A Thesis Submitted to Kyoto University, Japan for Application of Doctoral Degree of Engineering

Waste Sludge Barrier for Landfill Cover System

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Abstract

This is an environmental geotechnical study about the utilization of waste sludge material as cover material in landfill site, emphasizing the significance of cover system with waste sludge barrier layer for reducing the pollution risk of leachate to the nearby environment. Application of waste sludge as cover material in landfill site not only provides an alternative waste management method for reducing the wastes as resource, but also improves the traditional landfill technology in Japan. The objectives of this study are: (1) to clarify the feasibility of waste sludge materials as barrier layer in cover system, (2) to predict the effects of cover systems on reducing the quantity of rainwater percolating through the cover systems, and (3) to evaluate the effect of cover system with sludge barrier on the migration properties of landfill gas.

To determine the feasibility of waste sludge as barrier layer in cover system, various geotechnical tests including hydraulic conductivity tests, as well as a chemical analysis of the effluents, were conducted on the waste sludge materials such as paper sludge (PS) and construction sludge (CS). The long-term behaviors as well as the durability of waste sludge materials under wetting-drying cycles, with respect to settlement and hydraulic conductivity, were also examined by geotechnical centrifuge tests. The discussions derived from these experimental approaches contribute to the development of effective management of wastes, so as to reuse the waste sludge in a practical way.

In order to predict the rainwater interception performance of cover systems with the sludge barriers, water balance analysis was conducted with consideration of the metrological conditions in Japan, and the of saturation condition of the sludge barriers tested. The results indicates that the infiltration of rainwater into waste layer can be considerably minimized by the installation of cover system even in Japan with a monsoon climate, where the rainfall infiltration reduction by cover systems will be much greater than that in the United States and the most European countries. The significance of the cover system as one of the most important waste containment facilities is emphasized because it can greatly reduce the negative effect to surrounding geo-environments by cutting down leachate generation.

To investigate the influence of the cover system on the generation and migration of landfill gas, the gas permeability of the sludge materials as sludge barrier in cover system was measured. Also, the performance of the passive gas vent well on controlling the landfill gas migration within the waste layer with cover system installation was evaluated. Few studies have been done on designing the cover system and gas collection system to control the landfill gas. Therefore, the evaluation techniques with creative experimental and analytical strategies in this research will provide good reference to the similar research followed.

In Japan, evaluations of the cover system still have not been discussed. This study has a distinctive feature in evaluating the cover system which controls rainwater infiltration into waste layer. Furthermore, the feasibility of waste sludge materials for barrier layer in cover system, as clarified in this study, makes it possible that reuse of waste sludge can be substituted for the landfill space occupied by the barrier materials such as clay materials. Thus, the study responds to the social requests to rationally reduce the waste to be landfilled, and to reduce landfill site construction costs. The application of the waste sludge to barrier material in waste containment facilities is ingenious scheme for developing the landfill technology in Japan. The result obtained by this study contributes to sustained development of the global society over the future, and will provide precious data to promotion of effective utilization of waste, and establishment of landfill waste containment technology.

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CHAPTER 1

Introduction

1.1. General Remarks

Under present world conditions, the environmental problems have represented an important issue in most parts of the world. The resolution of environmental problems related to the geo-environment relies on the environmental geotechnical approaches. Kamon and Jang (2001) defined the area of the environmental geotechnics as a field dealing with all kind of environmental problems related to the geotechnical engineering. In recent year, it has been revealed that the geo-environments in surrounding areas of landfill sites, farmlands, and old factory sites are polluted by toxic substances, such as heavy metals and organic compounds. Such environmental pollutions are one of the areas to which the environmental geotechnics have to contribute for the resolution. A typical geo-environmental contamination at landfill site caused by the leakage of waste leachate is illustrated in Fig. 1.1. The infiltrations of

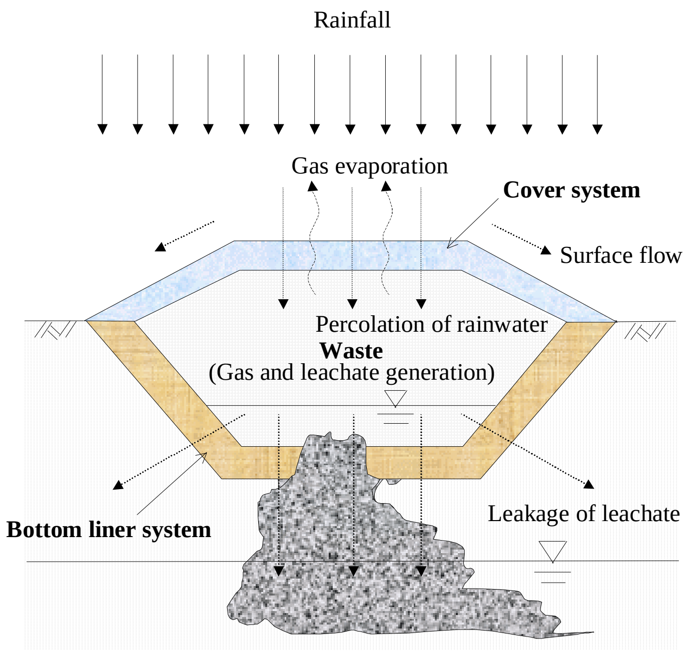

Fig. 1.1 Conceptualization of the leakage of toxic substances from a landfill due to rainfall

rainwater and surface water into waste layer cause the accumulation of leachate generated in the waste on a bottom liner, and increase the risk of the leahate leakage into surrounding groundwater environments. About the environment protection in the landfill sites, for example, it is required that the leachate generated in the waste should be managed so that it does not affect the surrounding geo-environments such as groundwater environments (Kamon et al., 2001). Usually, waste containment facilities are required to protect the peripheral geo-environment from being polluted by the migration of waste leachate. Effective design of the waste containment facilities meanes not only to install bottom liner system, cutting down the leachate produced but also to establish the landfill cover system, preventing the infiltration of rainwater and surface water into the waste layer and to minimize the generation of waste leachate. Perhaps due to the historical landfilling practices in Japan, however, technical guidelines for the cover system at landfill sites are not announced officially, and insufficient discussions have been made on this issue (Kamon and Katsumi, 2001).

In the present research, a waste containment technology which consists of a cover system, to minimize the infiltration of rainwater into waste, and the negative impacts of the waste leachate on the surrounding environments of landfill, is highlighted by the environmental geotechnical approaches. In particular, the significance of cover system with waste sludge barrier layer is emphasized on reducing the rainwater penetrations into landfills to decrease the pollution risk of leachate to the nearby environment. In fact, the cover system with sludge barrier is confirmed, from the worldwide successful application, that it reduces greatly the rainwater penetrations into landfills and so as to decrease the pollution risk of leachate to the nearby environment.

Application of waste sludge materials to the barrier layer in cover system, in this study, is derived from the technical background for generation and management problems of waste mentioned as following. With increase in the population and the energy resource consumption, a large quantities of waste have being produced and discharged throughout the world. The increasing waste generation may directly cause the environmental problems in the landfill site. In the present conditions of the waste generation in Japan, although the amount of waste generating is being progressively managed with extrication from dependence on lifestyle based on large-scale production, consumption, and waste-producing, the diversification of quality of waste and the scarcity of residual space of landfill sites have developed. Furthermore, the environmental pollution due to the generation and management of waste has become one of the most emergent problems to which the human being should find a solution (Katsumi, 1996). In order to mitigate the environmental problems related to waste generation and management, the basic strategies have been directed at the reduction of waste production and the recovery of usable materials from waste as well as the reutilization of waste as raw materials whenever feasible (Gu, 1998; Nontananandh and Kamon, 2000). From the viewpoint of environmental geotechnical engineering, the utilization of waste as construction materials provides an attractive alternative in the case that the land available is limited (Kamon et al., 2000). The application of wastes in geotechnical construction works has been strongly recommended based on a viewpoint of the environmental geotechnics. In the last decade, the researches about the recycling and reusing of various wastes as geo-materials have been actively carried out. Under these circumstances, the present study discusses the applicability of paper sludge (PS) and construction sludge (CS) as classified as wastes, to the barrier layer in cover systems, and the water interception performance of cover system with PS and CS barriers. In fact, reuse of PS and CS as cover materials have been reported in the United States and most European countries. These examples further support the challenge to the effective utilization of the waste sludge materials to the cover materials in this research.

This is a geo-environmental study about the utilization of waste sludge material as cover material (barrier layer in cover system) in landfill. Geotechnical properties of waste sludge materials, especially the characteristics of hydraulic conductivity under various conditions of sludge materials, are understood thoroughly from the sound laboratory experiments. These data will provide a good foundation for the potential utilizations of waste sludge materials as barrier layer in cover system at landfill sites in Japan. As a case study, the rainwater interception by cover system of waste sludge materials is estimated theoretically using water balance in cover system. This research is of importance in effective reuse of waste sludge as cover materials, and in minimizing the environmental risk of landfill sites in Japan.

1.2. Objectives and Scope

The objectives of this study are: (1) to clarify the feasibility of waste sludge materials as barrier layer in cover system, (2) to predict the effects of cover systems on reducing the quantity of rainwater percolating through the cover systems, and (3) to evaluate the effect of cover system with sludge barrier on the migration properties of landfill gas.

In order to achieve these objectives, various geotechnical tests including hydraulic conductivity, shear strength, consolidation, x-ray diffraction, and digital microscopy tests, as well as a chemical analysis of the effluents, are conducted to evaluate the geotechnical and chemical properties of waste sludge materials, and the suitability of applying the waste sludge materials as barrier layer in cover system. The long-tern behavior as well as durability under wetting-drying cycles of waste sludge materials, with respect to settlement and hydraulic conductivity, is also examined by geotechnical centrifuge tests. The rainwater interception performance of cover systems with the sludge barriers is analytically predicted using water balance analysis with consideration of the metrological conditions in Japan, and the saturated and unsaturated characteristics of the sludge barriers which are obtained experimentally. After experimentally clarifying the gas permeability of the sludge materials as sludge barrier in cover system, the performance of the passive gas vent well is discussed to evaluate its performance on controlling the landfill gas migration within the waste layer with cover system installed.

1.3. Outline of the Dissertation

This dissertation is composed of 8 Chapters. The research flow and content of this dissertation is shown in Fig. 1.2. Chapter 1 clarifies the objectives and the contents of the dissertation. Chapter 2 reviews the present status of the waste managements and landfill technologies and a

