ISSN 0853-5272

Volume I Number 2

November 2010

CIVIL ENGINEERING JOURNAL

MECHANICAL BEHAVIOR OF THE BENTONITE MIXED-KAOLIN AND SAND

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ABSTRACT

Understanding characteristics of soil mixtures lead to increasing the confidence level before applying such materials in the field. The outcomes of this study can provide insight into mechanical behaviour of a soil mixture between non-swelling and bentonite soil. Series of laboratory tests were conducted to evaluated index properties, compaction, strength, and swelling behaviour of the soil – bentonite mixtures. The result of this study indicated that the existence of bentonite in the soil mixtures influence the swelling behaviour, which follows hyperbolic curve model. Amount and size of non swelling fraction reduced the swelling and compressibility of expansive soils.

Keywords: expansive soil, bentonite, kaolin, sand, soil-mixtures, swelling, compressive strength

INTRODUCTION

In Yogyakarta, Muntohar and Hashim (2002) studied that roadway construction built on natural expansive clay soil experienced serious damage owing to uplift pressure generated from the swelling mechanism. As a result, the pavement deterioration was appeared along the roadway. In the field, many direct and indirect factors influence the swelling behaviour of the expansive soil such as water content, clay mineral and clay size content, density, and applied load. The soil can be very susceptible due to the water content. Physically, the expansive soils will be very stiff in dry condition and become very soft in wet. In general, swelling and shrinkage subject to wetting and drying of the expansive soils often caused detrimental, such as differential settlement and ground heaving. But recently, in the other hand, expansive soils can be beneficial in various geotechnical and engineering field. Expansive soils were attracting greater attention for designing soil mixtures in various applications such as back-filling, buffer materials for high-level nuclear waste, soils barrier for municipal waste landfill liner, and backfilled vertical cut-off walls (Yong et al., 1986; Alawaji, 1999a; Daniel and Wu, 1993).

Many experiments have been performed to investigate the swelling behaviour of soil mixtures. Commonly, the soil mixtures are blends of swelling with non-swelling soils (Misra et al., 1999; Sivapullaiah et al., 1996). Stewart et al. (2003) mentioned the sand-bentonite mixtures as bentonite-enhanced sand (BES). Thus, further understanding of mechanism controlling swelling of soil mixtures lead to increasing the confidence level

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before applying such materials in the field. In this context, this article gives an account of experimental investigation of the influence of amount of non swelling fraction such as kaolin and sand fraction on the swelling characteristics of expansive soils. Understanding of the soil mixture on the index properties, compaction, swelling – compressibility and strength behaviour of kaolin – bentonite and sand – bentonite mixtures are discussed in this paper. The significant of this study can be extended to provide insight into the behaviour of soil mixtures in such used as backfill materials or liner materials in waste disposal scheme.

EXPERIMENTAL DESIGN

The research area was including indirect and direct measurement of swelling potential. The consistency limits and clay size content were used to estimate the range of swelling potential for the various engineered soil-mixtures. Direct measurement of swelling potential and pressure were performed by using fixed-ring oedometer for selected soil samples. The indirect methods correlate the swell potential with combinations of consistency limits and clay content, whereas, the direct measurement is conducted using the oedometer tests. At the first stage of the research, the various soil mixtures are determined. The design of laboratory tests in this study is presented in Table 1. The bentonite content was varied from 5% to 80% to represent a lower to very high expansive soils.

Table 1 Laboratory test scheme

Soil Mixtures	Designed	PSD	ATT	LS	PS	UCS	Swelling
Bentonite	HQB	●	●	●	○	●	○
Kaolin	KAO	●	●	●	●	●	○
Mining Sand	MS	●	○	○	○	●	○
5% HQB + 95% Kaolin	KB1	●	●	●	●	●	●
10% HQB + 90% Kaolin	KB2	●	●	●	●	●	●
20% HQB + 80% Kaolin	KB3	●	●	●	●	●	●
30% HQB + 70% Kaolin	KB4	●	●	●	●	●	●
50% HQB + 50% Kaolin	KB5	○	●	○	○	○	○
80% HQB + 20% Kaolin	KB6	○	●	○	○	○	○
10% HQB + 90% Sand	SB1	●	●	●	●	●	●
30% HQB + 70% Sand	SB2	●	●	●	●	●	●
50% HQB + 50% Sand	SB3	●	●	●	●	●	●

Note: ● = tested; ○ = not-tested; HQB = High Quality Bentonite; PSD = particle size

The Soil

The soils used in this study are commercially Wyoming bentonite, Malaysian kaolin, and mining sand. The particle size distributions of the soils are shown in Figure 1, and the basic properties are presented in Table 2. The average particle diameter of bentonite, kaolin, and mining-sand specimen were about 0.6 μm, 4.1 μm, and 820 μm respectively. The bentonite and kaolin specimen were predominantly comprised of clay and silt. The plasticity of bentonite was very high (262%) but the plasticity of kaolin was quite low that is about 12% of the bentonite. According to British Soil Classification System (BSCS) BS 5930-1999, the bentonite can be classified as clay of extremely high plasticity which is symbolized with **CE**, while the kaolin is categorised as silt of very high plasticity (**MV**). The sand used was comprised of 36% coarse, 24% medium, and 4% silt size, and then can be classified as well-graded sand (**SW**).

