figure 2	Lehane (2003)
8 Sub-Apennine Blue Clays, By	CL	49	26.4	Triaxial and 1D consolidation	C3, Figure 13	Cotecchia <i>et al.</i> (2007)
9 Sub-Apennine Blue Clays, Bg	CL	51.1	27.7	Triaxial and 1D consolidation	C3, Figure 13	Cotecchia <i>et al.</i> (2007)
10 Sub-Apennine Blue Clays, P9	CH	69.3	38.4	Triaxial and 1D consolidation	C1, Figure 5	Cotecchia <i>et al.</i> (2007)
11 Sub-Apennine Blue Clays, P19	CL	51.8	28.8	Triaxial and 1D consolidation	C1, Figure 5	Cotecchia <i>et al.</i> (2007)
12 Sub-Apennine Blue Clays, P25	CH	65	35	Triaxial and 1D consolidation	C3, Figure 11	Cotecchia <i>et al.</i> (2007)
13 Sub-Apennine Blue Clays, P33	CH	53.4	27.5	Triaxial and 1D consolidation	C1, Figure 5	Cotecchia <i>et al.</i> (2007)
14 Sherbrooke laminated clay	CL	45	18	Triaxial	C3, Figure 7(a)	Long (2006)
15 Compacted clayey silt fill	CL	45.6	20.1	Triaxial	C3, Figures 5(c) and 5(d)	Almeida <i>et al.</i> (2012)
16 Bengawan Solo fill D1	MH	54	18	Triaxial	C3, Figure 8	Mountassir <i>et al.</i> (2011)
17 Bengawan Solo fill D2	MH	53	16	Triaxial	C3, Figure 8	Mountassir <i>et al.</i> (2011)
18 Mixtures of kaolin, sodium bentonite and London Clay	CL	28	18	Triaxial	C3, Figure 6	Cunningham <i>et al.</i> (2003)
19 Mexico Clay – oven-dried	MH	93	23	1D consolidation	C1, Figure 16	Mesri <i>et al.</i> (1975)
20 Residual London Clay	CH	80	51	Triaxial	Cohesive, Figure 11	Skempton (1985)
21 KM35	MH	62	16	Triaxial	C2, Figure 5	This paper
22 KM55	MH	64	15	Triaxial	C2, Figure 5	This paper
23 Grey organic clay	CL	38	19	Triaxial	C3, Figure 11	Long and O'Riordan (2001)
24 Bolkin silt	CL	29.4	15.6	Triaxial	C3, Figure 1	Wang <i>et al.</i> (2002)
25 Hong Kong marine deposits C4	CL	60	32	Triaxial and 1D consolidation	C3, Figure 12	Yin (1999)
26 Kaolin soil 68-32	ML	47	17	Triaxial and 1D consolidation	C3, Figure 6	Anantanasakul <i>et al.</i> (2012)
27 MSM10-3	CL	35	15	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
28 MSM10-6	CL	38	19	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
29 MSM10-14	ML	49	20	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
30 MSM10-43	CL	42	22	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
31 MSM10-48	CL	36	13	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
32 MSM10-52	CL	38	19	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
33 MSgM1-2	CL	34	14	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
34 MSgM1-3	CL	32	13	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
35 MSgM1-10	MH	56	24	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
36 MSgM1-22	MH	62	28	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
37 MSgM1-22b	MH	62	28	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
38 MSgM1-24	CL	41	19	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
39 MSgM2-11mb	ML	46	15	1D consolidation	C1, Figure 8	Biscontin <i>et al.</i> (2007)
40 Completely decomposed tuff	ML	43	14	Triaxial – consolidation	C2, Figure 3(a)	Chiu and Ng (2012)
41 Natural soil	CL	38	16	Triaxial	C1, Figure 3	Cetin and Soylemez (2004)
42 Nancy North-west silt	MH	56	25	Triaxial	C3, Figure 2	Ltifi <i>et al.</i> (2014)

