京都大学防災研究所年報　第44号 B-2　平成13年4月
Annuals of Disas. Prev. Res. Inst., Kyoto Univ., No.44 B-2, 2001

Suitability Assessment of Two Landfill Clay Liners

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Synopsis

In this study, falling head permeability, Water retentivity, water migration, direct shear and consolidated undrained triaxil compression tests were performed on two landfill clay liner specimens (clay-bentonite and sand-bentonite) to evaluate the permeability, water migration and shear strength properties. The slope stability of a typical canyon solid waste landfill relating to the two clay liners under various conditions was analyzed with the shear strength parameters obtained from the above tests. The suitability of clay-bentonite and sand-bentonite mixtures as landfill bottom liners was assessed from the viewpoint of laboratory permeability tests and the landfill stability.

Keywords: landfill stability; clay liner; permeability test; water retentivity test; water migration test; direct shear test; consolidated undrained triaxial compression test

1. Introduction

In Japan, many solid waste landfill sites are located in canyon areas. Compacted clay liners (CCLs) are becoming increasingly used to prevent leachate from polluting the environment. However, constructing liners in canyon-type landfills, two most important issues should be concerned. One is whether the hydraulic conductivity of liners is low enough to prevent the leachate percolating. The other is whether the shear strength of liners under various conditions is high enough to resist sliding. In particular, the later one has not been focused on significantly in Japan. In previous reports, however, several large-scale landfill failures have resulted in great damage. Most of those failures have occurred fully or partially along bottom liners (Koerner and Soong 2000). For example, Kettleman Hills waste landfill slope failure was due to the sliding along interfaces within the composite multilayered bottom liner with the geosynthetics and the CCL. The trigger for this failure was reported to be the excessive wetness of the clay component in the liner system (Mitchell et al. 1990, Seed et al. 1990). Generally, the geological conditions in canyons is complicated, especially groundwater level changes greatly due to the precipitation. However, high groundwater level might increase the water contents of CCLs, resulting in the reduction in the shear strength of CCLs, and consequently affect the landfill stability (Kamon et al. 2000, 2001). In addition, the presence of various chemical substances in leachate such as inorganic salts, acids and base might affect the shear strength properties of liners. Therefore, it is necessary to evaluate the properties of CCLs mentioned above under various conditions prior to their constructions.

2. Experimental

2.1 Materials

Na-bentonite, Toyoura sand and Fukakusa clay were used to form bentonite mixtures in the experiments. Basic properties of these materials were shown in Table 1. Mixing ratio of the bentonite in this study was 15% in dry weight, which was determined according to the previous study that the hydraulic conductivity of bentonite mixtures exhibits a lowest value when the mixing ratio of the bentonite reaches 15% by dry weight, and the increase in the bentonite ratio larger than 15% does not lower the hydraulic conductivity significantly (Imamura 1996). Main properties of the bentonite mixtures are shown in Table 2, and standard Proctor compaction curves of these materials are shown in Fig. 1.

2.2 Falling head permeability test

Falling head permeability test was conducted on clay-bentonite and sand-bentonite specimens with the apparatus shown in Fig. 2. Specimens of bentonite mixtures with 6 cm in diameter and 2 cm in height were compacted at water contents of 28.4, 31.0, 32.8 and 34.9% for clay-bentonite, and w_{opt} for sand-bentonite. Rubber membrane was placed around the specimen and sealed at the cap and base with O-rings. Cell pressure of 30 kPa was applied in order to resist the swelling pressure of the bentonite mixtures (Imamura 1996) and to prevent water percolating through the gap between the specimen and the rubber membrane.

Figure 3 shows the hydraulic conductivities of the bentonite mixtures with the pore volume of flow. The hydraulic conductivity of clay-bentonite is in the range from 6.0×10^{-9} to 3.0×10^{-8} cm/s, and that of sand-bentonite is in the range from 1.0×10^{-9} to 3.0×10^{-9} cm/s. With reference to the Japanese standard for the landfill bottom clay liner that the thickness of clay liners is larger than 50 cm and the hydraulic conductivity is lower than 1.0×10^{-6} cm/s, the hydraulic conductivity of both clay-bentonite and sand-bentonite are much lower. Therefore, from the viewpoint of the hydraulic performance, these two bentonite mixtures are suitable for the landfill bottom liner materials.

Table 1 Properties of materials

Na-bentonite	
Liquid limit = 616%	
Plastic limit = 61%	
Water content = 11.3%	
Bulk density = 0.7 g/cm^3	
SiO_2: 69.9%	CaO: 0.95%
TiO_2: 0.15%	Na_2O: 2.13%
Al_2O_3: 15.9%	K_2O: 0.72%
Fe_2O_3: 2.93%	MgO: 1.64%
Ignition loss: 4.80%	
Fukakusa clay	
Liquid limit = 55.5%	
Plastic limit = 26.6%	
Particle density =2.68 g/cm^3	
Maximum dry density = 1.46 g/cm^3	
Optimum water content = 29.3%	
Toyoura sand	
Particle density = 2.64 g/cm^3	
Grain size: D_{10} = 0.11 mm	
D_{90} = 0.25 mm	
Maximum dry density = 1.58 g/cm^3	
Optimum water content = 7.5%	

Table 2 Properties of bentonite mixtures

Bentonite mixtures	Sand-bentonite (SB)	Clay-bentonite (CB)
Mixing ratio (in dry weight)	Na-bentonite: 15% Toyoura sand: 100%	Na-bentonite: 15% Fukakusa clay: 100%
ρ_{dmax} (g/cm^3)	1.68	1.42
w_{opt} (%)	17	31