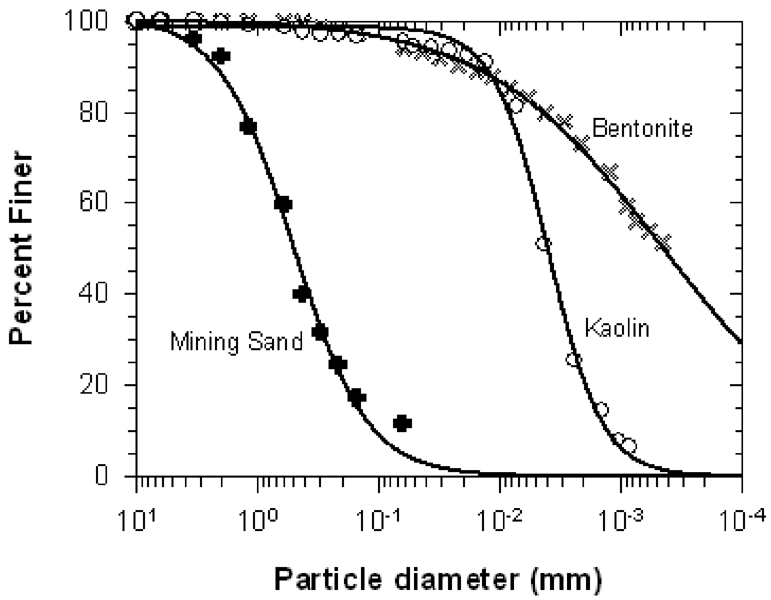

Figure 1 Grain size distribution of the bentonite, kaolin and mining sand

Table 2 Index properties of the soil used

Properties	Bentonite	Kaolin (KAO)	Mining Sand
Specific gravity of solid, Gs	2.79	2.53	2.15
Soil fraction:			
Gravel (%)	-	-	24
Sand size (%)	6	4	72
Silt size (%)	21	76	4
Clay size (%)	73	20	-
Liquid limit, LL (%)	307	72	-
Plastic limit, PL (%)	45	40	-
Plasticity Index, PI (%)	262	32	-
Linear shrinkage (%)	17	7	-
Uniformity coefficient, Cu	-	-	9.3
Curvature coefficient, Cc	-	-	1.2
Average particle diameter, D_{50}	0.6	4.1	820
Activity, A = PI/Clay	3.6	1.6	-

The soil minerals have been determined by X-ray diffraction test as shown in Figure 2 for bentonite and kaolin specimens. The bentonite was comprised primarily of montmorillonite minerals. In Figure 2a, the montmorillonite mineral was found strongly in bentonite at basalt spacing of 5.75 Å, 17.25 Å, and 19.98 Å. Quartz and illite mineral was strongly appeared at basalt spacing of 3.36 Å and 3.19 Å respectively. The quartz was also found at 4.28 Å, 2.46 Å, and 2.29 Å in the low intensities. Mixed layer between illite and montmorillonite was found at 4.52 Å of basalt spacing. The kaolin specimen was detected consist of mainly kaolinite minerals which is obviously at peak 12.7 Å, 24 Å, and 38.43 Å (Figure 2b). The other mineral, illite was also detected interlayer with montmorillonite and kaolinite at 5.01 Å, 4.46 Å and 4.37 Å. The micalike clay minerals, phlogopite (brown mica) was observed either at 8.89^{o}.

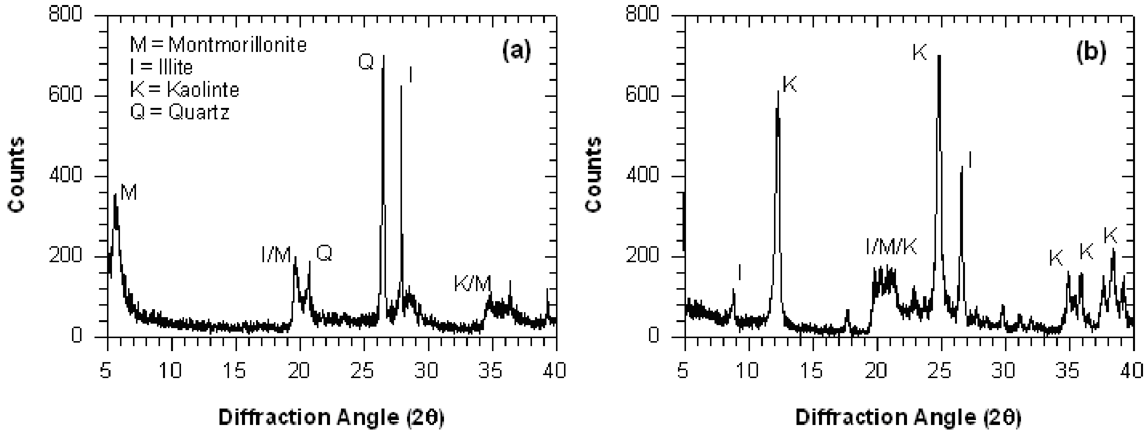

Figure 2 X-Ray diffraction result (a) bentonite, (b) kaolin

Laboratory Test Programe

A series laboratory experiments are designed to investigate various soil mixtures as presented in Table 1. The soils were oven dried and sieved through 425 μm for preparing liquid limit, plastic limit, and linear shrinkage tests. Cone penetrometer method was used to determine the liquid limit. Particle size distribution of the soil specimens was determined with sedimentation and sieving test. Sedimentation test was used to define colloidal particle less than 63 μm while dry sieving was employed to determine particle size retained on the 63 μm sieve. The consistency limit test, linear shrinkage, and particle size distribution test was performed according to the standard presented in BS 1377: Part 2 – 1990. The compaction test was according to BS 1377: Part 4 – 1990. Proctor standard compaction was examined for determination of the compaction characteristics of soil mixtures. Water were added to the soil mixtures and compacted until maximum dry density (MDD) has been reached. The optimum moisture content (OMC) was defined at the MDD. Unconfined compressive strength was used to observe the strength. The test was performed according to BS 1377: Part 7 – 1990. The specimen of 50 mm diameter and 100 mm height were compacted by static compaction method. The desired amount of soil was placed into cylindrical mould and then compressed on the hydraulic jack. Then, the specimen was extruded and mounted in the compressive machine.