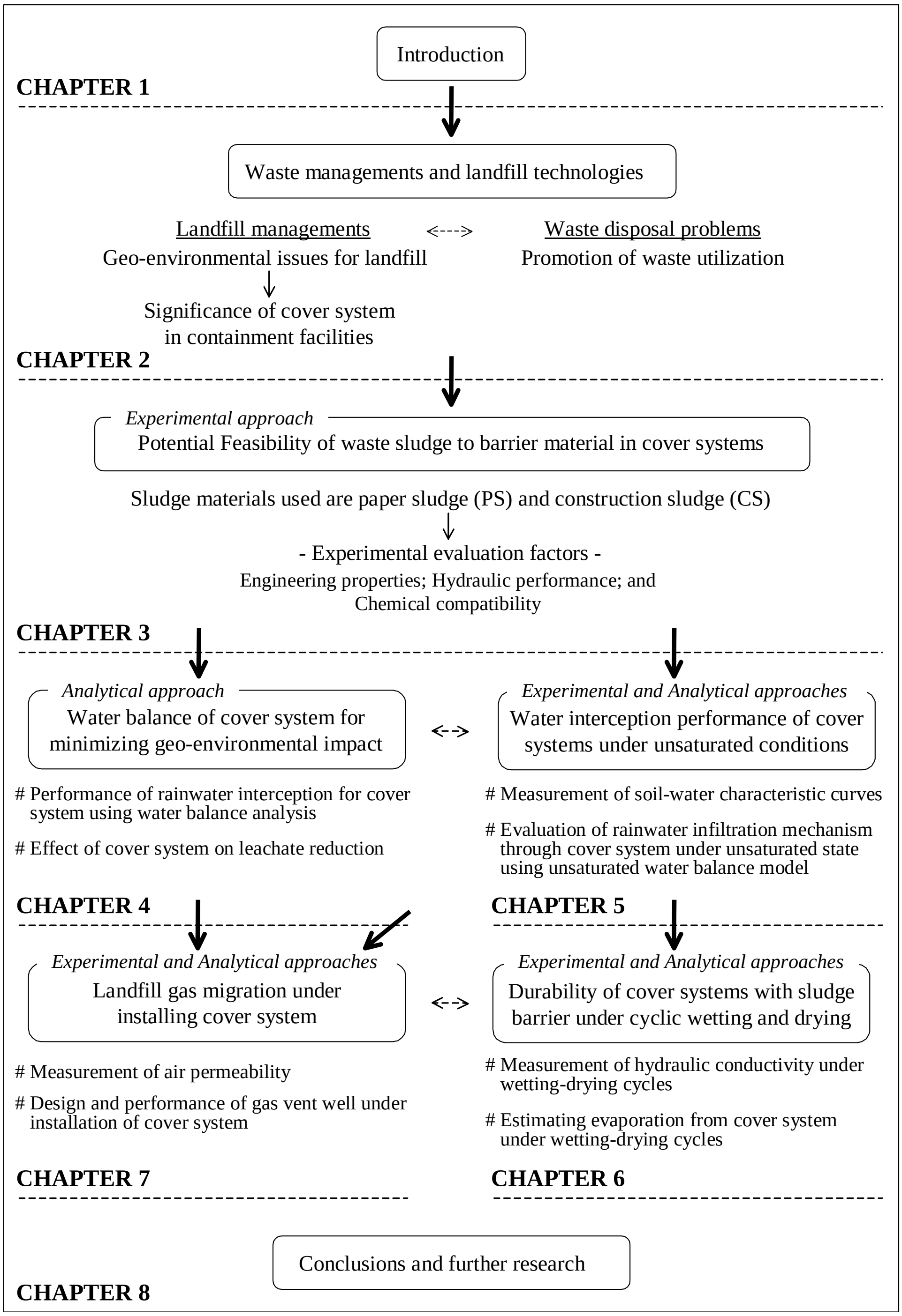

Fig. 1.2 Contents and structure of this dissertation

discussion is made to compare the difference in some regulations between Japan and other countries. Chapter 3 shows the possibility of using PS and CS as landfill cover materials (i.e., sludge barrier) by experimentally evaluating their geotechnical and chemical properties. Chapter 4 clarifies the performance of cover systems with sludge barriers for minimizing geo-environmental impacts at several sites in Japan by conducting the water balance analysis. Chapter 5 predicts the effects of the water interception of cover systems with PS and CS barriers using the unsaturated infiltration characteristics of PS and CS obtained experimentally, and meteorological data on each place in Japan. Chapter 6 shows the hydraulic performance and desiccation shrinkage (cracking) behavior of PS and CS subjected to the wetting-drying cycles as the sludge barrier. The durability of the compacted PS and CS as a barrier layer in cover system is evaluated by calculating the changes in the suction distribution under exposing to the measured field weather conditions. Chapter 7 indicates the gas permeability the PS and CS as sludge barrier in cover system. After clarifying the gas permeability of PS and CS, the performance of the passive gas vent well in the waste layer with cover system is discussed. The influenced region by single passive gas vent well on reducing the pressure of ambient landfill gas is calculated. Chapter 8 summarizes the whole results of the present study, and points out the further research needed.

1.4. References for Chapter 1

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CHAPTER 2

Current Studies on Waste Managements and Landfill Technologies

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2.1. General Remarks

A large quantity of waste, which is discharged as the results of human activities, have been generated and the problems related with the treatment and disposal of them have been emphasized as a critical issue to be resolved, in recent decades. Various sectors therefore have been attempting the effective utilization of waste. Among the challenges in various sectors, since a construction field uses a large number of materials, it is particularly expected as the final destination of effective utilization of waste and as the field which can receive the most of waste. In the field of environmental geotechnical engineering, many measures for reusing wastes alternative of soil-materials have been aggressively performed. On the other hand, the place where large quantity of waste, which has not been reused effectively, is disposed of is “landfill”. However, the geo-environmental pollutions caused by the landfill, such as soil pollutions, groundwater pollution and so on, provide an important problem; therefore various measures for the safe disposal methods have been gradually researched.

A large quantity of by-product from construction field itself is also generated, and the quantities of surplus soil and construction sludge are especially enormous; the effectively reusing technologies with both hard and soft aspects are thus developed. For effectively reusing waste as geo-materials, the environmental impacts due to wastes reusing have to be assessed. Many discussions are performed for the technique of environmental impact assessments.

In this Chapter, the present status of the waste managements and landfill technologies are reviewed and a discussion is made to compare the difference in some regulations between Japan and other countries. This will provide an environmental geotechnical background for the reusing of waste sludge, such as paper sludge and construction sludge, for cover material, and for emphasizing the engineering significance of a cover system in the landfill containment facilities in Japan.

2.2. Literature Reviews of Waste Managements

2.2.1. Definition and Classification of Wastes in Japan

Waste is defined as any materials of solid, liquid, gas, or vapor, those are not used, any more, in the production of a commercial product or the provision of a service, those are not intended commercial product, and those are unwanted, unusable, and surplus. Wastes are thus regarded as the "by-products" or "end products of the production and consumption process", respectively.

Wastes are classified into two categories, under the "Waste Disposal and Cleansing Act" (i.e., "Waste Disposal Law") in Japan, as a municipal solid waste (MSW) and an industrial waste, as shown in Fig. 2.1 which briefly shows the classification of waste under the "Waste Disposal Law". The municipal solid wastes are composed of wastes generated by households and wastes of similar character from shops, markets, offices, open areas, and treatment plant sites, respectively. As noted, in Japan, the household-generated solid waste is legally designated as a municipal solid waste. The municipal solid waste also includes wastes from offices and enterprises, but not wastes generated during industrial production processes. Local governments in Japan typically handle the municipal solid waste. In the United States and European countries, the term "municipal solid waste" does not include waste generated by enterprises; and each enterprise is responsible for the cost of treating and disposing of all types of waste generated by their operations (Yasuda, 1997).

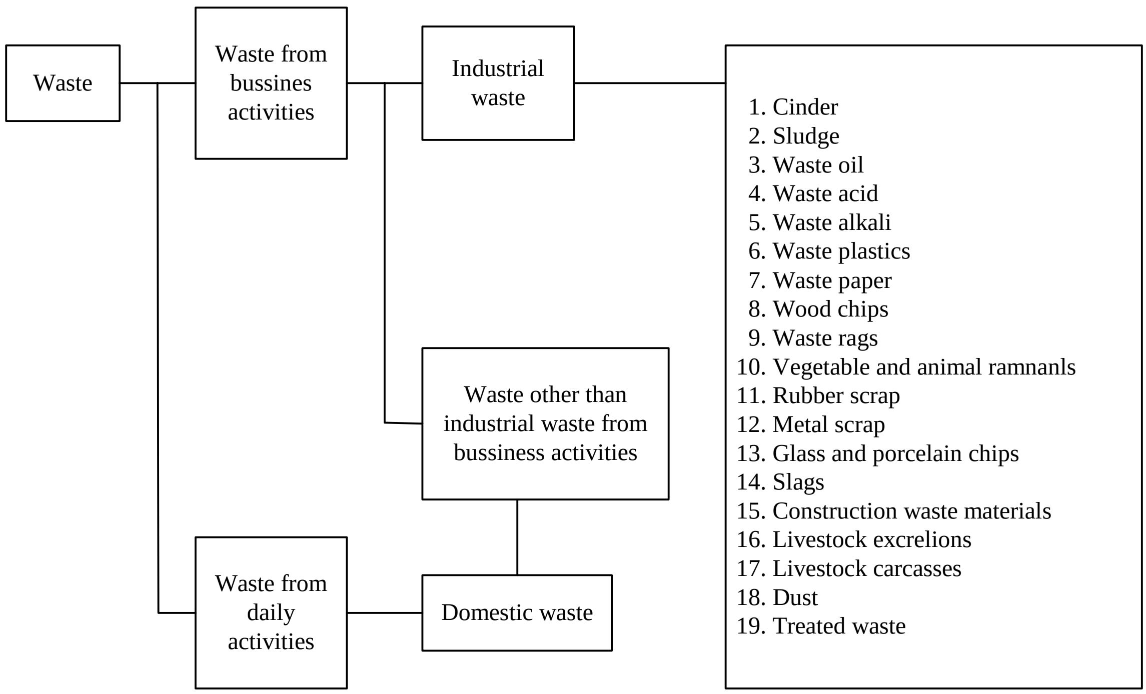

Fig. 2.1 Classification of waste in Japan

Industrial wastes include a very wide range of wastes and the actual composition of industrial wastes will depend on the property of the industrial base. The industrial wastes may occur as relatively pure substances or as complex mixtures of varying composition and in varying physicochemical states. The examples of the industrial waste are excavated soil, slurry, or sludge by the construction industry, general factory rubbish, organic wastes from food processing, acids, alkalis, and tarry residues (*see* Fig. 2.1). The most important feature of industrial wastes is that a significant proportion is regarded as hazardous or potentially toxic, thus requiring special handling, treatment, and disposal.

In Japan, the disposal of municipal wastes is the responsibility of the municipalities. The disposal of industrial wastes is the responsibility of the organizations, which generate such wastes. For this reason, the industrial waste is being treated and discharged for charge at treatment facilities involving private waste disposal companies and public sector.

2.2.2. Basic Principles of Waste Management

Waste management is the control and administration of activities involving waste. These activities include waste prevention, elimination, reduction, recycling, treatment, and disposal, in order of preference, commonly referred to as the waste management hierarchy. Other activities are the generation, handling, storage and transport of waste (Hartlén, 1994).

The basic principle of waste management in Japan, where it is extremely difficult to secure the land for landfill sites due to limited availability for the land space, is as follows: (1) effort to minimize the amount of waste, (2) segregated disposal, (3) promotion of recovery for resources and recycling, (4) reduction of volume, creation of nonhazardous waste, and stabilization and conversion of waste into resources through intermediate treatment such as incineration and shredding, and (5) final treatment of residue (Hanashima and Furuichi, 2000). The example of treatment and disposal flow for municipal solid waste is illustrated in Fig. 2.2.

In order to achieve positively the waste management mentioned above, the waste management should be conducted rationally based on an environmental geotechnical approach. Kamon et al. (2000) discussed the current states of waste management in Japan from environmental geotechnical perspectives. They pointed out that large amounts of waste are generated from various industries and activities of human being, and much of them are not being utilized, but disposed of in the limited sites which will be exhausted in the near future. To preserve natural resources and minimize the need for landfilling, the utilization of wastes as construction or geotechnical material has been strongly recommended, and other attempts of geotechnical applications also have been undertaken. Various kinds of ground improvement and soil stabilization techniques have been widely used to modify the engineering properties of waste for the geotechnical utilization as reported by Kamon and Katsumi (1994a). The potential environmental risk by the geotechnical utilization of wastes needs to be avoided. Many waste materials might be contaminated by toxic and hazardous substances and require treatment for safe disposal. Finally, Kamon et al. (2000) proposed that

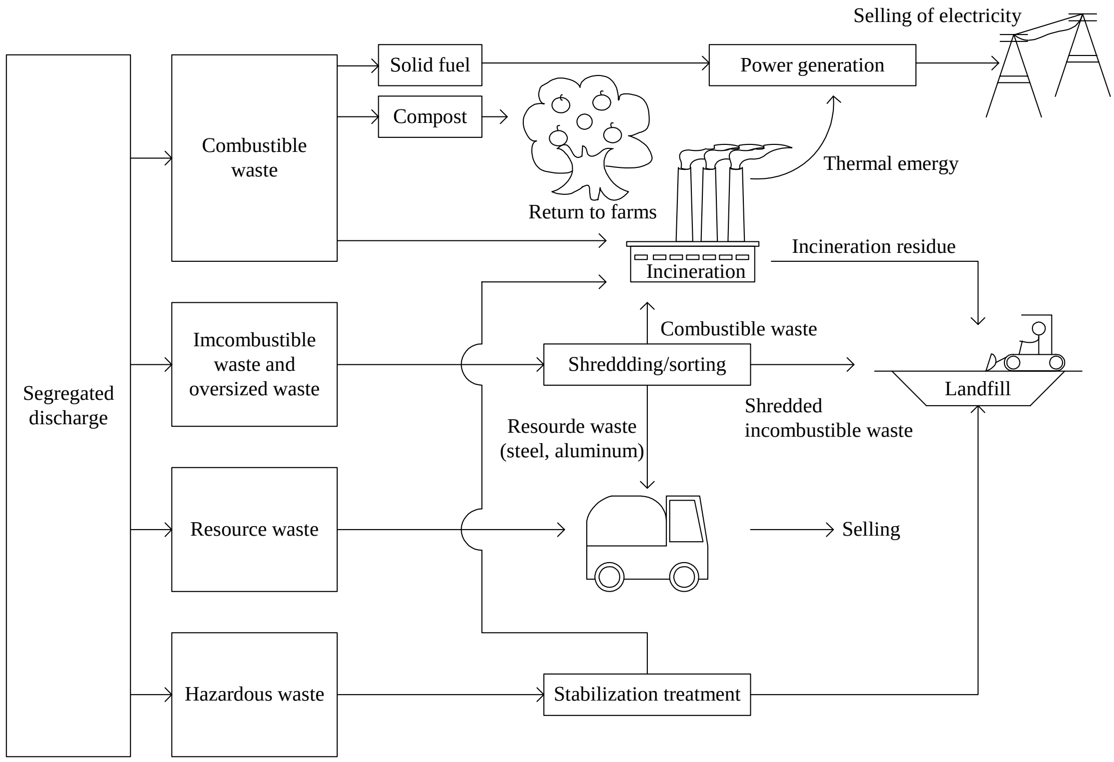

Fig. 2.2 Treatment and disposal flow of municipal solid waste (MSW)

the geotechnical waste utilization can serve not only to prevent the negative environmental impact but also to preserve and protect nature.