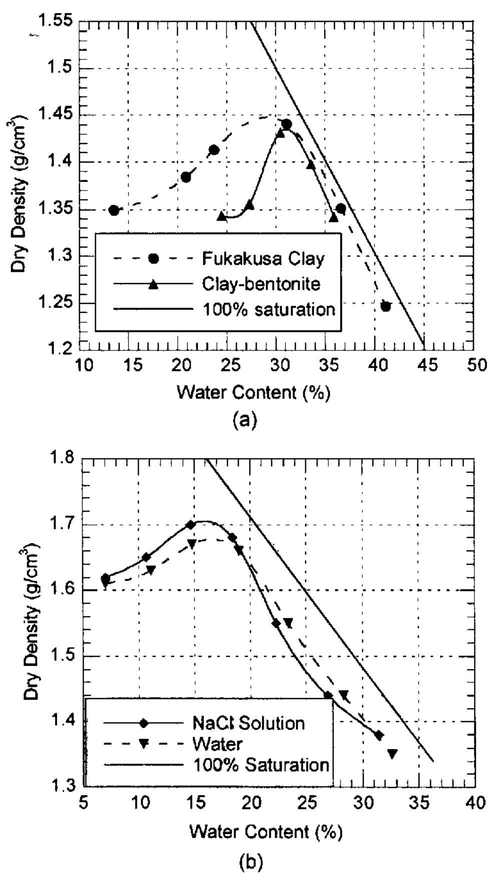


Fig. 1 Compaction curves for (a) Fukakusa clay and clay-bentonite, and (b) sand-bentonite with distilled water and 0.5 M NaCl solution

2.3 Water retentivity test

Water retentivity tests were conducted on bentonite mixtures to evaluate the water retention properties, which have significant effect on the water migration to CCLs, by using triaxial compressive testing apparatus. Specimens of 10 mm height and 50 mm diameter were prepared. After a specimen was saturated with 100 kPa back pressure in the triaxial cell, a saturated ceramics plate (500 kPa of air entry value) was placed at the bottom of the specimen. Air pressures from 10 kPa to 500 kPa were applied respectively.

Testing results are shown in Fig. 4. The matric suction of clay-bentonite and sand-bentonite compacted at their optimum water contents are about 450 and 400 kPa respectively, which are much higher than that of ordinary base soil, i.e. matric suction of compacted Toyoura sand at water contents greater than 5% is less than 6 kPa, according to the report by CAPUG (1997). These results show that pore water in base soil has great potential to migrate to CCLs, which results in the water content increase in CCLs.

2.4 Water migration test

Water migration characteristics between CCLs and base soil were examined using the apparatus shown in Fig. 5. In a polyvinyl chloride (PVC) mold with 50 mm inside diameter and 100 mm height, Toyoura sand specimens compacted at 10,

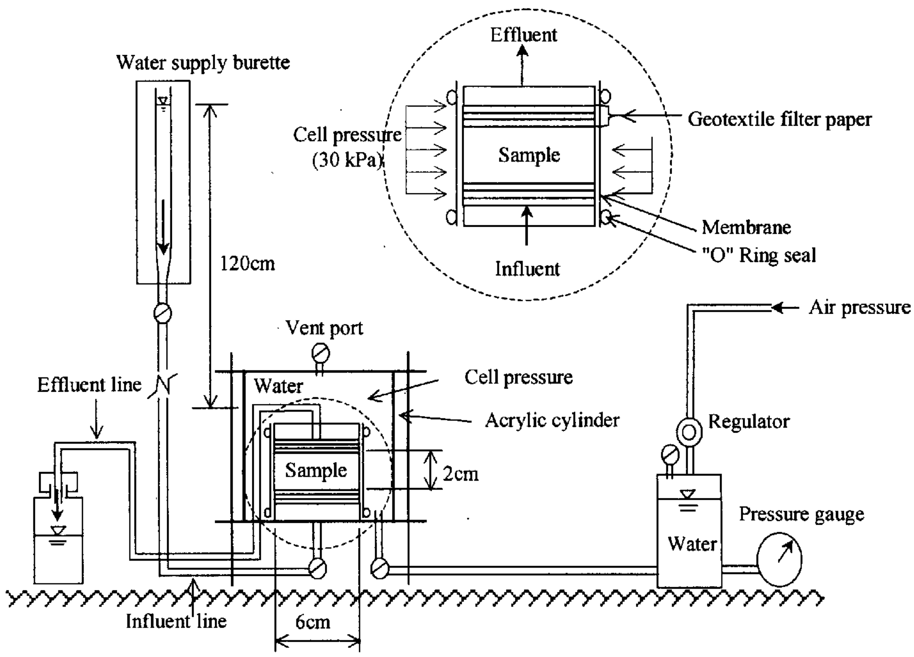


Fig. 2 Apparatus of falling head permeability test (Inazumi, 2000)

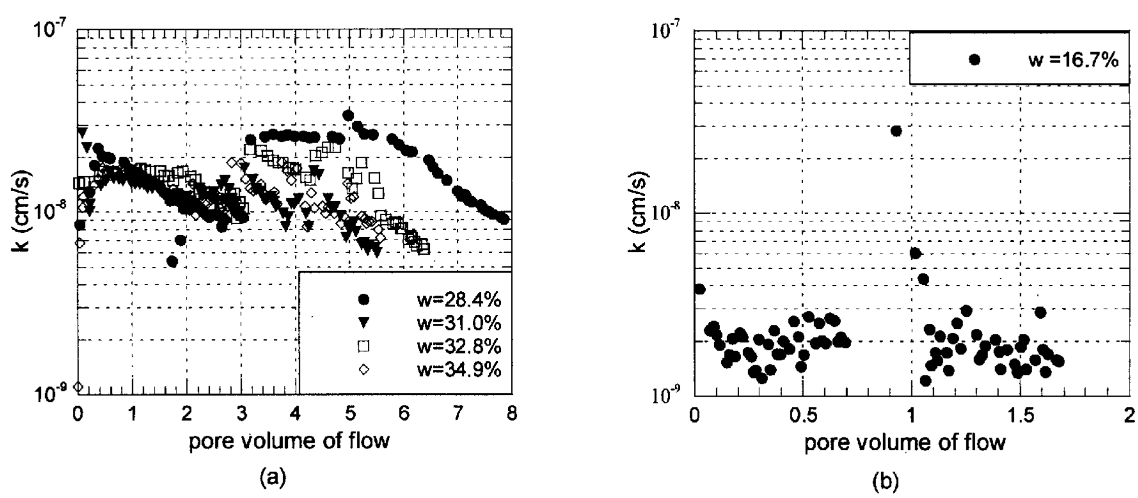


Fig. 3 Results of falling head permeability test for (a) clay-bentonite, and (b) sand-bentonite

15, 20 or 25% (saturated) water contents were placed into the lower half part of the mold, and the compacted bentonite mixture specimens shown in Tabele 2 were placed into the upper half part of the mold directly contacted with the sand base. Top of the mold was sealed with an impermeable sheet to prevent water evaporation. Confining pressure of 3.1 kPa was applied to CCLs to restrict the volume increase. After 3, 7 and 21 days, each specimen was cut into 10 equally thick slices from top to bottom, and the water content for each slice was measured.