The swelling behaviour was determined according to ASTM D4546-03. A conventional oedometer apparatus was used for determination of the swelling and compressibility of soil mixtures. Required quantities of soil mixtures, at optimum moisture content, were transferred to consolidation ring of 50 mm internal diameter and 20 mm height. All the soil mixtures were compacted statically to the same maximum dry density that is 1.70±0.05 Mg/m^3. The specimen was positioned in the loading frame with a seating load of 7 kPa. The soil samples were then inundated with distilled water and allowed to swell until they reached equilibrium values of swelling. At this point a standard consolidation test is conducted by applying incremental loads starting with 14 kPa and ending with 1600 kPa.

RESULTS AND DISCUSSIONS

Particle Size Distribution and Conssitency Limits

The particles size distribution of soil mixtures and consistency limits are presented in Table 3. Soil mixed with bentonite showed various silt and clay content. There was different amount of soil fraction between the measured and theoretical or calculated (the calculated soil fraction was typed in bracket as presented in Table 3). For the instances is for SB1 mix (10% bentonite – 90% mining sand), the amount of sand fraction from bentonite is $0.1 \times 5\%$, and sand fraction from mining sand is $0.9 \times 72.2\%$. Thus, the theoretical amount of sand fraction is summation of sand fraction from bentonite and mining sand that is 65.7%. However, the amount of sand fraction from laboratory test was 72.2% which differs with the calculated. This discrepancy was attributable to the physicochemical interaction between bentonite and water. This tendency can be enlightened using physicochemical interaction between clay content as indicated by normalized consistency limits. This behaviour was explained by Mitchell (1993) and Al-Shayea (2001). The second reason, sedimentation test in particle size distribution test assumed that the shapes of soil particles are spherical according to Stoke's law. In fact, the particles of silt and clay size were not spherical (Fukuda and Suwa, 2001).

Consistency limits are useful in geotechnical engineering for identification, description, and classification of soils, and as a basis for preliminary assessment of mechanical properties of soils. Liquid limit and plastic limit are also valuable indicators to know the degree of expansion and strength of soils (Grim, 1968). In general, consistency limit is dependence on fines fraction content in a soil. Clay content in soil mixture affect to increases the LL, PL, and PI almost linearly respect to clay content. A greater quantity of clay size content in a soil will result in soils exhibiting higher plasticity and the greater potential for shrinkage and swelling. The potential expansiveness of the soil – bentonite mixtures is shown in Figure 3. The range of soil swelling potential in Figure 3 is originally proposed by Van der Merwe (1964). The figure illustrated the plasticity index as a function of liquid limit and clay content which is divided into different zones. The sand-bentonite mixture, KB5 (50% bentonite – 50% kaolin) and KB6 (80% bentonite – 20% kaolin) mixes are shown to lie on the line of A = 2.0. While, the kaolin – bentonite mixtures are closed to line of A = 1.5 in which corresponds to a pattern similar to Ca-montmorillinite.

The values of liquid limit and plasticity index of the soil mixtures are also plotted on modified plasticity chart as shown in Figure 4. The chart was proposed by Dakshanamurthy and Raman (1973). The A-line is a line that distinguishes clay size where lie above the line, and silt size where fall below the line. The soil – bentonite mixtures were mostly fall below the A-line except the soil mixtures that containing 50%

and more bentonite (e.g. KB5, KB6, and SB3). Hence, the studied soil – bentonite mixtures were mostly classified as silt size soils. Commonly, a soil has high plasticity behaviour if the liquid limit is greater than 50% (LL > 50%).

Table 3 Soil fraction and Consistency limit of the Soil Mixtures

Designed	Soil Fraction (%)			Consistency Limits (%)			LS (%)	BSCS Symbol*
	Sand	Silt	Clay	PL	LL	PI		
KB1	6.6 (4.3)	63.4 (72.9)	30 (22.6)	37.6	80.1	42.5	7.7	MV
KB2	2.4 (4.4)	70.7 (70.2)	26.9 (25.2)	46.0	85.9	39.9	12.3	MV
KB3	4.4 (4.5)	63.1 (64.8)	32.5 (30.6)	40.7	90.7	50.1	13.8	ME
KB4	1.1 (4.6)	59.9 (59.4)	39 (35.9)	42.8	99.5	56.7	14	ME
KB5	- (4.8)	- (48.5)	- (46.5)	40.2	145.5	105.3	-	CE
KB6	- (5.2)	- (32.3)	- (62.5)	35.3	174.1	138.8	-	CE
SB1	72.2 (65.7)	23.8 (5.7)	4.0 (7.3)	21.1	42.9	21.8	2.4	SMI
SB2	57.7 (51.9)	20.6 (9.1)	21.7 (21.9)	27.2	85.1	57.9	4.3	CVS
SB3	42.5 (38.5)	10.5 (12.5)	47.0 (36.5)	43.2	138.3	95.1	8.8	CES

Note: * BS 5930-1999, Number in bracket ( ) is the theoretical amount of soil fraction. LL = liquid limit, PL = plastic limit, PI = plasticity index, LS = linear shrinkage, MV = silt of very high plasticity, ME = silt of extra high plasticity, CE = clay of extra high plasticity, SMI = very silty-sand of intermediate plasticity, CVS = sandy-clay of very high plasticity, CES = sandy-clay of extremely high-plasticity

Dakhanamurthy and Raman (1973) link the threshold value of high swelling behaviour corresponds to a value of liquid limit, LL = 50% as shown in figure 4. The figure reveals that 90% sand – 10% bentonite mixtures falls in the region of low expansive potential. The other tested mixtures are identified as having high to very high-plasticity. They tend to have a high potential of expansiveness. According to the figure, 5% bentonite in the kaolin – bentonite mixtures (KB1 mixes) exhibited high-plasticity behaviour (LL ≥ 50%). But for sand – bentonite mixtures, high-plasticity soil will be obtained at 21.7% clay in which correspond to SB2 mixes. The kaolin – bentonite mixtures are classified as elastic silt and heavy clay that has low to high soil plasticity. Sand – bentonite mixtures are basically comprised of fines sand and clay fraction, which is classified into sandy silt and sandy clay. Since the clay fraction is swelling clay (bentonite), the sand – bentonite mixtures is possible to have high-plasticity. Based on the chart given in Figure 3 and 4, the studied soil – bentonite mixtures can be classified into three groups that are soil with low to medium swelling potential (i.e. SB1), high to very high (i.e. KB1, KB2, KB3, and SB2) and very high to extra high (i.e. KB4, KB5, KB6, and SB3).