Table 7. Continued

Soil name	Soil classification, ASTM D 2487 (ASTM, 2000)	L_L : %	P_I : %	Tests	Categories and evidence	References
43 Gorgon muddy silt	ML	45	15	Monotonic simple shear	C3, Figure 2	Mao and Fahey (2003)
44 Mud cake in silt	CH	58.5	28.8	Triaxial	C3, Figure 5	Zhang <i>et al.</i> (2009)
45 In situ soil in silt	CH	52.6	25.3	Triaxial	C3, Figure 5	Zhang <i>et al.</i> (2009)
46 Mud cake in clay	CL	43.9	20	Triaxial	C3, Figure 6	Zhang <i>et al.</i> (2009)
47 Italian silt (clay content 25%)	CL	46	22	Triaxial	C3, Figure 7	Nocilla <i>et al.</i> (2006)
48 Italian silt (clay content 45%)	MH	60	33	Triaxial	C3, Figure 7	Nocilla <i>et al.</i> (2006)
Cohesionless or sand-like behaviour						
1 KM20	MH	51	7	Triaxial	S2, Figure 5	This paper
2 KM25	MH	59	11	Triaxial	C2, Figure 5	This paper
3 Residual soil from Botucatu Sandstone	CL-ML	20	6	Triaxial and 1D consolidation	S1, Figure 5	Ferreira and Bica (2006)
4 Brown laminated clay	ML	35	12	Triaxial	S3, Figure 11	Long and O'Riordan (2001)
5 MRV silt	CL	28	6	Triaxial	S3, Figure 6	Wang <i>et al.</i> (2011)
6 Silt at Moss Landing B7-03	ML	36	11	Cyclic triaxial	S4, Figure 11	Boulanger <i>et al.</i> (1998)
7 Silt at Moss Landing B7-03	ML	31	6	Cyclic triaxial	S4, Figure 11	Boulanger <i>et al.</i> (1998)
8 Delhi silt S60M40	ML	27.5	4.5	Triaxial	S3, Figure 15	Usmani <i>et al.</i> (2011)
9 Delhi silt S20M80	CL	30	8	Triaxial	S3, Figure 15	Usmani <i>et al.</i> (2011)
10 Kaolin soil 45-55	CL-ML	28	7	Triaxial	S3, Figure 6	Anantanasakul <i>et al.</i> (2012)
11 Kaolin soil 24-76	CL-ML	20	4	Triaxial	S3, Figure 6	Anantanasakul <i>et al.</i> (2012)
12 Manglerud quick clay	ML	27	8	Direct shear	S3, Figure 9	Bjerrum and Landva (1966)
13 Adapazari silt	CL	30.5	5.5	Triaxial	S3, Figure 15	Arel and Onalp (2012)
14 Limestone powder	CL-ML	24	6	Triaxial	S3, Figures 5 and 6	Hyde <i>et al.</i> (2006)
15 Norwegian glaciomarine silt	CL	33	12	Triaxial	S3, Figure 9	Long <i>et al.</i> (2010)
16 Italian silt (clay content 4%)	CL	34	12	Triaxial	S3, Figure 7	Nocilla <i>et al.</i> (2006)
17 Italian silt (clay content 8%)	CL	37	13	Triaxial	S3, Figure 7	Nocilla <i>et al.</i> (2006)
18 Fraser River silt	CL	30.4	4.1	Cyclic direct simple shear	S4, Figure 5	Wijewickreme and Sanin (2010)
19 Blended silt mixture 1	ML	26	0	Triaxial	S3, Figure 9	Boulanger and Idriss (2006)
20 Blended silt mixture 2	ML	30	4	Triaxial	S3, Figure 9	Boulanger and Idriss (2006)
21 Blended silt mixture 3	ML	36.5	10.5	Triaxial	S2, Figure 8 (replotted including all scatter points)	Boulanger and Idriss (2006)

Extracted from Boulanger and Idriss (2004)