Figure 6(a) shows the result for clay-bentonite and base soil after 3 day testing. The water contents of all clay-bentonite specimens increased, the most at the interface with sand base and the least at the middle part of specimens. The water contents at the upper part of specimens were also increased, higher than those at the middle part of specimens. This was possibly due to the impermeable sheet at the top of specimens, which controlled upward water migration. Specimens after 21 day testing had the similar water content distribution shown in Fig. 6(b). The water contents of all slices from base soil decreased, the most at the upper and the least at the lower part of specimens. Figure 7(a) shows the results for sand-bentonite and base soil after 3 day testing. The water contents of all sand-bentonite specimens increased, the most at the interface with sand base. Unlike clay-bentonite, the water contents of the upper and middle part of the specimens did not increase in the 3 day testing specimens, but after 21 day testing, more water migrated to sand-bentonite specimens, as shown in Fig. 7(b).

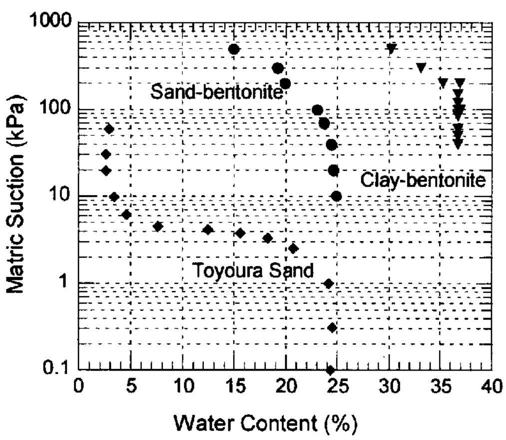
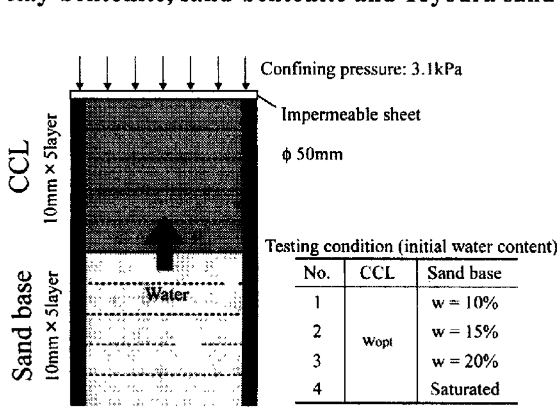


Fig. 4 Matric suction against water content for clay-bentonite, sand-bentonite and Toyoura sand



No.	CCL	Sand base
1	Wopt	w = 10%
2		w = 15%
3		w = 20%
4		Saturated

Fig. 5 Apparatus for water migration test

Results of water migration tests can be summarized as follows: (1) water moved from base soil to liners due to capillarity, (2) most of the water moved from base soil to clay liners within the first 3 days, except for one testing case (sand-bentonite on the saturated sand base), and (3) after 21 days, the water contents of clay-bentonite near the interface on the base soil having water contents of 10, 15, 20 and 25% reached 36, 36, 38 and 40%, and those of sand-bentonite near the interface increased to 22, 24, 24 and 30%, respectively.

2.5 Direct shear strength test

Direct shear tests were performed to evaluate

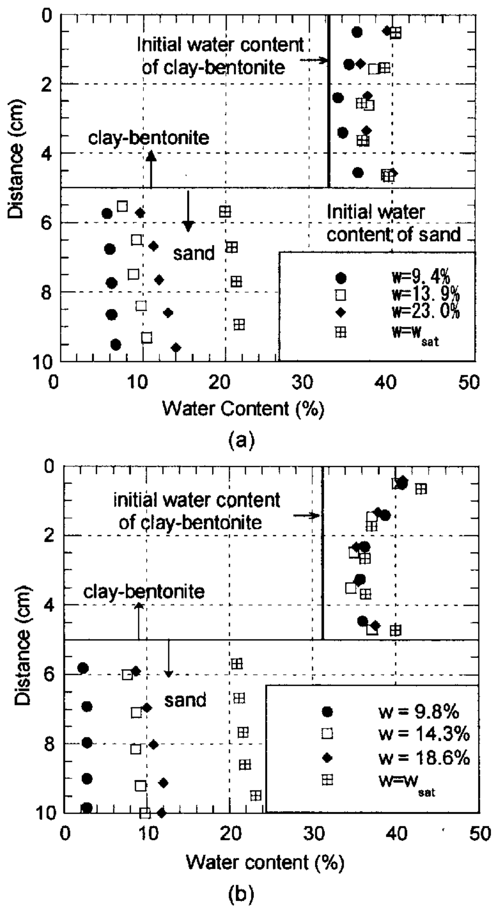


Fig. 6 Water migration test results for clay-bentonite; (a) after 3 days, (b) after 21 days

the effect of the increase in water content on the shear strength along clay liners with the consolidated constant pressure direct shear test box. Internal shear strength test on bentonite mixtures and interface shear strength test between bentonite mixtures and porous stone were conducted. Porous stone was used to simulate the base soil. Specimens of bentonite mixtures of 10 mm height and 60 mm diameter were prepared. Tests were conducted under 5 different initial conditions as shown in Table 3, including (1) bentonite mixture of w_{opt} and dry porous stone, (2) bentonite mixture with a water content w_m, determined from the water migration test results, and saturated (but not submerged) porous stone, (3) bentonite mixture with a water content of w_m and submerged porous stone, (4) saturated bentonite mixture and submerged porous stone, (5) the conditions of CCL water content and base soil are the same as case (3), but using 0.5 M NaCl solution instead of water. Shear strain rate was 0.05 mm/min, and the vertical stresses were 100, 200, 400 and 500 kPa, respectively.