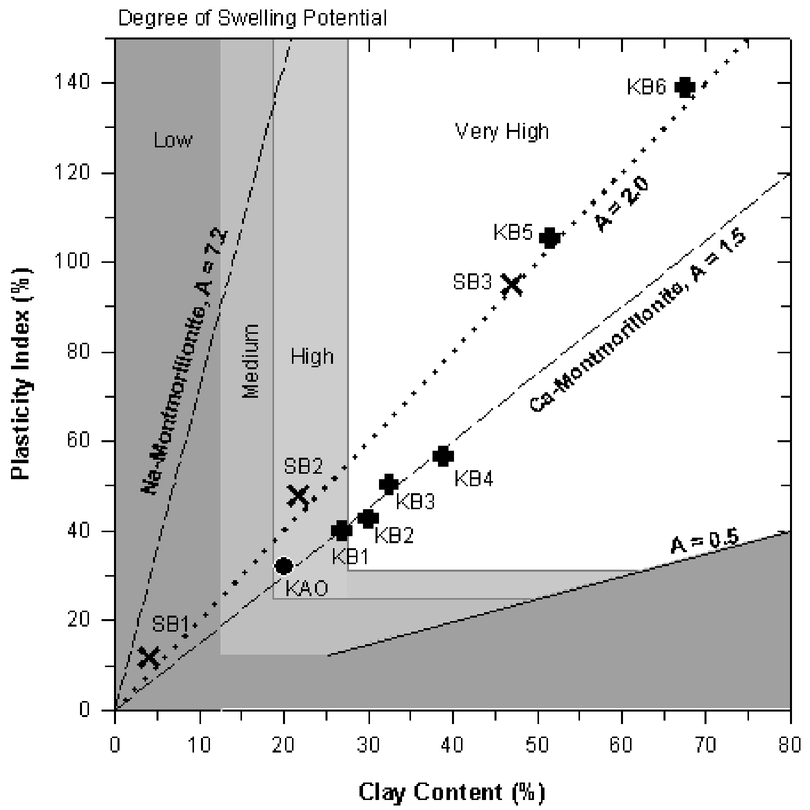

Figure 3 Potential expansiveness of soil mixtures.

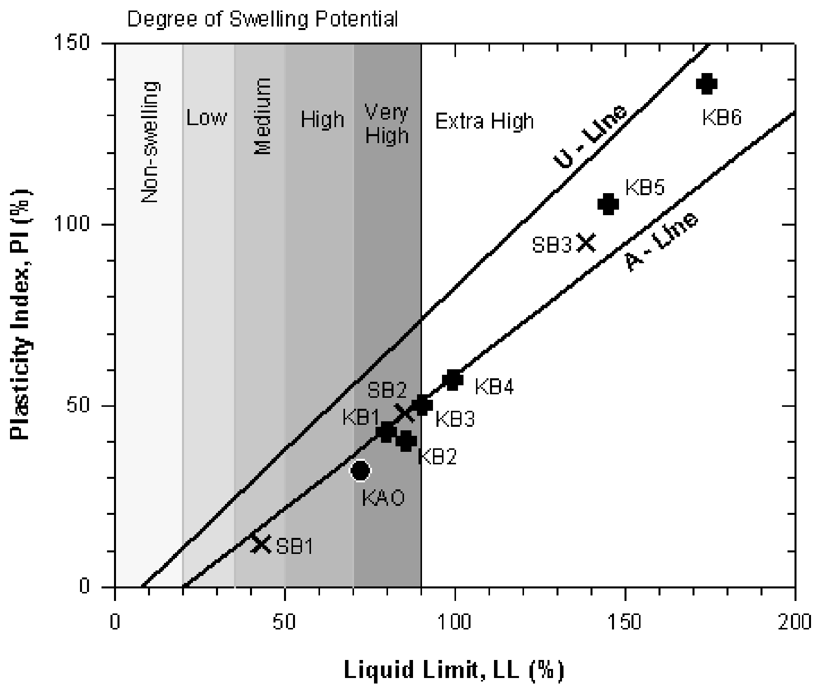

Figure 4 Plasticity chart for measuring degree of swelling potential

Linear Shrinkage

The tendency for expansive clays to swell and shrink is a major cause of damage to structure infrastructures. Shrinkage caused by loss of moisture content from soils either during initial curing or following later moisture movements, is one of the causes of cracking in common cases. The linear shrinkage of the sand – bentonite mixtures is presented in Table 3. Linear shrinkage of the soil was expressed as percentage of the original length specimen, L_o (in mm), which is obtained from equation (1):

$$LS = \left(1 - \frac{L_D}{L_o}\right) \times 100 \qquad (1)$$

where L_D is the length of the oven-dry specimen (in mm).

The linear shrinkage increases corresponding to the amount of clay fraction in the soils. The shrinkage of clay – clay mixture (kaolin – bentonite mixtures) shows greater than sand – bentonite mixtures for the same percentage of bentonite. This is attributably by the presence of coarser fraction and capability of water absorption. Sridaharan and Prakash (2000) introduce that shrinkage of soils is primarily a function of the relative grain size distribution of the soil, irrespective of the principal clay mineral of the soil and that the shrinkage limit does not depend on plasticity characteristics of soil. Furthermore, addition of non-cohesive soil fraction (fine and medium sand) result in lower shrinkage limits. For the systems having the same grain size distribution, the one that has higher shearing resistance at the particle level will shrink lesser.