2.2.3. Waste Generation and Its Reduction

The generation of MSW and industrial waste, and their treatment and disposal statistics are shown in Fig. 2.3. In year 1995, the generation of MSW and industrial waste was 50.69 and 374 million tons, respectively. This means that a total of 424.69 million tons of waste was discharged that year. Furthermore, out of the entire MSW generation in the year 1995, 76.2% was incinerated, 11.5% was directly reclaimed and 12.3% was reused as resources and for other purposes. Out of the entire industrial waste output in the year 1995, 77% was put through intermediate treatment such as incineration, 13% was reused directly and 10% was directly reclaimed. In addition, 37.3% was ultimately recycled and 17.7% was reclaimed (*see* Fig. 2.3). As shown in Fig. 2.3, the amount of waste increased rapidly during year 1985 and year 1990 as a result of active consumption activities supported by the booming economy and has remained at a high level since then. Around 50 million tons of municipal solid waste were discharged in Japan per year. This figure has remained roughly the same for the past few years. For this reason, the volume reduction of waste has become a difficult but an important issue that must be resolved urgently for leading a healthy social life. The biggest problems with

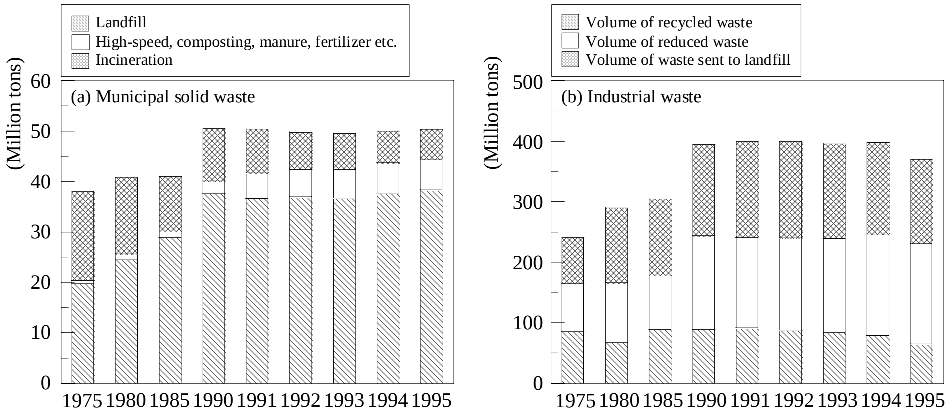

Fig. 2.3 Annual quantity and ratio of generation, treatment, and disposal of (a) MSW and (b) industrial waste (Data from Kankyo Sangyo Shinbunsha, 1995; Environmental Improvement Section, 1997; Office of Measures on Industrial Waste, 1997)

regard to treating this waste are difficulty in securing land for new treatment facilities and the limitations of disposal sites. This has brought about a recent shift away from the traditional focus on treatment and disposal. Instead, there has been more emphasis on the reduction, advanced treatment, and recycling of the waste. Recent years have also brought greater emphasis on control of chemical substances such as dioxins, prevention of global warming, strengthening legislation, and corporate self-regulation as stipulated in the Environmental Management Standards, International Organization Standardization 14000 (ISO 14000). Under these circumstances, the development of intermediate treatment technology for waste has been undertaken at a rapid pace in response to an increase in the amount of waste generation and changes in its quality. Remarkable progress is seen particularly in the incineration technology which is most effective for the volume reduction and stabilization of waste. One of the most distinctive features of Japanese municipal solid waste treatment is the comparatively high rate of incineration. Waste incinerators can be roughly classified into mechanical stoker types and fluidized bed types. Countermeasures against dioxins are the major problem in waste incineration. Those are handled with by the suppression of dioxin generation by efficient combustion, and the collection and decomposition of dioxins using bag filters (Committee for Studying Transfer of Environmental Technology, 1996). Regarding the issues of dioxin decomposition, prolonging disposal sites (detoxifying ashes, recycling), and improving heat recovery, a shift is expected from conventional incineration technologies to new technologies. These will include advanced stoker type incinerators combined with ash melting and solidification technologies, and gasification and ash melting technologies (Committee for Studying Transfer of Environmental Technology, 1996). The incineration rate of municipal solid waste reached 75.5% in year 1994 (*see* Fig. 2.3) and is expected to increase further due to the requirement to prolong the life of landfill sites (Environmental Improvement Section, 1997).

As Japanese social economic activities continued to develop and the society becomes more and more oriented towards the paradigm of mass-production, mass-consumption, and mass-disposal, many problems are emerging. The amount of waste abandoned was increasing; the component parts of this waste were becoming increasingly diverse. In year 1993, the amount of waste abandoned at landfill sites amounted to approximately 100 million tons (15 million tons of municipal solid waste and 84 million tons of industrial waste). Because of the mass-generation of waste and the environmental impact posed at every stage of its existence, from initial resource extraction to final disposal, it is especially important to promote waste recycling measures to reduce environmental impact caused by the use of materials and products. Current economic and social activities and lifestyles are dependent on mass-production-consumption-disposal activities which can be considered to be wasting resources and also energy, an increased impact is placed on the environment at every stage from resource extraction to disposal. Therefore, it is necessary, firstly, to revise the current production and consumption system, and reduce the generation of waste at all stages. Furthermore, in order to prevent environmental contamination, the amount of toxic substances in waste needs to be reduced.

If a waste is reused as it is, it generally contributes to the reduction of waste. At the same time, reusing an article means that the environmental impact which would normally occur in its production (at all stages, from initial material extraction to final production) are avoided (Environment Agency, 1997). Therefore, considering the impact on the environment, from a comprehensive viewpoint, if waste is going to be generated, then the reuse of the waste should be heavily promoted. In Japan, there are two laws that aim to establish a system for recycling and reusing, the “Recycled Resource Use Promotion Law” and the “Law for Promotion of Sorted Collection and Recycling of Containers and Packaging”. There is concern regarding the possibility that toxic substances such as heavy metal in reusing waste might contaminated soil or underground water supplies. More investigation should be performed to prevent environmental pollution being produced when reusing toxic substances.

2.2.4. Utilization of Waste in Geotechnical Engineering

Industrial wastes are usually classified based on the generating industrial processes. For geotechnical utilization, classification should include the fate and related characteristics of wastes. For example, waste materials after treatment are divided into three groups (Katsumi, 1996; Kamon et al., 2000): Those generated by excavating or crushing (surplus soil, waste concrete powder, waste rock powder), those generated from incineration or melting (coal ash, iron slag, incinerator ash), and those left “as it is” without any treatment (waste slurry or sludge, waste oil).

The first group of wastes is generated from construction works in large quantities. Construction work generates several types of waste. For example, excavation in urban areas for the construction of lifelines, subways, etc. and in mountainous areas for dams and tunnels produces enormous amounts of surplus soil and waste sludge. Geotechnical engineers are expected to contribute in saving-resources and waste utilization, since these industries are responsible for the consumption of resources, manufacturing of goods, and discharging large amounts of waste materials.

Residues are generated by incineration or melting, which means thermal power generation, iron and steel smelting, and incineration treatment of waste sludge and municipal waste. The characteristics of these wastes depend on their raw materials, incineration temperature and time, and boiler system. They are classified roughly as: fly ash collected from fuel gas, bottom ash left at the bottom of boiler, and slag produced by melting. These wastes categorized in the first and second groups are considered to be stabilized by compaction or chemical additives, namely solidification, after which they can be utilized as road materials or in embankment.

The last group contains waste sludge, waste oil, waste plastics and so on, the treatment of which are very difficult for various technical and economic reasons.

For efficient utilization, the properties and generating conditions of various wastes must be taken into consideration. The former includes whether the waste material is inorganic or organic, whether it contains heavy metals, and so on. The latter refers to when, where, and how many waste materials are generated.

The main wastes are slag from the iron and steel industry, sludge from the chemical, paper, and glass industries, and coal ash from the electric supply works. In addition to construction rubbish, large amounts of waste sludge and surplus soil are discharged during the foundation works and dredging works.

Intermediate treatment methods include dehydration, screening, crushing, aggregating, solidification, combustion, and melting. Combustion or incineration, which realizes volume reduction and sanitary resolution against harmful substances, is becoming more widely used. By reducing the organic content by incineration, the material will be stable over time as well. Sludge, which has high water content, is reduced in volume by dehydration as well as combustion while the melting method has been developed mainly for the treatment of sewage sludge. Unfortunately, rubbish from construction works is scarcely reduced by intermediate treatment. Table 2.1 shows the possible treatment methods of wastes, and their applications. Slag and dust are reused, but sludge and construction rubbish generated in large quantities have not been well utilized.

2.2.5. Potential Utilization of Waste Slurry or Sludge

Katsumi (1996) and Kamon et al. (2000) investigated the amounts of industrial wastes, including waste slurry or sludge, generated in Japan, and they suggested and developed the reusing technology based on the environmental geotechnology for the industrial wastes. The investigation of the feasibility of two kinds of dehydrated waste sludge materials as landfill cover materials performed in the present research, includes one aspect of utilization of the

Table 2.1 Treatments and uses of wastes and by-products (adopted from Kamon, 1998; Kamon et al., 2000)

Treatment method	Non-treatment	De-hydration	Crushing	Screening	Aggre-gating	Solidi-fication	Com-bustion	Melting
Embank-ment	CA PSIA	WCS	WC WT	PSIA WRP WES	CFA SIA			MSWIA
Space filling					CFA	WCS CFA SSIA		
Slope cover	WT			PSIA				
Road pavement	SS		WC WT WAC	MSWIA	CA CFA	WCS		MSWIA SSIA
Aggregate	CFA		WC WAC	SS CS CA	CA		CFA WCS	MSWIA SSIA PSIA
Light-weight fill	CFA PSIA		WP WEPS			CFA	WCS	MSWIA
Stabilizing agent	BFS			CFA			MSWIA	
Cement material	CA			CA			MSWIA	
Concrete product	BFS MSWIA							MSWIA

Note: BFS; Blast furnace slag,
SS; Steel slag,
CS; Copper slag,
CA; Coal ash,
CFA; Coal fly ash,
PSIA; Paper sludge incinerated ash,
SSIA; Sewage sludge incinerated ash,
MSWEA; Municipal solid waste incinerated ash,
WCS; Waste construction slurry,
WRP; Waste rock powder,
WC; Waste concrete,
WT; Waste tire,
WAC; Waste asphalt concrete,
WP; Waste plastic,
WEPS; Waste expanded polystyrene.

waste sludge based on the environmental geotechnology.