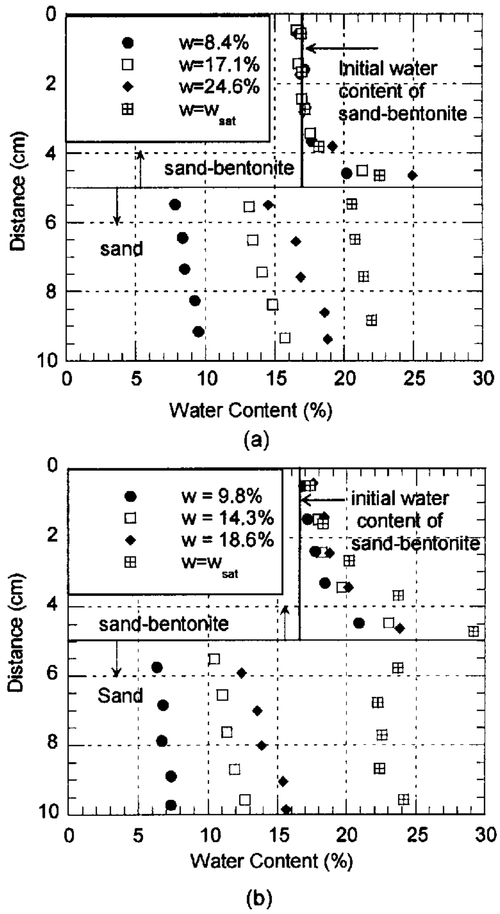


Fig. 7 Water migration test results for sand-bentonite; (a) after 3 days, (b) after 21 days

Figure 8 shows Mohr-Coulomb diagrams for each testing case, and the determined shear strength parameters are summarized in Table 3. Friction angles of clay-bentonite specimens under case (1), (2) and (3) were 21.4, 23.4 and 24.1 degree, and cohesions were 141, 40 and 31 kPa respectively. These results indicate that the friction angles of clay-bentonite under these three cases were similar, but the cohesion under case (1) was much greater than those obtained under case (2) and (3). Similar results were obtained for sand-bentonite internal shear strength, and sand-bentonite/porous stone and clay-bentonite/porous stone interface shear strength under the three cases. Although, under the case (4), the cohesions of both sand-bentonite/porous stone and clay-bentonite/porous stone interfaces are greater than those obtained from case (2) and (3),

Table 3 Conditions and results of direct shear tests

Material	Case	Initial conditions		Strength parameter results	
		Specimen	Porous stone	Cohesion (kPa)	Friction angle (degree)
Clay-bentonite internal	(1)	w_{opt} = 30.0%	Dry	141	21.4
	(2)	w_m = 38-40%	Unsubmerged	40	23.4
	(3)	w_m = 38-40%	Submerged	31	24.1
Clay-bentonite/ porous stone interface	(1)	w_{opt} = 30.0%	Dry	77	24.7
	(2)	w_m = 38-40%	Unsubmerged	13	26.3
	(3)	w_m = 38-40%	Submerged	16	27.0
	(4)	100% saturation	Submerged	42	22.0
Sand-bentonite internal	(1)	w_{opt} = 16.4%	Dry	111	36.6
	(2)	w_m = 24-26%	Unsubmerged	67	38.8
	(3)	w_m = 30%	Submerged	66	38.2
	(5)	NaCl solution	Submerged	50	44.4
Sand-bentonite/ porous stone interface	(1)	w_{opt} =16.4%	Dry	15	34.4
	(2)	w_m =24-26%	Unsubmerged	11	35.5
	(3)	w_m =30%	Submerged	9	34.6
	(4)	100% saturation	Submerged	39	23.4

Notice: w_m: water content determined by water migration tests; w_{opt}: optimum water content their friction angles are much less than those obtained under the other cases. From these results, it can be concluded that the shear strengths of both bentonite mixture internal and bentonite mixture /porous stone interface tend to diminish with the water content increase in bentonite mixtures and base soil. The cohesion of sand-bentonite internal obtained under case (5) is almost the same as that obtained under case (3), but its friction angle is about 6 degree greater. This shows that the considerate concentration of NaCl, which was generally included in leachate, would not have the negative effect on the shear strength of liners.

2.6 Consolidated-undrained triaxial compression test

Consolidated-undrained triaxial compression tests were conducted on saturated clay-bentonite and sand-bentonite specimens. Specimens were compacted at optimum water contents using a cylindrical spilt mold with 50 mm in diameter and 103 mm in height. To achieve the full saturation for bentonite mixtures, a saturation procedure was employed as follows: (1) CO_2 was pumped from the bottom to the top of specimens for 3 to 4 hours in order to exchange the air in the specimen set system, (2) deaired water was induced from the bottom to the top of specimens for 2 to 3 days, (3) back pressures of 150 kPa for clay-bentonite specimens and 300 kPa for sand-bentonite specimens were applied to achieve the Skempton's pore pressure parameter, B, higher than 0.95 (Black and Lee 1973). Generally, more than 1 week was required to achieve the desired degree of saturation. In order to model the short-term undrained loading, specimens were tested at a fast compression rate of 0.5 mm/min under cell pressures of 100, 200 and 300 kPa for clay-bentonite specimens, and 50, 100 and 150 kPa for sand-bentonite specimens.