Compaction Characteristics

Proctor standard compaction of the soil mixtures is illustrated Figure 5. In general, the compaction characteristics are affected by the soil type and the composition of soil particles. Figures 5a shows that there was significant change in water content and maximum dry density for the bentonite content lesser than 30% in the kaolin – bentonite mixtures. In contrary with the sand – bentonite mixtures, addition of bentonite from 10% to 30% exhibited to increase significantly the maximum dry density from 1.60 Mg/m^3 to 1.65 Mg/m^3. These characteristics revealed that clay in mixtures with 30% bentonite did not cause definite structure and act as binding agent between the particles leading to increase in maximum dry density. In contrary with sand – bentonite mixtures, the MDD decreases corresponding to addition of bentonite. For a larger bentonite content (30% and 50%), the swelling of clays become more dominant. The clay begins to fill the voids between sand leading to replace sand particles. Concomitantly, the volume of voids reduces and as a consequence the void ratio (e) increases. As a result, the maximum dry density of the sand – bentonite mixes decreased as low as 1.60 Mg/m^3. Similar behaviour was also observed by Alawaji (1999b) and Al-Shayea (2001) for sand – bentonite mixtures.

Swelling – Time Characteristics

Figure 6 shows typical time – swell relationship for various kaolin – bentonite and sand – bentonite mixes respectively. The swell is expressed as a percentage increase in height. Several attempts have been established by earlier researchers to obtain time – swell relationships for expansive soils. Some progresses have been made towards characterizing swelling characteristics, despite the complexity of the behaviour. Seed et al. (1962) report that the time required for completion of swelling is time consuming. It has been observed that the swelling and time relationship was the S-shape curve that comprised of the initial, primary and secondary swelling stage. When the water uptake, the swelling with log time increase slowly at the beginning, increases steeply, and then reaches an asymptotic value as illustrate in Figure 6. The time required to reach an asymptotic value varies considerably, depending upon the percentage of the bentonite content. The maximum amount of swelling generally increases with increasing bentonite content. The maximum swelling and swelling behaviour can be predicted using a hyperbolic model (Al-Mukhtar et al, 1999).

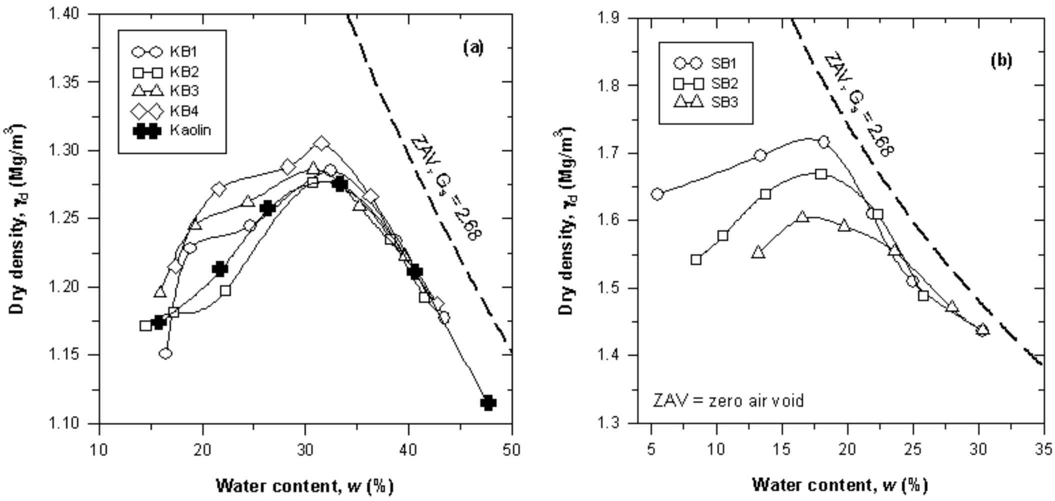

Figure 5 Proctor standard compaction of the (a) kaolin – bentonite mixtures, (b) sand – bentonite mixtures

Considerable differences exist in the nature of time-swell relationship between kaolin – bentonite mixes and sand – bentonite mixes. Before water uptake, the voids of the soil – mixtures are occupied by air and free water. After water uptake, at initial stage, the kaolin – bentonite specimens experienced to swell greater than the sand – bentonite specimens. It is because of the ability of the soil - mixtures to absorb the water during water uptake. For the sand – bentonite mixtures, during water uptake, the volume of montmorillonite increased by absorbing the water and the voids in soil matrix were filled with the swollen bentonite. At low bentonite content, the rate of swelling is very slow but increases gradually with decrease in the particle size of the non swelling fraction. The swelling seems to be ceased after 10 days. Initially, the swelling is generally less than 10% of the total swelling. This is essentially due to swelling of the bentonite clay particles within the voids of the coarser non-swelling fraction. Primary swelling develops when the void can no longer accommodate further clay particle swelling. It occurs at a faster rate. After the primary swelling was completed, slow continued swelling occurs (secondary swelling). It is observed that the end of primary swelling of kaolin-bentonite mixtures are varies within 200 – 1000 minutes. In general, the time needed for completion the primary swelling increases in relation with bentonite content. But, this appearance is not clear in the sand-bentonite mixtures. When a small