As the background, large amounts of by-products are generated from construction works. Surplus soil can be dealt with in three ways. Good quality surplus soil, such as sandy soil, is utilized as filling or for embankments without any treatment. Other kinds of surplus soil are reused after improvement. Lastly, some surplus soil is disposed of without being used with/without treatment. This is because an effective treatment system has not been established. Table 2.2 shows the classification of surplus soils. Slurry or sludge in the bottom line in Table 2.2 is regarded as industrial waste. Although some waste slurry or sludge is reduced by intermediate treatment such as dehydration, the vast majority of it, together with dehydrated slurry are placed in disposal sites. One problem with waste slurry utilization is that the material treated for utilization is often regarded as waste and have to be disposed of in designated areas. Another problem is that we cannot divide these by-products into valuable soil and waste slurry and this will led to the illegal dumping of large amounts of waste sludge.

Waste slurry is a dredged spoil and by-product of the cast-in-place concrete pile method, continuous diaphragm walls method, shield tunneling method, and so on. It cannot be discharged into rivers and seas and cannot be utilized in embankments as soil material.

Table 2.2 Classification of surplus soil for construction works

Class	Cone Index, q_c (kPa)	Compressive strength, q_u (kPa)	Content	Use
1st class soil	-	≥ 500	Sand, gravel, and the corresponding	# Back filling for construction work # Back-fill for structure # Road embankment # Fill for building lot
2nd class soil	≥ 800	≥ 200	Sandy soil, gravelly soil, and the corresponding	# Back-fill for structure # Road embankment # River dike # Fill for building lot
3rd class soil	≥ 400	≥ 100	Clay soil which can be executed on, and the corresponding	# Back-fill for structure # Road subgrade embankment # River dike # Fill for building lot # Water area reclamation
4th class soil	≥ 200	≥ 50	Clay soil, except for 3rd-class soil	# Reclamation in coastal area
Sludge	< 200	< 50	Waste sludge / slurry	# No use

Table 2.3 Examples of effluent standards in Japan

	Water Pollution Control Law	Sewage Law	Environmental Standard	
			River & Lake	Sea
SS (mg/L)	200	600	below 1 - below 25	-
PH	5 - 9	5 - 9	6.5 - 8.5	7.8 - 8.3
BOD (mg/L)	160	600	below 1 - below 10	-
COD (mg/L)	160	-	below 1 - below 8	below 2 - below 8
Mineral oil (mg/L)	5	5	-	-
Animal oil (mg/L)	30	30	-	-

Wastewater includes water that discharges from tunnels and rainwater collected in land development areas. Waste slurry and water are generally subjected to proper intermediate treatment, after which the treated water can be released into rivers or as sewage according to the environmental criteria (for example, listed in Table 2.3). Unfortunately, meeting the criteria for SS (suspended solids), pH, COD (chemical oxygen demands), and oil content are generally difficult (Kamon et al., 2000). Soils and cakes produced by treatment are transported to landfill sites for disposal.

Utilization of waste slurry or sludge can be classified relating to the treatment methods as shown in Fig. 2.4 (Kawachi et al., 1996; Kamon et al., 2000). They are summarized as: (1) flowable materials for grouting or excavating slurry, (2) earthen materials

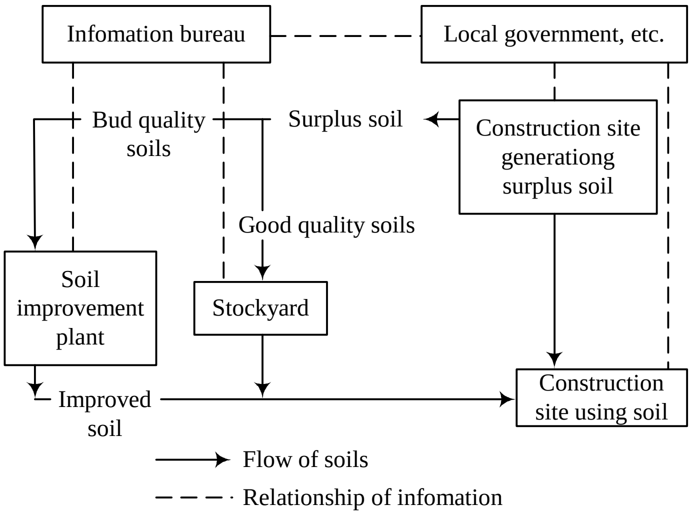

Fig. 2.4 Concept of treatment method and application of construction waste sludge (adapted from Kawachi et al., 1996)

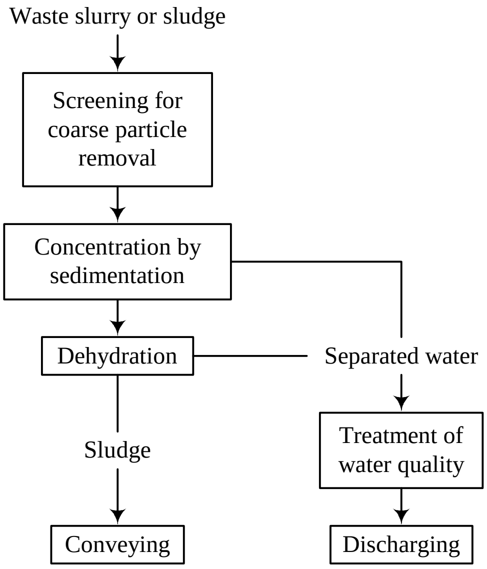

Fig. 2.5 Typical management flow of waste slurry or sludge (adapted from Kamon et al., 2000)

for embankment or backfilling, (3) clay materials for cement or ceramic manufactures, and (4) granular materials for aggregate. Figure 2.5 shows the general flow of waste slurry or sludge treatment. Some processes can be omitted and others must be added, depending on the characteristics of the waste, environmental criteria and the applicability of dehydrated soil or discharged water (Kamon et al., 2000). Tsukada and Ogawa (1996) also proposed ongoing activities for waste sludge utilization as shown in Table 2.4. Kamon and Katsumi (1994b)

Table 2.4 Ongoing activities of creative reuse of waste slurry or sludge (adopted from Tsukada and Ogawa, 1996)

Use	Type of waste	Treatment method
· Backfill, etc.	· Clay · Waste slurry	Sediments obtained by the coagulating sedimentation method from clay left after sand gravel have been removed from excavated material or from slurry used in reverse circulation piles are dewatered using filter press. The dewatered material is granulated with an extrusion type granulator and baked in a rotary kiln to 20 - 30 mm grains.
· Base course, etc.	· Sludge · Concrete fragments	Mixtures of crushed concrete and sludge or mud are stabilized with a cement stabilizer, and hardened mixtures are broken up.
· Lightweight aggregate	· Sludge	Powder of spontaneous combustibles is added to sludge or mud, and the mixture is let to burn in a vertical or other type of kiln. Since the combustible will burn away, the mixture will become a porous material usable as lightweight aggregate.
· Lightweight aggregate · Horticultural soil · Filter material	· Sand pit sludge · Sludge from stone cutting · Purification plant sludge · Bottom sediment	A foaming agent is added to a mixture of sand pit sludge and sludge produced by stone cutting or other materials to form 3 - 30 mm grains. The grains are cried, baked in a rotary kiln and cooled in a rotary cooler. Lightweight aggregate can be obtained by sieving the material thus obtained.
· Lightweight aggregate · Horticultural soil · Filter material	· Quarry sludge	Dewatered cakes of quarry sludge (sandstone, shale) and purification plant sludge are crushed and re-formed into 0.6 - 20 mm grains. These grains are being dried. The baked and foamed grains are then cooled in a rotary cooler and sieved to produce a material usable as lightweight aggregate, etc.
· Lightweight aggregate · Horticultural soil · Filter material	· Wastes containing cement	Dewatered cakes of cement-containing waste liquid are dried and pulverized. Aluminum sludge and calcareous stone powder are added to the pulverized material. After the mixture is baked and rapidly cooled, it is pulverized again to obtain this hydraulic material.
· Molten slag	· Sludge	Dried sludge and 15 % or so steel dust are mixed and molten in a furnace. The molten mixture in put into alkaline hot water, and the solids thus obtained are crushed with an impact crusher to obtain slag aggregate.
· Base course, etc.	· Self-hardening sludge (cement-containing)	Porous and highly absorptive sand-like material manufactured by baking old paper ash is added to and mixed with self-hardening (cement-containing) sludge. The mixture is then improved to obtain a material usable as base course material, etc.

proposed the conceptual outline of waste sludge utilization system. It involves dehydration and solidification and results in efficient treatment, decrease in volume, stabilization, and recycling of resources. The selection of treatment method is carried out based on the qualities of waste sludge; the density and funnel-viscosity, universally measured to determine the character of sludge at the excavation sites. Density indicates the solid content of sludge. The funnel-viscosity is increased by the remaining bentonite and dispersant, which indicate the possibility for or the effectiveness of dehydration treatment. Attempts for volume reduction by dehydrating a high solid content sludge is not always the best strategy from technical and economical point of view. Sludge with low density can be dehydrated easily, but sludge with high viscosity is difficult to dehydrate even if it has low density, because of the remaining dispersants. With the use of a high pressure dehydrator, the strength of dehydrated cakes can easily increase, and thus be directly utilized as embankment and subgrade material. Kamon et al. (1998) proposed a continuous dehydration-solidification treatment system by introducing a parameter, w/w_L (water content of the sludge normalized by the liquid limit). Application of the treated sludge as an earthen material is reviewed in relation to the treatment level. Kawaguchi et al. (1998) developed a new effective treatment system for dewatering the waste bentonite slurry. They introduced a unique polymer flocculant to have higher efficiency in lowering water content.

2.2.6. Comparison of Waste Management in Japan and the United States

The disposal of solid waste continues to be one of the more serious and controversial urban issues facing local governments in both Japan and the United States. Despite innovative technologies, production decisions and marketing strategies that have helped in better managing solid waste, per capita generation of garbage continues to surge (Japan Local Government Center in New York, 1997). Local governments in both countries currently dispose of solid waste through three main strategies: composting, incineration, and land filling. Composting, a process involving bacteria as an agent to decompose waste materials into soil additives, removes leaves and yard waste from the waste stream, thereby lessening volume. Incineration involves the burning of solid waste, and not only effects significant volume reduction but also produces energy in the form of steam or electricity. Because other disposal alternatives leave some portion of the waste untreated or produce some type of residue, landfills serve as the terminal approach to waste management. Although using the same methodologies, solid waste management practices still differ widely between the United States and Japan. Due to geographic considerations, Japan burns more than 70% of its solid waste, while more than 84% of waste goes into landfills in the United States (*see* Fig. 2.6). Unfortunately, both methods may contribute to increase in pollution. With its heavy reliance on incineration, Japan must deal with dangerous air emissions and ash disposal. Incinerator smoke contains such toxic substances as dioxins, sulfur dioxide, and oxides of nitrogen. Recent research has found the concentration of dioxins in atmosphere in Japan is three times that of the United States (Japan Local Government Center in New York, 1997). Landfills are also environmentally challenging. They contain significant contaminants that can pollute underground aquifers and surface water. Eventually, landfills reach capacity and many are being capped at great government expense. As a case in point, the Solid Waste Management Plan for the City of New York includes a firm commitment to close Fresh Kills, the most celebrated landfill in the United States, by the end of the year 2001 (Japan Local Government

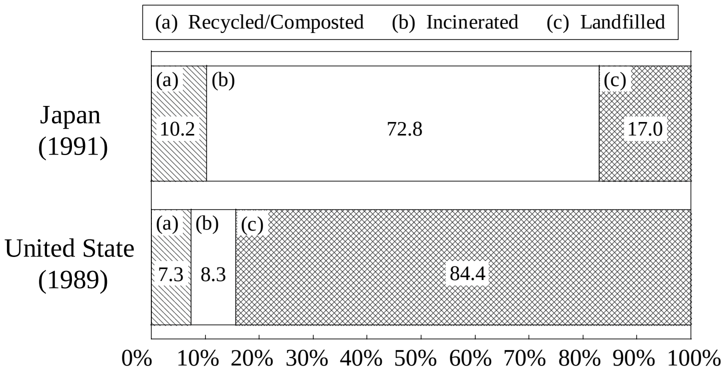

Fig. 2.6 Difference in solid waste disposal between Japan and the United States (adapted from Japan Local Government Center in New York, 1997)

Center in New York, 1997). Technology exists to remove toxic chemicals from incinerator smoke, and to prevent landfills from leaking, but it is often expensive and subject to strict federal and state regulation. The laws of United States authorize local governments to participate in preparing and implementing local plans that embody sound principles of solid waste management, natural resource conservation, energy production, and employment opportunities. Solid waste can also be managed voluntarily by reducing waste and recycling. Reducing waste output is becoming a major environmental priority. Many local governments in both countries now require recycling and waste reduction by residents and businesses as the cornerstone of an effective solid waste program.