The details of testing conditions and results before and after loading were listed in Table 4. Compared with the optimum water contents shown in Table 2, the water contents of the bentonite mixtures after compacted are a slightly lower due to evaporation. The void ratios of the bentonite mixtures after shear (e_f) are greater than those

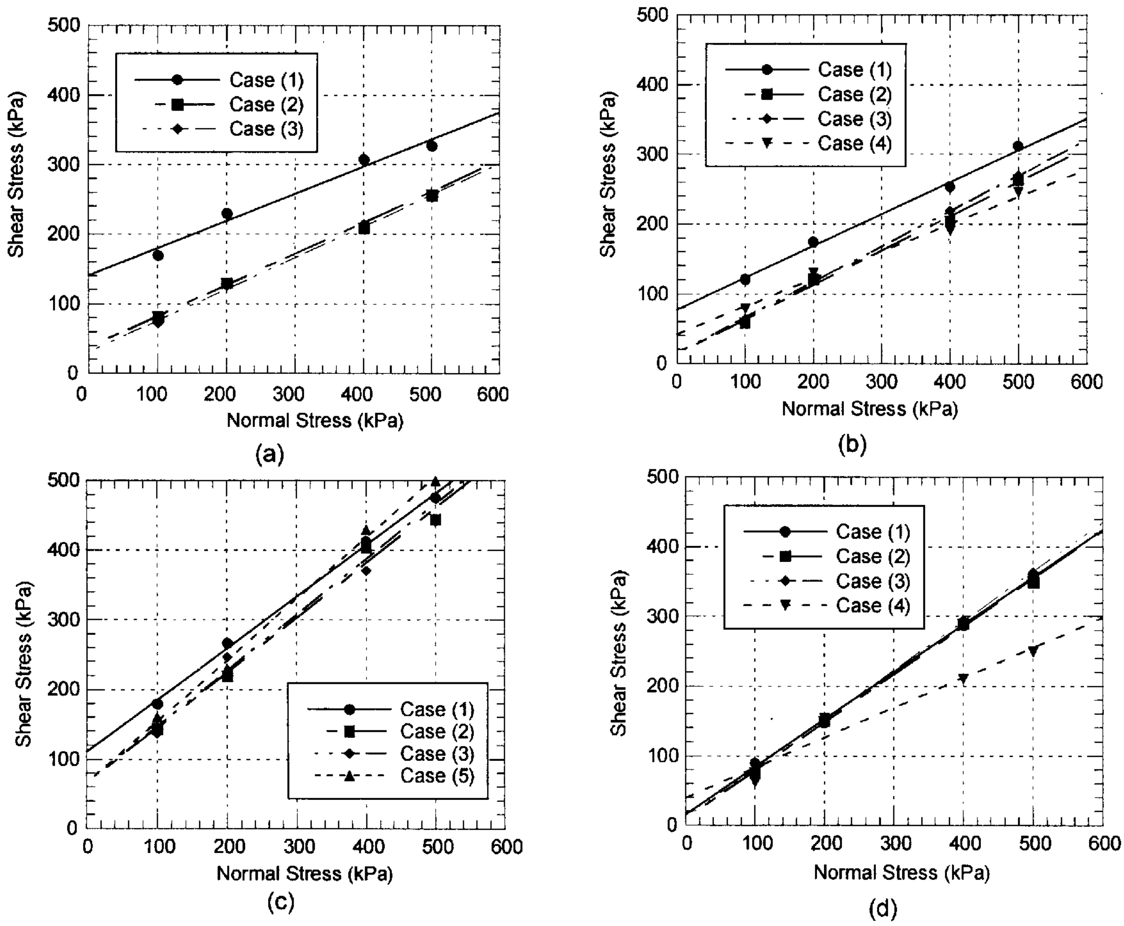


Fig. 8 Determination of shear strength parameters for (a) CB internal, (b) CB/porous stone interface, (c) SB internal, (d) SB/porous stone interface

before shear (e_0). This shows that the specimens still swelled much although 20 kPa cell pressure was applied during saturation.

The behaviors of stress-strain and pore pressure-strain for the bentonite mixtures are shown in Fig. 9 and Fig. 10, respectively. For clay-bentonite, with axial strain increasing, the deviator stress and pore pressure increased very quickly before axial strain reached 3%, but after the axial strain reached 3%, the deviator stress increased very slowly and the pore pressure began to decrease, and when axial strain reached 10%, the deviator stress remained approximately constant for any further increase of the axial strain. Compared with those of clay-bentonite, the deviator stress of sand-bentonte increased slower and smoother, but

Table 4 Details of the consolidated undrained triaxial tests on the two liners

Specimen number	J13	J14	J15	J17	J20	J21
Specimen material	Clay-bentonie			Sand-bentonite		
Back pressure (kPa)	150			300		
Cell pressure (kPa)	100	200	300	150	100	50
B value	0.9950	0.9862	0.9887	0.9930	0.9970	0.9886
Water content (%) after compaction	28.70	28.72	27.10	16.90	15.03	15.58
Water content (%) after shear	38.94	37.43	35.71	23.13	24.25	25.01
e_0 (after compaction)	0.88	0.91	0.89	0.57	0.62	0.62
e_1 (after shear)	1.03	0.99	0.94	0.61	0.64	0.66
Peak deviator stress (kPa)	174	270	365	1378	1127	780
Pore pressure at peak stress (kPa)	34	88	128	-214	-109	-100
Axial strain at peak stress (%)	10	10	10	15	15	15
Friction angle for total stress (degree)	18.5			48.5		
Cohesion for total stress (kPa)	30			90		
Friction angle for effective stress (degree)	27.2			35.0		
Cohesion for effective stress (kPa)	25			100		

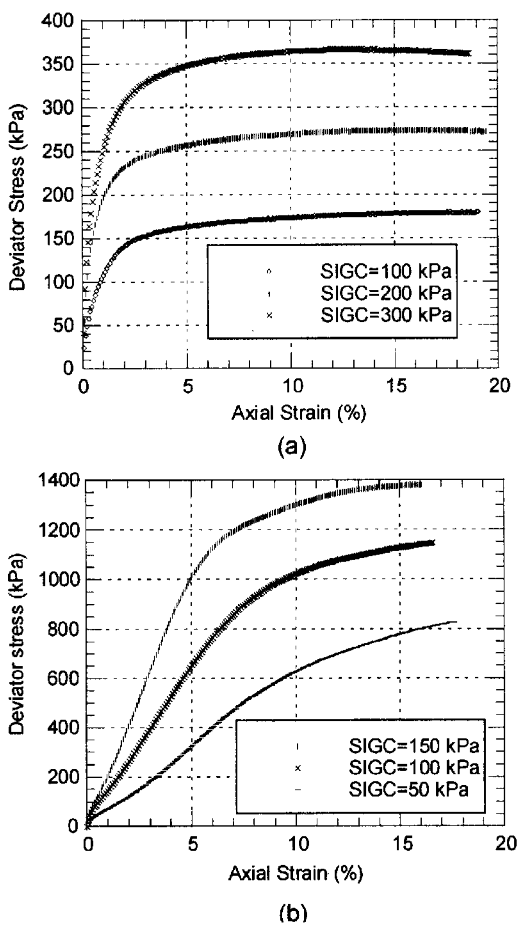


Fig. 9 Plot of deviator stress against axial strain for (a) clay-bentonite, (b) sand-bnetonite