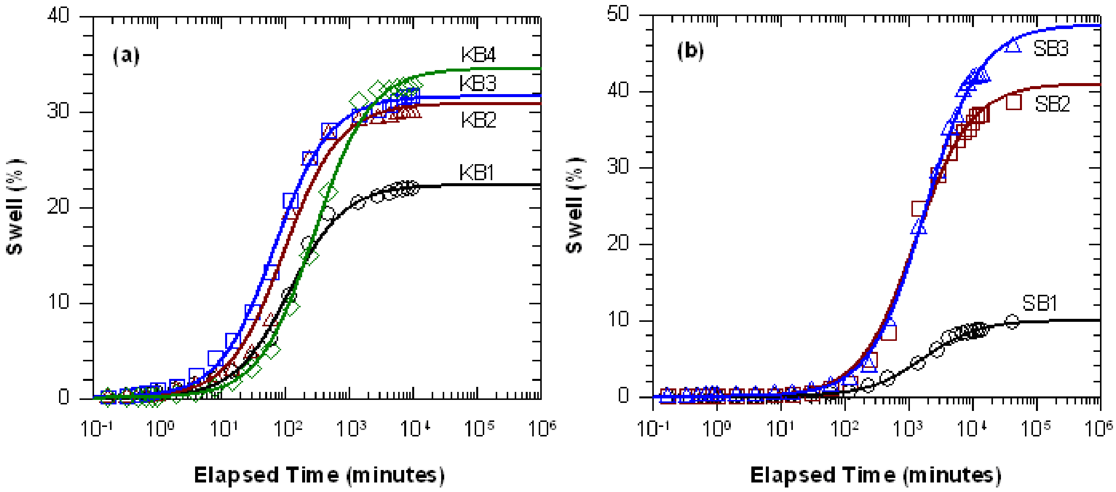

Figure 6 Swelling and elapsed time relationship
(Note: the symbols are measured swelling, the solid lines are the hyperbolic model)

portion of bentonite is added to kaolin and sand, the bentonite can accommodate itself within the voids. In the case of sand – bentonite mixtures, because of the size of the bigger voids, even the swollen bentonite can still be accommodated within the voids without causing any primary swelling. The mechanism is called inter-void swelling (Sivapullaiah et al., 1996; Komine and Ogata, 1994). With smaller soil size fraction, kaolin, the inter-void swelling may not occur and the primary swelling starts immediately.

Figure 7 illustrates a correlation between swelling potential and plasticity index of the present study and compares the results with the published prediction equation which was proposed by Holtz and Gibbs (1956), Seed et al. (1962), and Chen (1982). The best-fit correlation between swelling potential and clay size content of the investigated soil-mixtures were plotted as solid line in figure 7. The best-fit line was close to the result obtained by Seed et al. (1962) and Chen (1982) which determined the swelling potential under a 6.89 kPa (1 psi) of seating pressure. Seed et al. (1962) used clay and sand compacted at optimum moisture content using AASHTO standard for compaction. The correlation obtained by Chen (1982) was based on the various natural soils compacted to a dry unit weight ranging from 15 kN/m^3 to 17kN/m^3 at water content of 15% to 20%.

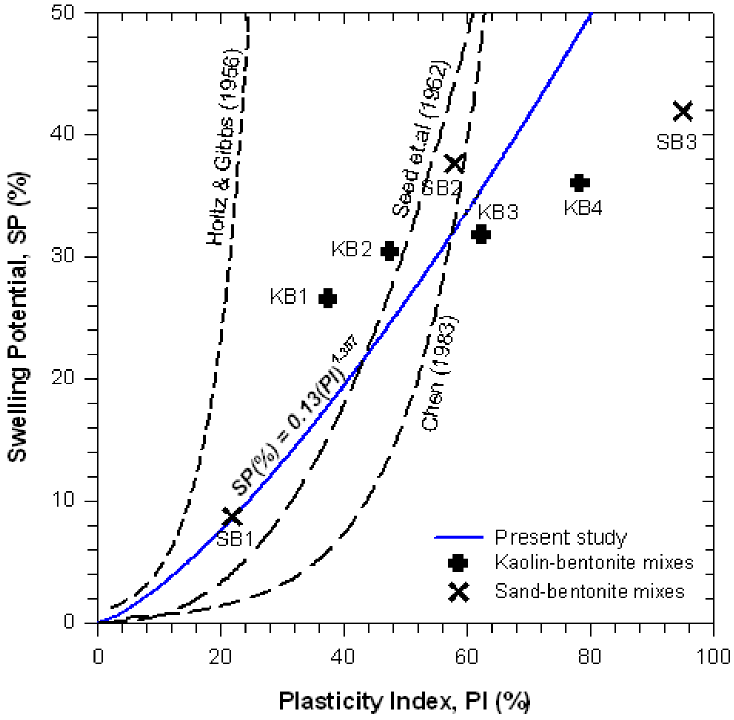

Figure 7 Correlation between plasticity index and swelling potential

Swelling – Compressibility and Swelling Pressure

Figure 8 illustrates the typical swelling in the form of percentage of swell versus log pressure which is determined from oedometer test. The swell is expressed as a percentage of the increase in sample height. The pressure required to revert the specimen to its initial height was determined as the swelling pressure. At least two essential values can be attracted from these results that are compressibility and swelling pressure. The virgin compressibility is defined as the linear part of the axial strain (ε_v) – log pressure (*p*) curve under virgin loading path (Alawaji, 1999b). For soil mixtures containing bentonite up to 10% (i.e. KB1 and SB1 specimens), the soils start to experienced quick compression at a vertical pressure of 10 kPa. However, in general, compressibility characteristics of the soils – bentonite mixtures were not clearly evident up to vertical pressure of 25 kPa. Beyond a pressure of 25 kPa, the soil – bentonite mixtures initiated to experience highly-compressible. The coefficient of compressibility (C_{ε_v}) is then defined as the slope of linearised portion of the virgin compression curve and calculated with Equation (2).

$$C\varepsilon_v = \frac{\Delta\varepsilon_v}{\Delta\, log(\ p\ )} \tag{2}$$

where $\Delta\varepsilon_v$ is difference between observed axial strain at the linear path and $\Delta log\ (p)$ is difference between given pressure at observed axial strain. The results have been tabulated in Table 4. It is noted that the compressibility, which is specified with coefficient of compressibility, increase associate with increase in bentonite content.