2.3. Landfill Technologies

2.3.1. Historical Development of Waste Landfill Technology

The changes in laws and regulations for waste disposal in Japan are shown in Table 2.5. Before the 1960s in Japan, the disposal of waste was dependent on “anaerobic landfill”, which merely abandons the waste to the depressions in the ground and to the place of shallow water along the coastal without any cover soil or leachate treatment facility, as shown in Fig. 2.7 which illustrated the historical classification of landfill type. The primary design of the anaerobic landfill was highlighted on only to reduce the waste odors, blowing litter, and to control insects and rodents. Accordingly, the concerns over the effect of anaerobic landfill on the surrounding environments were not poured out. However, securing landfills became increasingly difficult with owing to remarkable economic growth after the World War II, popularization of chemical fertilizers, and rooting of hygienic concept in Japan. It brought about problems such as offensive odors, fly infestation, and groundwater pollution by leachate. The “Filth Cleansing Law” was abolished in year 1954 for this reason, and the “Public Cleansing Law” was enacted with the main objective of promoting the incineration treatment

Table 2.5 Changes in laws and regulations related to landfill site in Japan

Year	Laws and regulations
1900	Filth Cleansing Law
1954	Public Cleansing Law
1970	Waste Disposal Law
1976	Revision of Waste Disposal Law
1977	Technical Standard for Landfill Sites
1979	Guideline for Landfill Site
1988	Amendment to Guideline for Landfill Site
1991	Amendment to Waste Disposal Law
1997	Amendment to Waste Disposal Law
1998	Amendment to Technical Standard for Landfill Site

of waste.

In the 1960s, there is no effects of establishment of "Public Cleansing Law" and a large volume of waste was generated with the high economic growth. Furthermore, an air pollution by factory smoke and a water pollution by factory waste-water were actualized. The pollution-induced diseases such as Minamata diseases (caused by methyl mercury), Itaiitai disease (caused by cadmium), and Yokkaichi asthma all emerged during this period. Meanwhile, the landfill, in where large quantities of waste is being reclaimed, caused a serious problem despite the establishment of legal system for promoting incineration treatment with new facilities, as the construction of incineration facilities was not able to keep pace with the rapid increase in waste generation. In year 1964, the leakage of polluted water from a landfill in Tokyo Metropolis was revealed and fly infestation became an issue the following year. It was in response to such background that the application of cover soil at waste landfills was started and leachate treatment facility was gradually constructed. In a sense, it was a period when the structure of waste landfill started changing from anaerobic landfill to "anaerobic sanitary landfill", in which cover soil was applied regularly.

In the 1970s and the 1980s, the "Technical Standard for Landfill Sites" amended in year 1977 clarified the landfill regulations represented by the "anaerobic sanitary landfill". In the amended Technical Standard, however, the guidelines for the waste containment facilities were not concretely described. This is because the waste of those days consists of mainly organic substances; even if leachate from the waste permeates underground, it expected to be diluted under the natural attenuation of subsurface soils. The design of the "anaerobic sanitary landfill" was "dilute and disperse" (Zhang, 2002). This assumed that within the groundwater the concentrations of any contaminants derived from landfill would reduce to acceptable levels as they dispersed and were diluted under natural process. Many dilute-and-disperse sites were constructed in former sandstone quarries with high permeability. Subsequently, the quality of disposed waste changed to inorganic, as from the growing of incineration treatment of waste. In year 1988, the "Guideline for Landfill Sites" was enacted and declared more

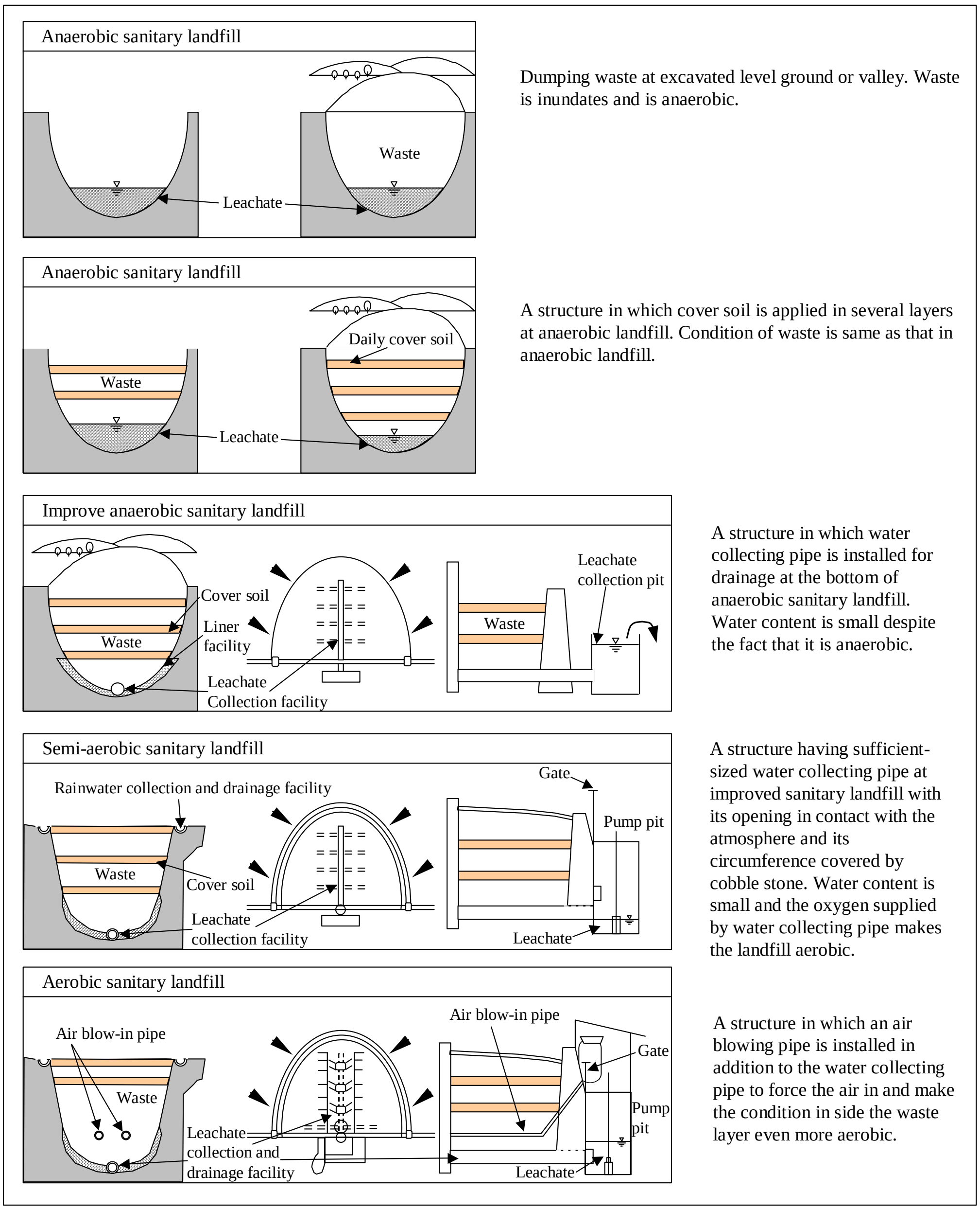

Fig. 2.7 Classification of landfills on land according to the major microbial reparative stage

concretely the standards for landfill facilities. For the waste containment facilities, although the importance for an impermeable sheet was stated, this guideline was still not clearly announced.

More recent findings have proven that these natural mechanisms do not fully protect the environment from landfill gas or leachate, because dilution simply reduce the concentration of leachate constituents and dispersion is a mechanical phenomenon that has no effect on the toxicity of pollutants (Zhang, 2002). Subsequently it has become accepted that many of these old “anaerobic sanitary landfills” represent potential environmental hazards. As the result that generalized such acceptance, the “Technical Standard for Landfill Sites” amended in year 1998 announced the clear standards for bottom liner systems as well as comprehensive regulations of landfill structures. However, the installation standard of a cover system still was not clarified. If the above-mentioned findings are more taken into consideration, establishing the installation standards for cover systems in landfill is immediately discussed as the most major subject and should be systematized.

2.3.2. Classification of Landfill Sites

In the waste management of Japan, landfills are classified, under the amendment of “Technical Standard for Landfill Sites", as Least-controlled, controlled, or strictly controlled, as shown in Fig. 2.8. Isolated landfills are used for the disposal of hazardous industrial wastes. Leachate-controlled landfills are used to dispose of both municipal wastes and industrial wastes other than hazardous and stable wastes. Non-leachate-controlled landfills are used to dispose of stable wastes, namely, waste plastics, rubber scrap, metal scrap, waste glass, and ceramics and demolition waste. The standards for landfill site structure and those for landfill site operation and maintenance have been developed in accordance with landfill type.

The landfill site structure, especially in the leachate-controlled landfill structure as shown in Fig. 2.8, intends that a cover system is not required as one of the waste containment facilities in Japan. This is because an aerobic and a semi-aerobic landfilling prevails in Japan, where landfill sites are expected to become stable enough for land utilization in a short time after landfill completion (Hanashima and Furuichi, 2000). The philosophy of the semi-aerobic landfilling method is to allow as much infiltration as would practically occur. This would bring the landfill to field capacity quickly and allow the removal of a large proportion of contaminants by the leachate collection system. Much infiltration is helpful to maintain an aerobic respiration within landfill, speeding up the decomposition of organic materials. The disadvantages of this approach are three-fold: Firstly, larger volumes of leachate must be treated; this has economic consequences for the proponent. Secondly, if the leachate collection system fails (e.g., clogging of the drainage pipe), a high infiltration will result in significant leachate mounding. Thirdly, the complete biodegradation of organic waste cannot be expected, especially in recent years that waste at landfill sites has been changed to incinerator ash from the organic raw waste.

There are various philosophies to approach the design and management of a landfill as pointed out by Rowe et al. (1995), among which the role of cover system should be noted. One is to provide a cover system as impermeable as possible and as soon as possible after the landfill has ceased operating, so as to minimize the generation of leachate. This approach has the benefits of minimizing both amount of leachate that must be collected and treated, and the mounding of leachate within the landfill. Anaerobic decomposition due to installation of the cover system provides a reducing condition within the landfills, which is favorable to the fixation of heavy metals, tending to decrease the pollution risk of leachate to the nearby

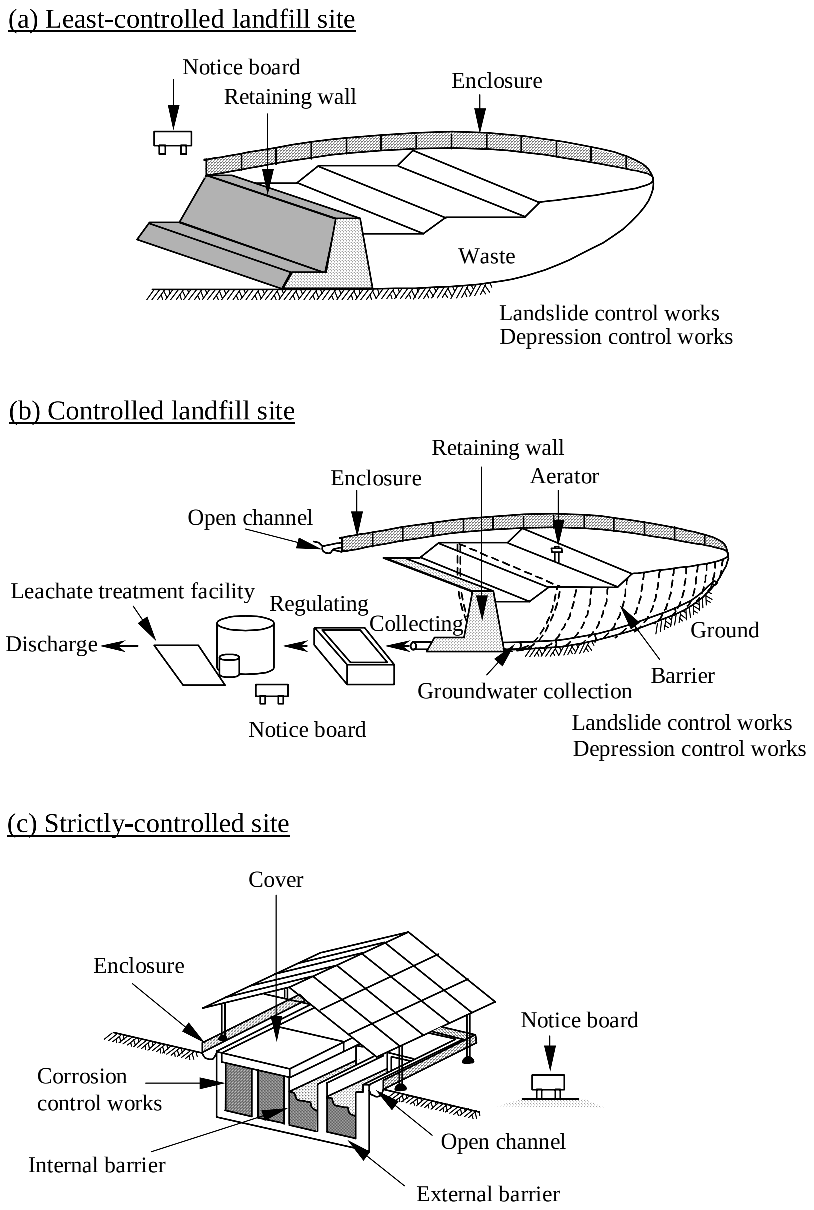

Fig. 2.8 Types of waste containment facilities in Japan

environment. It also has the disadvantage of extending the contaminating lifespan. With low infiltration, it may take decades to centuries before the field capacity of the waste is reached and full leachate generation to occur.