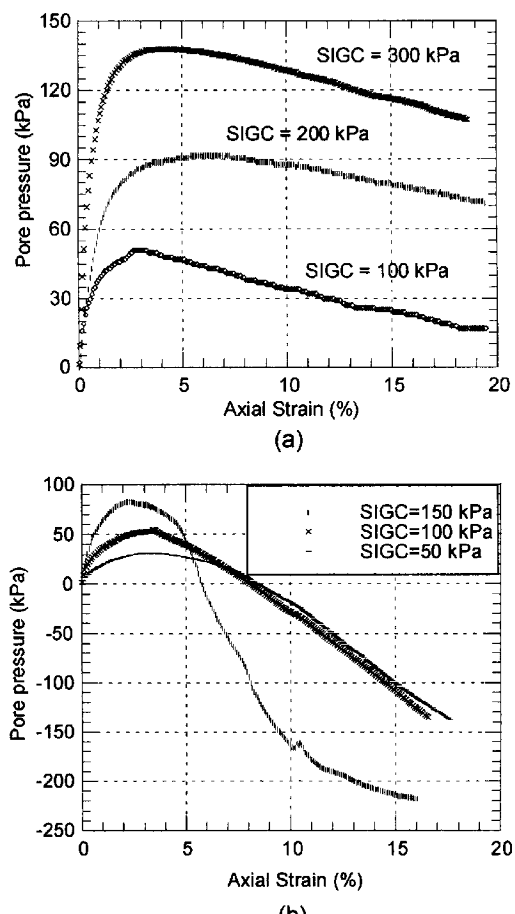


Fig. 10. Plot of pore pressure against axial strain for (a) clay-bentonite, (b) sand-bentonite

the pore pressure decreased more quickly, and became negative when axial strain reached 5.8, 8.1 and 8.6% for cell pressure of 150, 100 and 50 kPa respectively.

Figure 11 shows the total stress paths (TSPs), and effective stress paths (ESPs) for clay-bentonite and sand-bentonite (in Fig. 11, q : TSP, q' : ESP, q'(50) stands for the ESP under SIGC = 50 kPa). The ESPs for clay-bentonite lie to the left of the TSPs, while the ESPs for sand-bentonite go slightly to the left of the TSPs at first, then as the pore pressures become increasingly negative, the ESPs cross the TSPs to the right. The result shows that the compacted clay-bentonite showed normal consolidation characteristics, while the compacted sand-bentonite exhibited over-consolidation state.

Figure 12 indicates the Mohr circles at failure and Mohr-Coulomb failure envelopes for both total and effective stresses. Total stress strength parameters are ϕ = 18.5 degree and c = 30 kPa for clay-bentonite, and ϕ = 48.5 degree and c = 90 kPa for sand-bentonite; and effective stress strength parameters are ϕ'= 27.2 degree and c' = 25 kPa for clay-bentonite, and ϕ'= 35 degree and c' = 100 kPa for sand-bentonite. Because the hydraulic conductivity of both clay-bentonite and sand-bentonite is very low, and a very fast rate of strain was used, the pore pressure of the two bentonite mixtures was not exactly measured in the test.

3. Stability analysis

Stability of a typical canyon solid waste landfill along clay liners was analyzed based on the shear strength parameters obtained from laboratory tests. The assumed cross section of a canyon waste landfill is shown in Fig. 13. The method of Janbu's Generalized Procedure of Slices was applied in the calculation and WINSTABLE, a software for slope stability analysis developed by the University of Wisconsin was used for this stability analysis. Assumptions for the analysis are as follows:

(1) Solid waste was considered to be an incinerated ash. Wet and saturated unit weights are 15.5 kN/m^3 and 17.4 kN/m^3 (Hirano et al. 2000), and shear strength parameters are c = 0 kPa, ϕ = 43 degree (Daniel 1993).

(2) CCL was installed as bottom liner with 0.5 m thickness. Wet and saturated unit weights are 17.5 kN/m^3 and 18.5 kN/m^3 for clay bentonite liner (CBL), and 18.5 kN/m^3 and 20.5 kN/m^3 for sand-bentonite liner (SBL).

(3) The base has no possibility of failure within it.

(4) Sliding surfaces are assumed in two ways. One is that all sliding surfaces except for one (at the slope toe) are considered occurring along liners shown in Fig. 13; the slope was divided into 13 slices according to the variation of inclination of underlying base and cover