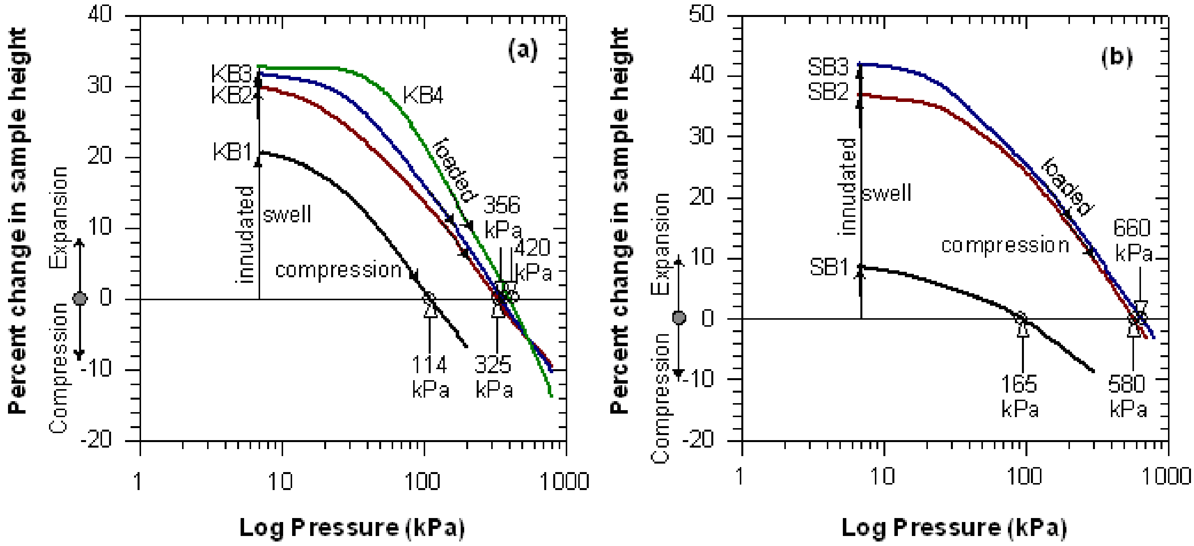

Figure 8 Swelling and log pressure relationship (a) kaolin – bentonite mixtures, (b) sand-bentonite mixtures.

Table 4 Swelling potential, swelling pressure, and coefficient of compressibility

Soil Mixes	Swelling Potential (%)	Swelling Pressure (kPa)	$C_{\varepsilon v}$ (x 10^{-2})
KB1	21.92	114	26.99
KB2	29.96	325	25.58
KB3	31.75	356	29.14
KB4	32.65	420	39.36
SB1	8.68	165	7.50
SB2	36.96	580	32.38
SB3	41.96	660	31.48

Note: C_{ε_v} = coefficient of compressibility

Figure 8 shows that the swell pressure and compressibility increases with increasing of the bentonite fraction or clay size content. Sand – bentonite mixtures exhibit greater swelling pressure than kaolin – bentonite mixtures for the same bentonite content except 10% bentonite content. It can be caused by the greater initial dry density and lower water content when the specimen was compacted. This is in agreement with El-Sohby and Rabba (1981). Mathew and Rao (1997) indicated that by increasing the valence of exchangeable cations in homoionized clay, the overall compression in the system reduces and the pre-consolidation pressure increases. The equilibrium void ratio at any applied pressure is a direct function of the repulsive forces arising from the interaction of adjacent diffuse double layers and pore fluid. As the valence of exchangeable cations in the clay increases, there is a reduction in the diffuse double –layer thickness and in the magnitude of the repulsive forces. These, finally, result in a lower equilibrium void ratio at any given pressure until higher pressures are reached. It is evident that there is a correlation between compressibility and swelling pressure as observed by Sridharan and Choudhury (2002).

Unconfined Compressive Strength

The axial stress and strain relationship that obtained from unconfined compressive strength test is shown in Figure 9. It was observed that the compacted mining sand has lowest compressive strength and smaller axial strain among the investigated soil (Figure 9a). While the compacted bentonite soil shows the highest compressive strength and longer axial strain. Mining sand is cohesionless soil while the bentonite and kaolin are predominantly clay mineral that is cohesive soils. This characteristic indicates that cohesiveness of the soil play an important role to retain the integrity of the soil structure from the applied load. Mixing the bentonite with kaolin produces a compressive strength characteristic that closer to bentonite soil as shown in Figure 9b. The behaviour of kaolin – bentonite mixtures exhibit a stiffer and brittle material when compacted at optimum moisture content and maximum dry density. In contrast with sand – bentonite mixtures, in general the compressive strength was lower than the kaolin – bentonite mixes. However, the sand – bentonite mixes exhibit a semi plastic to plastic material.

The change of compressive strength for soil – bentonite mixtures are shown in Figure 10. In general, bentonite contributes to enhance the strength of kaolin and sand. It was revealed in figure 10, Mixing 5% bentonite with kaolin enhanced the compressive strength about 56% from 160 kPa to 250 kPa. The compressive strength of kaolin – bentonite mixes increases slightly when the bentonite content was blended more than 30%. However, bentonite enhanced the compressive strength of sand significantly. The unconfined compressive strength increased almost linearly with increasing the amount of bentonite in sand – bentonite mixtures. This behaviour explained that increasing of sand fraction yield diminishing the cohesiveness of the soil mixtures. As a result, the soil lost its ability to retain the integrity of soil structure. The void in sand – bentonite matrix is possible to be larger with increasing of the sand fraction and resulted in decreasing of compressive strength. The results obtained from this study showed that the compressive strength was not proportional to the bentonite content solely. It has been shown that for the same percent of bentonite content, the compressive strength of sand – bentonite mixes were low. It indicated that the shape of non-swelling fraction influence in achieving compressive strength in soil – bentonite mixtures.