2.3.3. Impacts of Landfill on Geo-Environment

Where landfill technology are used judiciously and the design and management are of a high standard, this technology of waste disposal can be effective in safeguarding people and the environment from any damage that wastes might cause. Where this is not the case, serious air, water, and soil pollution may occur.

Figure 2.9 indicates the general issues associated with landfills and protection of the surrounding environment. The infiltration of rainwater into landfill, together with the biochemical decomposition of the wastes, produces a leachate which is high in suspended solids and of varying organic and inorganic content. All municipal and most industrial wastes will produce leachate. If the leachate infiltrates surface or groundwater before sufficient dilution has occurred, serious pollution incidents can occur. In surface waters, leachate with high in organic material and reduced metals will cause severe oxygen depletion and result in fish-kills. Leachate with high in non-biodegradable synthetic organic compounds is a particular threat: through bioaccumulation, concentrations of these substances may increase to toxic levels and endanger animal and human life. If leachate enters groundwater or shallow aquifers, the problems are more intractable. The dilution and removal of leachate is much slower in groundwater than in surface water and it may render the groundwater non-potable for the expected future. The contamination of groundwater is a serious problem of immediate concern. The pollution of shallow aquifers with high concentrations of chemicals can contaminate the soil and render an area uninhabitable. Consequently, the establishment of sophisticated containment facilities in landfill site is critical issue, in order for reducing the impacts caused by the landfill on the surrounding groundwater.

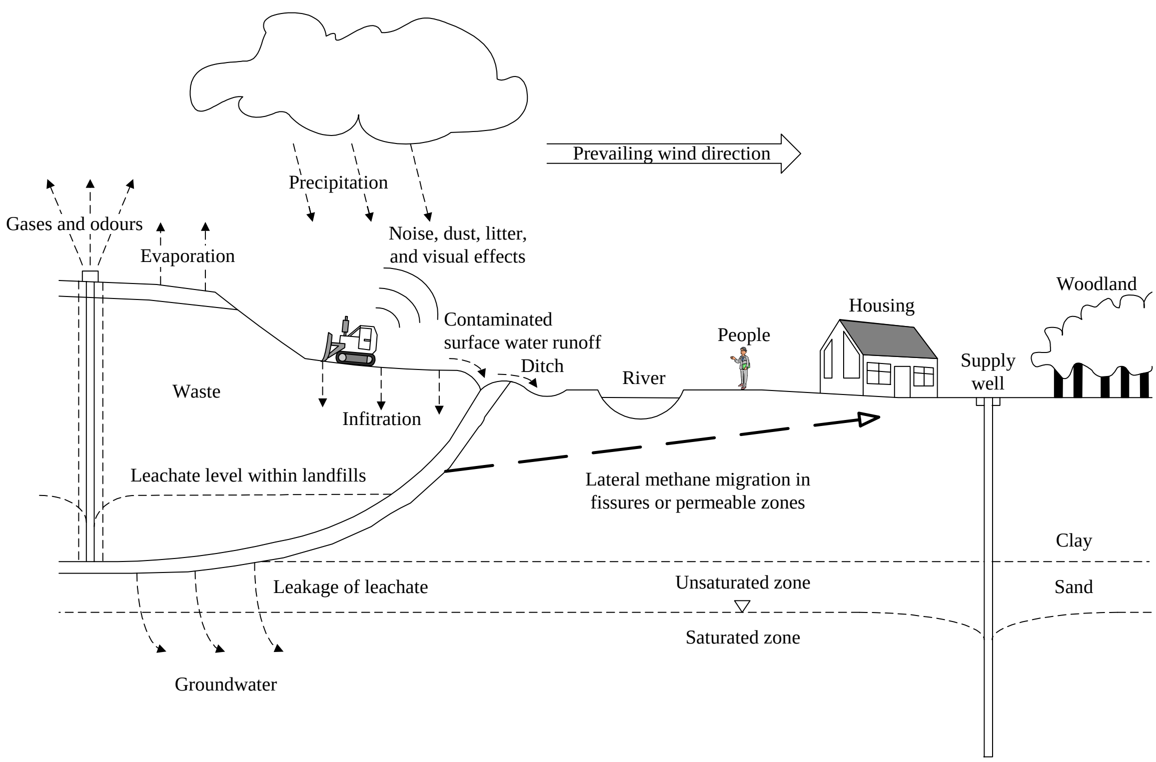

Fig. 2.9 General issues associated with landfills

2.3.4. Design of Waste Containment Facilities

The design of waste containment facilities typically involves some form of “barrier” that separates the “waste” from the surrounding groundwater environment. This barrier is intended to minimize the migration of contaminants from the landfill, thus the environmental impact of the landfill is intimately related to its design and long-term performance (Rowe, 1995). The waste containment facilities are used for two purposes in landfills: as cover systems to minimize leachate generation and surface water contamination by providing a barrier from precipitation and other percolating waters, and as side and bottom liner systems to contain leachate and minimize its downward migration into underlying groundwater. Manassero et al. (2000) summarized the basic components of these containment facilities as illustrated in Fig. 2.10. The components of these facilities are similar since they are both barriers; however, there are differences in regulatory requirements and other design- and performance- related issues. One of the major geotechnical progresses in modern solid waste containment facility within last decade is the introduction of new liner system (Manassero et al., 2000), as pollutant containment barriers including geosynthetic clay liners (GCLs) and the composite

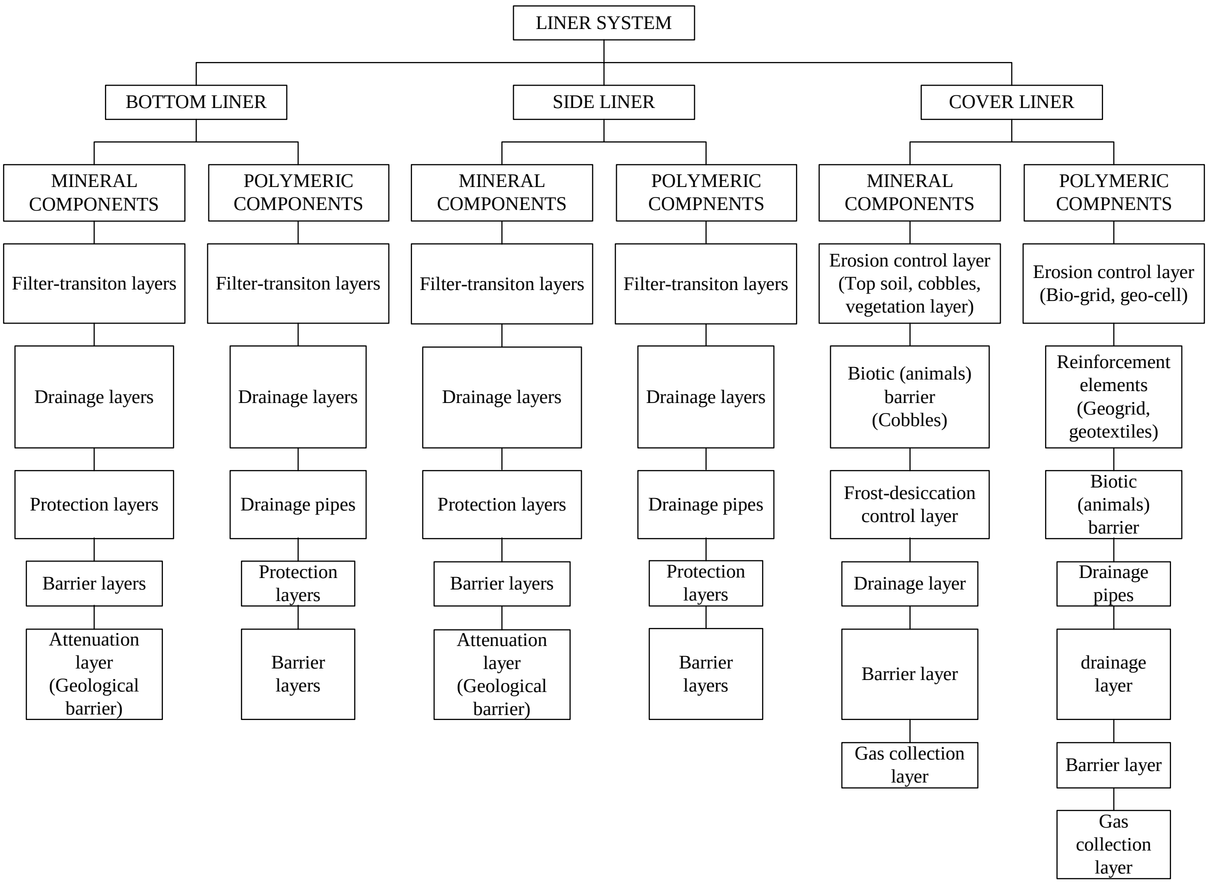

Fig. 2.10 Components of solid waste containment systems (adopted from Manassero et al., 2000)

barriers consisting of compacted clay liners (CCLs) or geosynthetic clay liners (GCLs), placed in close contact with a geomembrane (GM). Figure 2.11 shows a practical example of composite liner system used in Hinode landfill site, Tokyo, Japan (Kamon, 1999).

Kamon and Katsumi (2001) compared the representative regulations of bottom liner system for municipal solid waste landfills in some countries, as shown in Fig. 2.12. In Japan, natural clay ≥ 5 m in thickness with hydraulic conductivity ≤ 1 x 10^{-5} cm/s can be utilized as a liner, according to the amendment of “Technical Standard for Landfill Sites”, for controlled landfill. Otherwise, it requires one of three following liner systems: (1) two geomembranes (GM), which sandwich a non-woven fabric or other cushin materials, (2) a GM underlain by an asphalt-concrete layer ≥ 5 cm in thickness with hydraulic conductivity ≤ 1 x 10^{-7} cm/s, or (3) a GM underlain by a clay liner ≥ 50 cm in thickness and having hydraulic conductivity ≤ 1 x 10^{-6} cm/s (Kamon and Katsumi, 2001). It is argued that these requirements for controlled landfill in Japan are not enough for comparing those in foreign countries (*see* Fig. 2.12), also, from the engineering point-view (Kamon, 1999).

Containment facilities for cover systems are different than bottom liner systems because they provide a barrier from water rather than leachate. The chemical resistance required for cover systems is therefore less than that required for bottom liner systems. Cover systems are, however, more susceptible to durability and exposure concerns, such as clay desiccation, erosion, freeze-thaw conditions, burrowing animals, and root penetration (Sharma and Lewis, 1994).