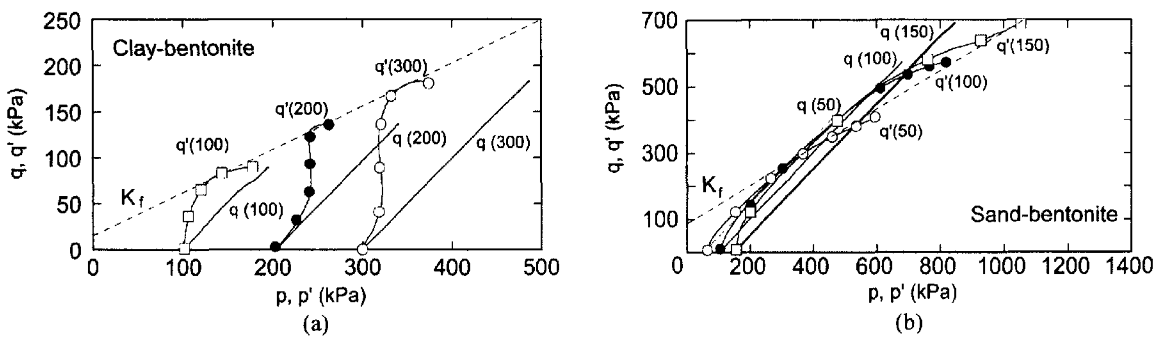


Fig. 11 Stress paths for (a) clay-bentonite and (b) sand-bentonite

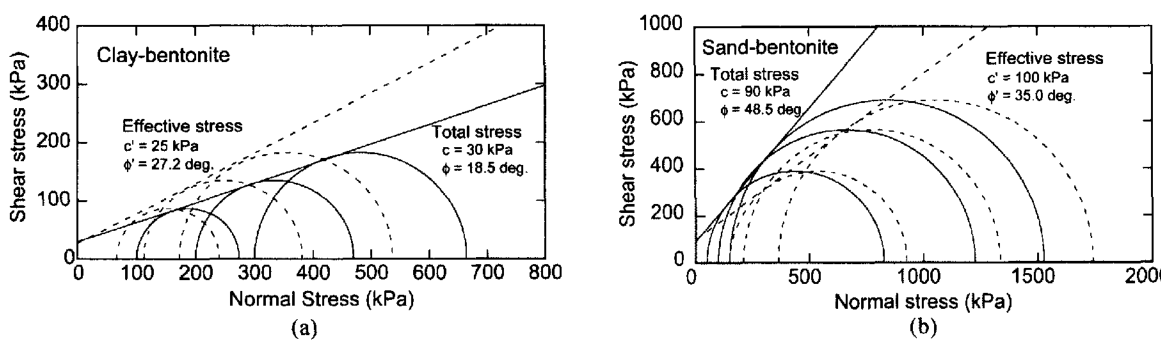


Fig. 12 Results of CU triaxial compression test for (a) clay-bentonite and (b) sand-bentonite

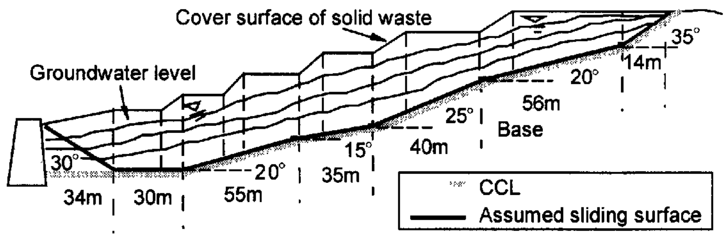


Fig. 13. Cross section of a canyon solid waste landfill

Table 5. Assumed condition in the stability analysis

Case	Groundwater level	CCL	Base soil
(i)	below liners	w_{opt}	dry
(ii)	below liners	water migrated	wet but un-submerged
(iii)	the top of clay liners	water migrated	submerged
(iv)	1/4 of the waste height	saturated	submerged
(v)	1/2of the waste height	saturated	submerged
(vi)	3/4 of the waste height	saturated	submerged

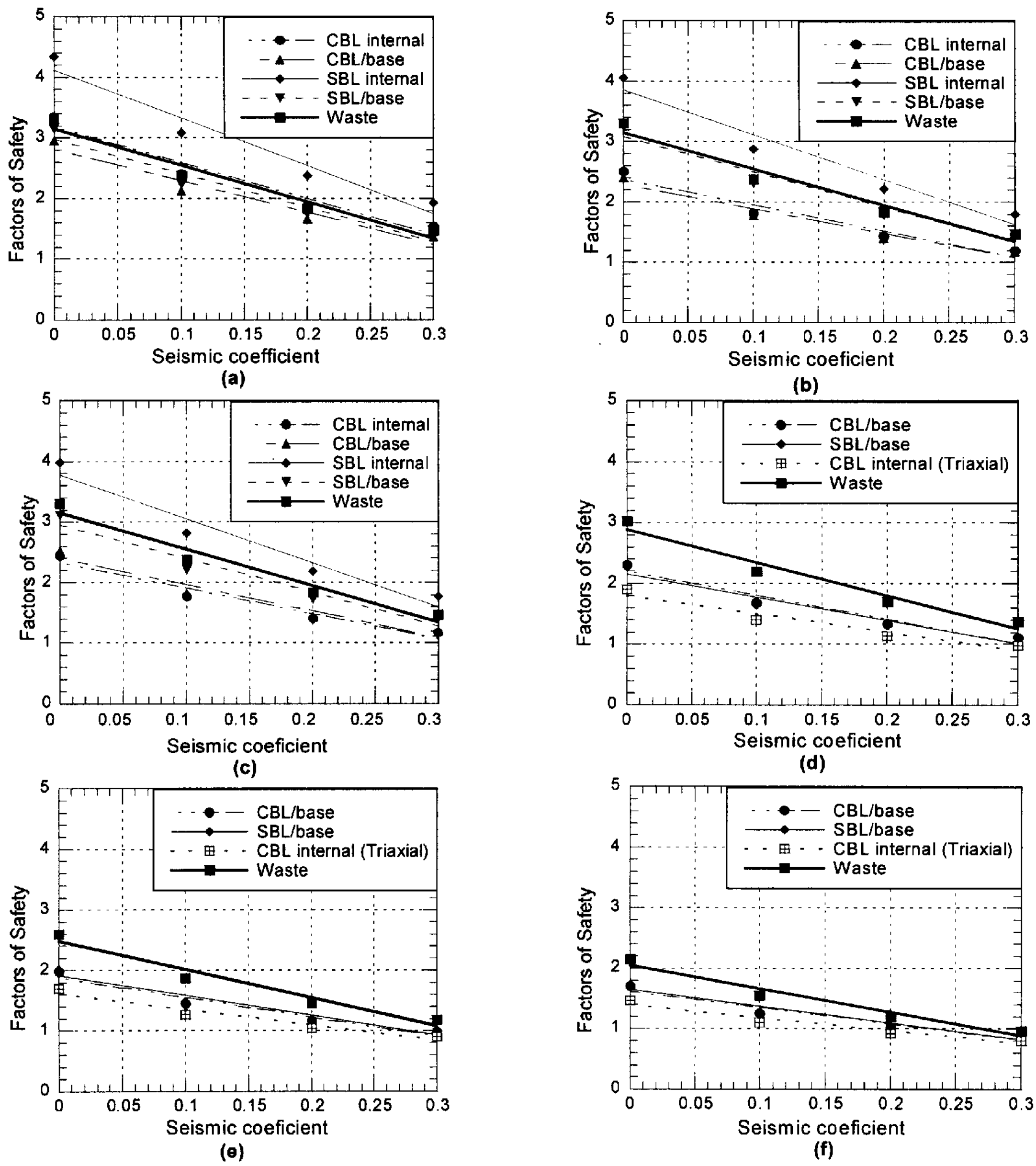


Fig. 14 Variation of the factor of safety with horizontal seismic coefficients; (a) calculation results under case (i) shown in Table 5, (b) case (ii), (c) case (iii), (d) case (iv), (e) case (v) and (f) case (vi).

surface. The other is that sliding is considered to occur on any surface in the landfill.