If the soil – bentonite mixtures will be applied as an isolation barrier used in waste containment a system, the soil mixtures are supposed to sustain certain amount of static load exerted by the overlying waste materials. In this regard, the barrier material must have adequate strength for stability. The bearing stress acts on the barrier system depends on the height of landfill and the unit weight of waste. Thus, to date, the minimum required strength of soil used for compacted soil liners is not specified. Daniel and Wu (1993) mentioned that soil used as barrier material should have minimum unconfined compression strength of 200 kPa. Test result shown in figure 10 yield that the kaolin – bentonite mixtures soil possesses higher strength than the recommended minimum strength of 200 kPa. But for sand – bentonite mixtures, based on the best-fit line (dashed-line in Figure 10), the amount of bentonite beyond 60% will result the recommended minimum strength as proposed by Daniel and Wu (1993).

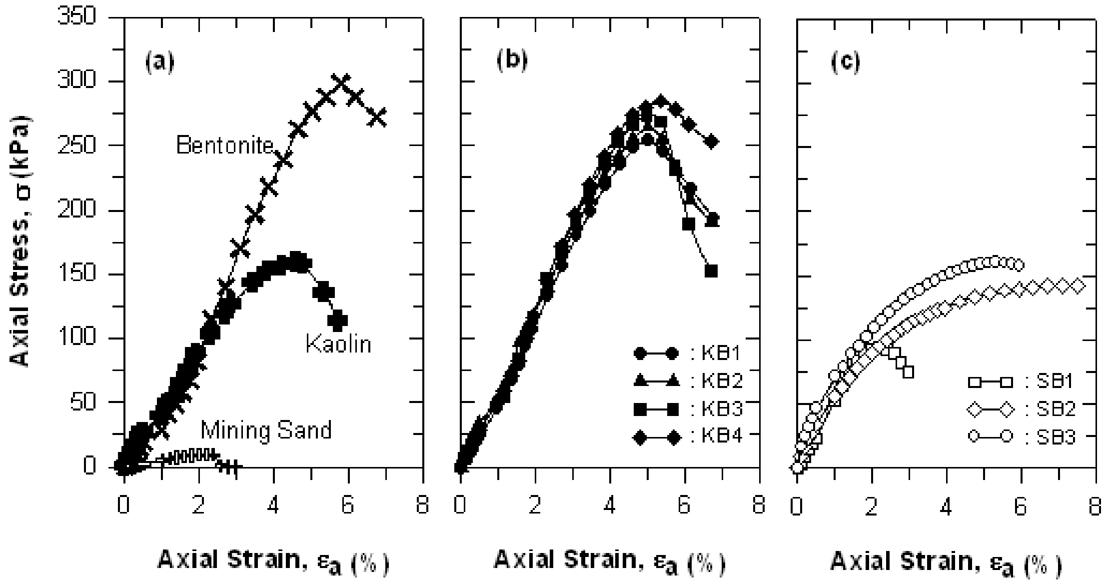

Figure 9 Axial stress and strain relationship (a) bentonite, kaolin, and mining-sand, (b) kaolin – bentonite mixes (c) sand – bentonite mixes

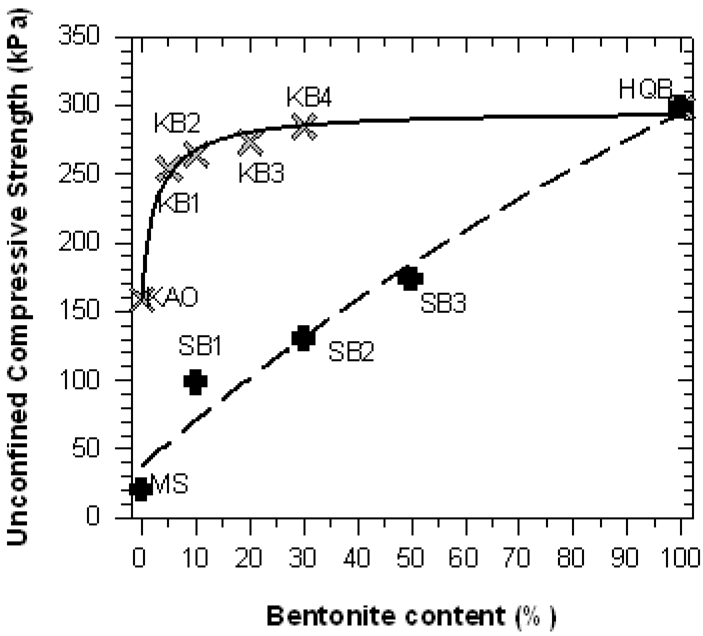

Figure 10 Unconfined compressive strength of the soil – bentonite mixtures

CONCLUSIONS

The synthetic soil – bentonite mixes are widely used as waste containment barriers, etc. The study of soil – bentonite mixtures was undertaken successfully and showing significant results to have understanding about the index and mechanical properties of the investigated soils. Based on the study undertaken, thus far some pointers of the research can be written as follows:

1. Clay fraction of the soil mixtures increases linearly with bentonite percentage. Consistency limits of the soil mixtures increase as the quantity of the clay increases. The swelling soil exhibits high expansion at the threshold percentage 20 % bentonite.
2. The optimum moisture content and maximum dry density of soil mixtures revealed slightly reduces when the bentonite increased.
3. The swelling and swelling pressure of bentonite mixed kaolin and sand, generally, increases correspond to increase in bentonite percentage. The magnitude depends on the consistency of soil, clay fraction, the amount and type of non-swelling soils.
4. The strength of soil – bentonite mixtures increases with bentonite percentage. The magnitude of compressive strength was affected by the type of non-swelling soils.

ACKNOWLEDGEMENT

Author is thankful to the Roslan Hashim, Professor at Department of Civil Engineering, University of Malaya for his discussion. The research is part of research grant supported by Institute of Research, University of Malaya through Fundamental Research Vot-F 2002/B and counterpart funding from Universitas Muhammadiyah Yogyakarta.

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