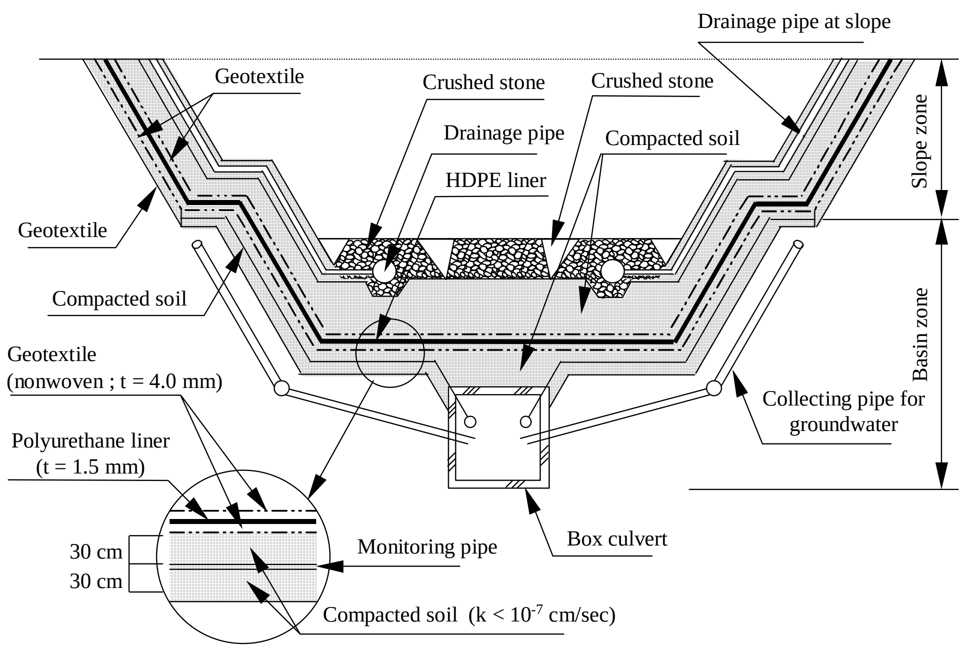

Fig. 2.11 Composite bottom liner system used in Hinode landfill site, Tokyo, Japan (adopted from Kamon, 1999)

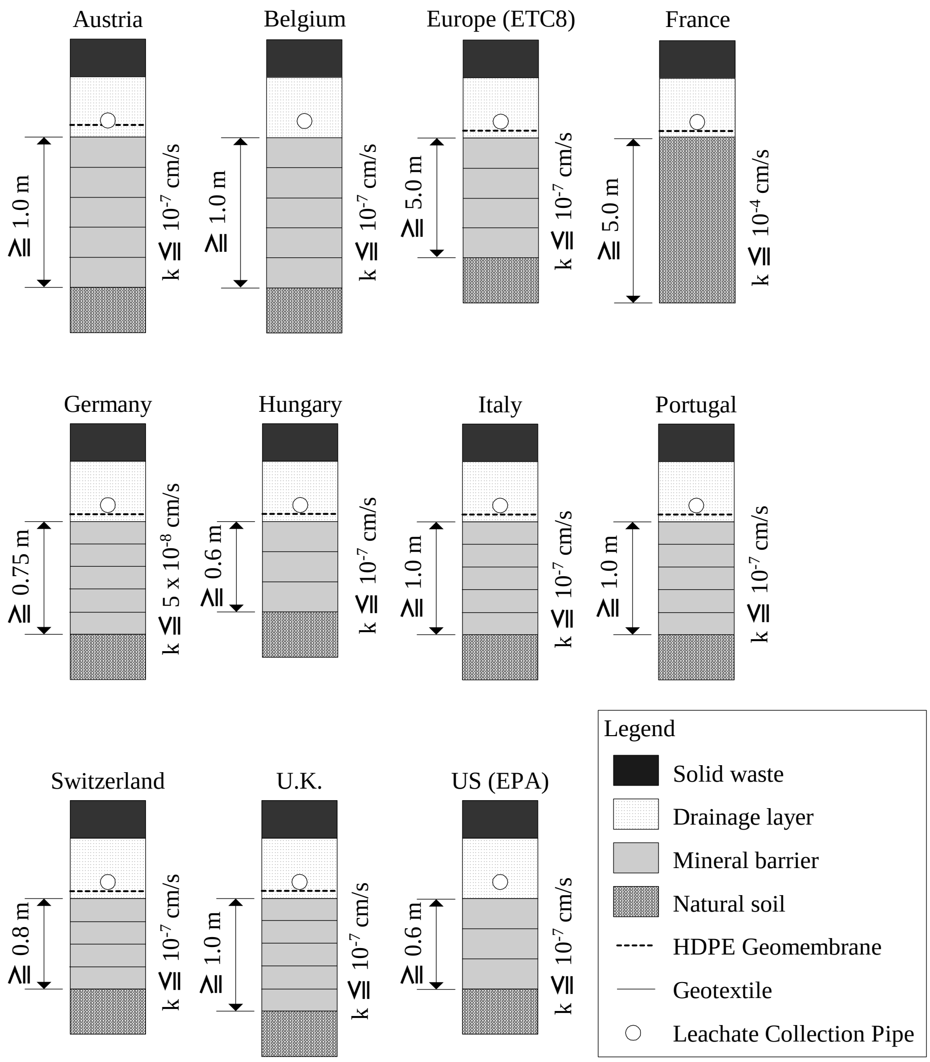

Fig. 2.12 Bottom liner system for MSW landfills in some countries (modified from Manassero et al., 1997)

2.4. Design of Landfill Cover Systems

2.4.1. Significance of Cover Systems

Water migration patterns around and within landfills must be carefully evaluated to effectively control water movement in landfills located in areas, where water might enter from surrounding surface. Landfills often are located on sloping or valley topographies that cause water to flow downhill and, if not carefully managed, into the landfill. Water that falls directly onto the surface of landfill while the landfill is open and after closure also must be managed. This water either will pass through the top of the landfill and into the waste, run off of the landfill, or evaporate. Controlling these natural processes is critical to achieving environmentally sound and operationally effective landfills. Consequently, the characteristics of landfill cover system that is placed over the waste during operational phases and after closure will determine how much water enters the landfill.

Generally, cover systems have been recognized as a critical component in landfills. The cover system separates reclaimed waste or contaminated material from the surface environment, restricts infiltration of water into the waste, and in some cases limits release of gas from the waste. If the objective is prevention of pollution to ground water, then one obvious strategy is to minimize the amount of water percolating through the cover system (EPA, 1989). In the United States and most European countries, it has already been recognized that the installation of cover and bottom liner systems as landfill containment facilities is an effective method of water interception for the prevention of leachate migration. The construction criteria and the evaluation techniques for landfill cover systems have already been established by many investigators (e.g., Daniel, 1995; Daniel and Koerner, 1995). In the United States, the EPA distributes the minimum criteria for the construction of final cover systems based on Subtitle D of Resource Conversation and Recovery Act (RCRA) in year 1992. According to the recommended construction criteria of the EPA, the thickness of the constituent layers of final cover systems and the standards for the hydraulic conductivity of barrier layers are recommended and are considered as shown in Fig. 2.13. The concept of this regulation is the prevention of the "bathtub effect" in which water accumulates in the waste layer. Daniel and Koerner (1995) reported that the hydraulic conductivity of a barrier layer in a cover system for a municipal solid waste (MSW) landfill has been determined at lower than or equal to 1 x 10^{-5} cm/s. In addition, a barrier layer, which has a level of hydraulic conductivity lower than 1 x 10^{-7} cm/s can perform excellently for all types of landfills. In general, it can be expected that final cover systems and bottom liner systems have similar levels of hydraulic conductivity (Parker et al., 1993).

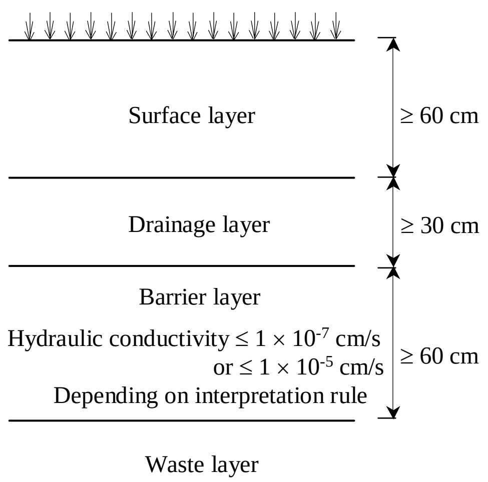

Fig. 2.13 Construction criterion for cover systems recommended by the US EPA

Japan is a country where there is no governmental requirement to design a modern cover system with low hydraulic conductivity in landfills. Therefore, the present research emphasizes the importance of cover system from the worldwide successful application of it, reducing greatly the rainwater penetrations into landfills and so as to decrease the pollution risk of leachate to the nearby environment.

2.4.2. Purposes of Cover Systems

Landfill cover systems are divided into final cover systems and daily cover systems. Final cover systems are designed as an impermeable cap on the top of landfill after the closure of landfill operations. A final cover system should semi-permanently prevent the infiltration of rainwater into the underlying waste layer, while a daily cover system should suppress the infiltration of rainwater into the waste layer during the waste reclamation stage.

The primary purposes of final landfill cover systems are: (1) to minimize the infiltration of rainwater and melted snow into the landfill after the landfill has been completed, (2) to limit the uncontrolled release of landfill gases, (3) to suppress the proliferation of vectors, (4) to limit the potential for fire, (5) to provide a suitable surface for vegetation at the site, and (6) to serve as the central element in the reclamation at the site. To attain these goals, landfill final cover systems must be able to: (1) withstand climatic extremes (e.g., hot/cold. wet/dry, and freeze/thaw) (2) resist water and erosion, (3) maintain stability against slumping, cracking, slope failure, and downward slippage or creep, (4) resist differential landfill settlement caused by the release of landfill gas and the compression of waste and foundation soil, (5) resist deformation caused by earthquakes, and (6) resist disruptions caused by plants, burrowing animals, worms, and insects (Hatheway and McAney, 1987; Koerner and Daniel, 1997; Tchobanoglous et al., 1993). It is important that legislation should exist which corresponds to attaining these above-mentioned goals and that it must continue to be updated in the future as necessary.

Daily cover systems are used to cover the waste which is discarded each day in order to eliminate the harboring of disease vectors, to enhance the aesthetic appearance of landfill sites, and to limit the quantity of surface infiltration (Tchobanoglous et al., 1993). The daily cover systems also serve to resist failure due to landfilling operations such as surcharge loads brought about by stockpiling and the driving of collection vehicles across completed portions of the landfill. Some of the water, in the form of rain or snow, enters while the waste is being placed in the landfill. However, the placement of a daily cover system can limit the quantity of surface water that enters a landfill. Although daily and final cover systems are somewhat structurally different, the goal of preventing the infiltration of water into the underlying waste layer is the same.

2.4.3. Hydraulic Barrier in Cover Systems

Materials commonly used in landfill final cover systems are listed in Table 2.6. Analysis and design methods have been developed to evaluate the degree to which these materials, alone or in combination, can satisfy the functional requirements (Othman et al., 1995).

The barrier layer (often called "hydraulic" barrier) is generally viewed as the most critical component of a cover system. The barrier layer minimizes percolation of water through the cover system directly by blocking water and indirectly by promoting storage of drainage of water in the overlying layers, where water is eventually removed by runoff, evapotranspiration, or internal drainage. Furthermore, the barrier layer prevents landfill gases from escaping into the atmosphere. Such gases have been shown to be major source of air pollution and ozone depletion.