BGL, below ground level; HCA, hollow cylinder apparatus

S1: sand-like; sands have a small enough compressibility that their void ratio does not change significantly as the effective consolidation stress is increased

S2: sand-like; the slope of the CSL in void ratio (e) against the logarithm of mean effective stress (p') space is different from the slope of virgin consolidation line

S3: sand-like; the effective stress paths for sand in undrained monotonic shearing often show an initially contractive response (positive pore pressure increments since volume change is zero) followed by a transition to an incrementally dilative response (decreases in pore pressure)

S4: sand-like; during the undrained cyclic stress-strain loops, the sands develop a very flat middle portion (where the shear stiffness is essentially zero) that is observed for sands after the excess pore pressure reaches a limiting value, which corresponds to the sample temporarily having zero effective stress ($r_u = 100\%$)

C1: clay-like; clays have a large enough compressibility that their void ratio is highly dependent on the effective consolidation stress and consolidation stress history

C2: clay-like; the slope of the CSL in void ratio (e) against the logarithm of the mean effective stress (p') space is the same as the slope of virgin consolidation line

C3: clay-like; the effective stress paths for clay in undrained monotonic shearing not following S3 behaviour

C4: clay-like; clays show a very plastic stress-strain response (nearly constant shear stress after yield) for O_{CR} of 1–8, while sands show a range of strain softening to strain hardening behaviour that depended on the sand's relative density and confining stress

C5: clay-like; during the undrained cyclic stress-strain loops, the clays do not develop a very flat middle portion (where the shear stiffness is essentially zero) that is observed for sands after the excess pore pressure reaches a limiting value, which corresponds to the sample temporarily having zero effective stress ($r_u = 100\%$)

Figure 15. Boundaries for clay-like and sand-like behaviours (this study and others)

well-documented soils sourced from the literature summarised in Table 7, as well as KM35 ($P_I = 16\%$) and KM55 ($P_I = 15\%$) as included in Figure 15. Indeed, Figure 8 evidently shows that the apparent clay-like behaviours of KM35 and KM55 have parallel CSLs and NCLs.

As all the well-documented data points, including those from this study, can be simultaneously included within the newly postulated framework as shown in Figure 15, this study has been proven to be reliable, consistent and beneficial to engineers seeking a less expensive classification method for identifying silt behaviour.

Conclusions

Reconstituted kaolin samples of high-plasticity silts (MH) with varying clay contents were used to investigate the transitional behaviour of silts. Atterberg limit, dry sieving, hydrometer, CIU and oedometer tests were conducted on these reconstituted samples to characterise the behaviours of the samples in terms of clay-like or sand-like behaviours.

In the v - $\ln p'$ space derived from the CIU tests, the gradients of the respective CSLs seem to be approaching the same gradient of the NCLs when the P_I of the reconstituted kaolins increased progressively from KM20 ($P_I = 7\%$), KM25 ($P_I = 11\%$), KM35 ($P_I = 16\%$) to KM55 ($P_I = 15\%$). KM35 and KM55 demonstrate a clay-like behaviour with P_I values greater than 13% , while KM20 and KM25 exhibit a sand-like behaviour with P_I values smaller than 13% . Therefore, with the support of well-documented soils sourced from the 33 papers summarised in Table 7 and the research work carried out in this paper, thresholds for (a) sand-like and (b) clay-like silts are thus proposed as (a) $P_I \leq 13\%$ and (b) $P_I > 13\%$, respectively. Based on the newly postulated framework, all the well-documented data points from Table 7 as well as the MH data points from this research can be simultaneously included.

Based on the mentioned important findings of this study, if the values of Atterberg limits are the only available data, silts can then be characterised in terms of sand-like or clay-like behaviours by using the newly proposed framework before an appropriate

advanced soil model is selected to represent its behaviour. With this knowledge, for example, the modified Cam Clay model may not be wrongly applied to silts with a sand-like behaviour. The results obtained in this study show good reliability when compared to 65 sets of past significant experimental studies derived from 33 established past research papers.

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