(5) Groundwater level and clay liners are parametrically presumed six conditions shown in Table 5. Shear strength parameters of CCL suitable for each condition are based on direct shear tests results shown in Table 3, and total shear strength parameters from triaxial compression tests shown in Fig. 12.

(6) Earthquake effect on the stability of the landfill was taken into account with the static seismic load method. The typical seismic coefficients used for the seismic stability of slopes in Japan are 0.15-0.25 (Abramson, et al, 1996). Thus, the maximum magnitude of the horizontal seismic coefficient k_h = 0.3 was selected for this calculation.

Figure 14 shows the calculation results for the factors of safety under each condition. The item "any surface" in each caption means the result when sliding could occur on any surface. From these results, the factors of safety for wet liner internal and interfaces are smaller than those for dry liner internal and interfaces with various horizontal seismic coefficients in case (i), (ii) and (iii). Thus, the landfill faces to significantly decrease the stability when dry liners become wet. Comparing the stability for two liner materials, the factors of safety for SBL/base, CBL internal and CBL/base with various horizontal seismic coefficients are almost the same in case (i). However, for case (ii) and (iii), the factors of safety for SBL/base with various horizontal seismic coefficients are greater than those for CBL internal and interfaces. Therefore, the landfill is considered more stable using SBL than CBL. In addition, the factors of safety for SBL internal are much greater than those for SBL/base in the three cases; the factors of safety for CBL internal are similar to those for CBL/base. Hence, with water content increasing, failures of landfills with SBL will occur along the interfaces between SBL and base, but with CBL will occur in both CBL internal and CBL/base interface.

Calculated results under case (iv), (v) and (vi) indicate that the landfill with various horizontal seismic coefficients is significantly less safety with the increase in groundwater level, for example, the factor of safety for CBL internal, when the ground water level is about one fourth height of the waste layer, is 1.9 in static condition, i.e. k_h = 0; and that when the ground water level is about three fourth height of the waste layer is less than 1.5.

When horizontal seismic coefficient k_h = 0, the factors of safety for all calculation cases are greater than 1.4; the landfill with SBL or CBL is stable when earthquake does not occur even when groundwater level is near cover surface. When k_h = 0.3, the factors of safety for all cases are greater than 1.0 with groundwater level keeping low at the bottom of the waste. With groundwater level increasing, however, the factors of safety will become less than 1.0. In case (v), the factors of safety for CBL internal, CBL/base and SBL/base are less than 1.0 when seismic coefficients are greater than 0.24, 0.27 and 0.28 respectively. In case (vi), the factors of safety for CBL internal, CBL/base, SBL/base and waste are less than 1.0 when seismic coefficients are greater than 0.17, 0.23, 0.24 and 0.27 respectively. Therefore, the landfill with high groundwater level is unstable either with wet CLB when earthquake magnitude is great enough to produce $k_h > 0.17$ or with wet SBL when earthquake magnitude is great enough to produce $k_h > 0.24$.

4. Conclusions

The suitability of sand-bentonite and clay-bentonite mixtures as landfill bottom liners was assessed from the viewpoint of laboratory permeability tests and the stability of landfills, considering the water migration between liners and underlying base soil. According to the experimental study and the landfill stability analysis, following conclusions can be made:

(1) The hydraulic conductivity of clay-bentonite is about 3.0×10^{-8} cm/s, and that of sand-bentonite is about 3.0×10^{-9} cm/s. Compared with the standard hydraulic conductivity used in Japan (1.0×10^{-6} cm/s), the hydraulic conductivity of both clay-bentonite and sand-bentonite is much lower.

(2) Pore water in the underlying base soil has great potential to migrate to the CCLs due to the capillarity, which results in significant increase in water content of CCLs.

(3) Shear strength of both CCLs internal and CCLs/base interface decreased with the increase in water content in CCLs.

(4) The stability of canyon-type landfill could be confirmed to be more safety using SBL than CBL. Landfills at gentle slope (total angle of

slope is not greater than 20 degree) do not fail in the both cases of SBL and CBL with horizontal seismic coefficient $k_h < 0.3$ under groundwater level keeping low at the bottom of the solid waste.

(5) Groundwater level affects the landfill stability significantly. Hence, leachate drainage in landfills is an effective option for the landfill stability.

In all, sand-bentonite and clay-bentonite mixtures are suitable to be landfill bottom liners for those landfills at gentle slope (total angle of slope is not greater than 20 degree). And as a landfill liner, sand-bentonite is better than clay-bentonite.

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要　旨

本研究では2種類のベントナイト混合土を対象に各種室内試験を実施し，廃棄物処分場遮水工材料への適用性を遮水性能および構造安定性の面から評価を行った。ベントナイト混合土の遮水性能については柔壁型透水試験装置によって計測した透水係数により評価した。一方，水分移動実験およびせん断試験を実施し，ベントナイト混合土の水分保持特性がせん断強度に与える影響を検討した。さらに，これらの結果に基づいて，二次元斜面安定解析を実施し，ベントナイト混合土を用いた底部ライナーのせん断破壊の可能性と影響要因を検討し，ベントナイト混合土の廃棄物処分場遮水工への適用性を構造安定性の側面から議論した。

キーワード：構造安定性，粘土ライナー，透水試験，保水性試験，水分移動実験，一面せん断試験，圧密非排水三軸圧縮試験