In the United States, many regulatory agencies have traditionally required a low-hydraulic conductivity, compacted soil liner (or the equivalent) as the primary hydraulic barrier layer within landfill cover systems. The thickness of compacted soil liner typically ranges from 30 to 60 cm, and the maximum allowable hydraulic conductivity is typically 1 x 10^{-7} cm/s. Geomembranes have been required by the EPA for hazardous landfill since year 1993. However, the MSW rules allow states to approve alternative designs. Geosynthetic clay liners (GCLs) are not required; the issue is whether they ought to be allowed as a substitute for compacted, clayey soil. Table 2.7 summarizes the relative advantages and disadvantages of the three barrier materials. Environmental stresses such as freeze-thaw, wet-dry, and distortion caused by differential settlement are much more damaging to low-hydraulic

Table 2.6 Potential materials for cover systems (adopted from Othman et al., 1995)

Surface layer · Top soil · Geosynthetic erosion control layer over top soil · Cobbles · Paving material · Others	***Barrier layer*** · Compacted clay · Geomenbrane (GM) · Geosynthetic clay liner (GCL) · Geomembrane/compacted clay composite · Geomembrane/GCL composite · GCL/compacted clay composite · Others
Protection layer · Soil · Cobbles · Others	***Gas collection layer*** · Sand · Gravel · Geotextile · Geonet · Others
Drainage layer · Sand · Gravel · Geonet · Others	***Foundation layer*** · Soil · Select waste · Others

Table 2.7 Principal advantages and disadvantages of barrier materials (adopted from Daniel, 1995)

Material	Advantages	Disadvantages
Compacted soil with low hydraulic conductivity	1. Long history of use, 2. Regulatory approval is virtually assured, 3. Large thickness ensures that layer will not be breached by puncture, 4. Large thickness provides physical separation between waste and surface environment, and 5. Cost is low if material is locally available.	1. Soil can desiccate and crack, 2. Liner must be protected from freezing, 3. Low resistance to cracking from differential settlement, 4. Difficult to compact soil above compressible waste, 5. Suitable soils net always locally available, 6. Difficult to repair if damaged, and 7. Slow construction.
Geomembranes (GM)	1. Rapid installation, 2. Virtually impermeable to water if properly installed, 3. Low cost, 4. Not vulnerable to desiccation or freeze-thaw damage, 5. Can withstand large tensile strains, 6. Low weight and volume consumed by liner, and 7. Easy to repair.	1. Potential strength problems at interfaces with other materials, and 2. Geomembranes can be punctured during or after installation.
Geosynthetic clay liners (GCLs)	1. Rapid installation, 2. Very low hydraulic conductivity to water if properly installed, 3. Low cost, 4. Excellent resistance to freeze-thaw, 5. Can withstand large differential settlement, 6. Not dependent on availability of local soils, 7. Low weight and volume consumed by liner, and 8. Easy to repair.	1. Low shear strength of hydrated bentonite, 2. GCLs can be punctured during or after installation, 3. Dry bentonite (e.g., at time of installation) is not impermeable to gas, and 4. Potential strength problems at interfaces with other materials.

conductivity compacted soil liners than to geomembranes and GCLs. On the other hand, the interfacial shear strength between a geomembrane, GCLs, and the adjacent material may limit the steepness of side slopes on which geomenbrane and GCLs can be used. Also, thin barrier layers such as geomembrane and GCLs are more vulnerable to construction damage or post construction puncture, although the consequences of an occasional, unanticipated puncture are much less severe in a cover system than a bottom liner system. Regulators and design engineers should weigh the pluses and minuses of the alternative materials and then select suitable materials depending on the specific requirements of individual projects.

In the United States and the most European countries, paper sludge have been used since year 1995 as the barrier layer for some final cover systems constructed in locations which have such material as a waste product. However, there is little performance data on the engineering properties of the sludge materials used. Moo-Young and Zimmie (1995) performed laboratory tests to determine water content, organic content, specific gravity, hydraulic conductivity, compaction, consolidation, and strength on seven paper sludge materials. They found that the sludge materials had a high initial water content ranging from 150 to 270%, an initial hydraulic conductivity ranging from about 1 x 10^{-7} cm/s to 5 x 10^{-6} cm/s, and behaved similarly to highly organic soil. Moo-Young and Zimmie (1995) also performed laboratory tests on six samples of a sludge used as a barrier layer material. Three samples were obtained shortly after construction and the other three samples were taken at 9, 18, and 24 months after construction. The results of the laboratory tests on these undisturbed samples indicated that the water content and hydraulic conductivity of the sludge decreased somewhat over times as the sludge consolidated and biodegraded (i.e., it mineralized to become more like a soil).

One distinctive feature of this research is to investigate the feasibility of paper sludge, which is generated in a paper factory in Japan, as a cover material based on the laboratory tests as well as on successful examples of the paper sludge application as cover material in other countries. In Japan, few research has been done on the reuse of the paper sludge for the environmental geotechnical purposes.

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CHAPTER 3

Experimental Study on Feasibility of Waste Sludge as Potential Barrier in Cover Systems

3.1. General Remarks

The objective of waste containment facilities is to limit the discharge of toxic contaminants into groundwater. Liners and covers are important components in this endeavor. While liners provide the final defense against groundwater contamination, covers provide the long-term control of rainwater percolation and leachate generation. However, few discussions have been dedicated to cover materials and waste containment facilities in Japan.

The dense population found in limited urban areas, the shortage of natural resources, and the rapid growth of industry in Japan have resulted in a huge amount of waste generation and have created a considerable challenge for the effective disposal of such waste. Kamon and Katsumi (1994) reported that there are several paper mill industries in Japan that generate considerable amounts of paper sludge (PS), which reaches up to 1.15 million tons every year. Furthermore, the amount of construction sludge (CS) has reached a level of 10 million tons per year due to slurry excavation works involving shield tunnels, cast-in-place concrete piles, and diaphragm walls. The disposal of both PS and CS has brought about a considerable challenge due to the difficulty of securing dumping sites. Currently, PS and CS are considered to be types of industrial waste sludge, and are disposed of in landfill sites either directly or after incineration. In the near future, however, landfill sites are expected to be exhausted; thus, new measures must be taken for the effective reuse of waste including PS and CS.

In this Chapter, the possibility of using PS and CS as barrier material in cover system is explored by evaluating their geotechnical and chemical properties. Various geotechnical efforts including hydraulic conductivity, shear strength, consolidation, x-ray diffraction (XRD), and digital microscopy tests, as well as a chemical analysis of the effluents, are carried out to evaluate the suitability of applying both PS and CS as barrier material in cover system. The long-term behavior of sludge materials, with respect to settlement and hydraulic conductivity, is also examined by geotechnical centrifuge tests. In addition, the achievement in this Chapter has been reported by Kamon et al. (2000, 2001a, 2001b, 2001c, 2002) and Rajasekaran et al. (2000a, 2000b, 2000c).

3.2. Waste Sludge as Barrier Material

3.2.1. Feasibility of Wastes as Barrier Materials

The geotechnical aspects of landfill cover systems have been reviewed and reported by Benson and Khire (1995). U.S. landfill liner and cover regulations state that the hydraulic conductivity of final cover systems has been made equivalent to that of the bottom liner or the foundation soil beneath the landfill (i.e., hydraulic conductivity is lower than or equal to 1 x 10^{-5} cm/s for municipal solid waste landfills, and 1 x 10^{-7} cm/s for hazardous waste landfills). This means that the final cover systems should have a similar hydraulic conductivity to the bottom liner systems.

The important function of waste containment facilities depends on the hydraulic conductivity of the constituent materials. Thus, an evaluation of the hydraulic conductivity of the barrier layers of cover and bottom liner systems will be important to the design of waste containment facilities in landfill sites (Daniel, 1995). Up to the present, compacted clays with a low hydraulic conductivity have generally been used as barrier materials. Recently, tests have been carried out to construct barrier layers using suitable waste materials that maintain a low hydraulic conductivity, due to social requests to reduce disposal site construction costs, and to rationally reduce the capacity of waste at landfill sites.

When waste or soil materials are applied to the barrier layer of a landfill cover system, whether or not the material can sufficiently perform in term of requirements such as the hydraulic performance, compressibility, and slope stability, is a critical issue. Table 3.1 lists the main design aspects when soil or waste materials are applied to cover systems. In the design, the performance of each material, regarding Table 3.1, should be clarified beforehand. Afterward, the selection of suitable materials, in relation to the performance anticipated at each landfill site, is very important. In this Chapter, special focus is placed on: (1) basic properties of PS and CS, (2) potential environmental effect due to reuse of PS and CS, (3) hydraulic performance of PS and CS, (4) compression characteristics of PS and CS, and (5) slope stability for the PS and CS. The potential of PS and CS as barrier materials in cover system is thereby evaluated (Kamon et al. 2002).

3.2.2. Paper Sludge and Construction Sludge

In view of the presence of organic fibers and a high initial water content, problems arise in determining the liquid limit and the plastic limit of paper sludge using standard laboratory testing procedures. Moo-Young and Zimmie (1996a) succeeded in obtaining the Atterberg limits of paper sludge under wet conditions. In a subsequent paper (1996b), they reported that the water content of paper sludge varies from 150 to 268%, and that the organic content values are high (up to 60%). In an extensive study on seven types of paper sludge (1996a), and they reported that paper sludge contains a high water content, is highly compressible, and

Table 3.1 Functions and design aspects of landfill cover system (summarized from Daniel, 1995; Othman et al., 1995)

Principal functions	· Minimize water and air infiltration into the landfill; · Minimize gas migration out of the landfill; · Serve as a system for the control of odors, disease vectors, and other nuisances; and · Serve as a component of the landfill surface-water management system.
Design aspects	· Flow of water in and through the cover systems; · Impacts of waste settlement on the performance of the cover systems; · Static and dynamic cover system stability; and · Surface-water management.
Complicated factor	· Temperature extremes, possibly including freeze/thaw to significant depths; · Cyclic wetting and drying of the soils; · Plant roots, burrowing animals, and insects in the soil; · Differential settlement caused by uneven settlement of the underlying waste or foundation soil; · Down-slope slippage or creep; · Vehicular movements on road traversing the cover; · Wind or water erosion; and · Deformations caused by earthquakes.

behaves like an organic soil. Floess et al. (1995) carried out a two-year study on paper sludge, and the results clarified the fact that a low hydraulic conductivity of less than 1 x 10^{-7} cm/s is maintained over two years; this result promoted its usage in U.S. landfills. A series of centrifuge model tests and field trials on paper sludge showed that the paper sludge is non-hazardous, and its behavior is similar to clay (Zimmie and Moo-Young, 1995). The beneficial uses of paper sludge, including its application for daily and final landfill cover systems, have been reported by Zimmie and Quiroz (1999). Benson and Wang (2000) reported that conventional small-scale tests (71 mm in diameter) are satisfactory for assessing the hydraulic conductivity of paper sludge in the field. The undrained shear strength of paper sludge landfill cover varies from 12 to 35 kPa for a water content of 96 to 180%, respectively (Quiroz and Zimmie, 2000). Paper sludge discharged in Japan is usually incinerated prior to the landfilling to reduce its volume. However, if it is possible to use the un-incinerated sludge, this will help to manage the waste in the paper industry.

In Japan, a large amount of construction sludge is discharged (10 million tons per year), and more than half of it is merely disposed of. Several researchers have reported the potential use of construction sludge for geotechnical applications (e.g., Ogawa, 1995; Tsukada and Ogawa, 1996). Ogawa (1995) reported the successful application of an improved construction sludge for constructing an embankment (3700 m^3) along the Tone River in Japan.

3.3. Experimental Procedures to the Applicability of Waste Sludge as Barrier Material in Cover Systems

3.3.1. Materials Used

The paper sludge (PS) used in the present study was collected from piles prior to incineration treatment. The PS had already been dehydrated by the belt press, and the dehydrated PS had a water content of about 130%. When the PS was dried for water content adjustment, the water content was decreased in a 35°C thermostatic chamber to avoid the removal of the organic matter in the PS. After drying, a water content of around 50% was obtained.

The construction sludge (CS) used in the present study was discharged from shield tunneling work in Osaka City, Japan. Generally, its water content was decreased to about 40% by mixing it with a coagulant, and subsequently, by carrying out mechanical dehydration. The CS used in the present study had a high water content of 330%, because it was collected prior to dehydration. The CS was also then dried in a 35°C thermostatic chamber, and a water content of around the liquid limit was obtained.

3.3.2. Tests for Basic Properties

The physical and the engineering characteristics of the PS and CS samples were investigated following the JGS (Japanese Geotechnical Society) standards. The PS tends to form flocks (which develop a coarse structure) that cannot easily be pulverized under dry conditions. Furthermore, PS contains an abundant amount of fibrous organic matter. Thus, an analysis of the grain size distribution following the JGS 0131 could not be carried out on the PS. In addition, Kraus et al. (1997) reported the determination of the liquid and plastic limits for PS using standard laboratory testing procedures is difficult due to organic matter contained in PS. Therefore, a falling cone method following the JGS 0142 was used to estimate the liquid limit for the PS in this study. Compaction tests following the JGS 0711 were carried out to examine the relationship between the dry density and the molding water content.

Batch-type leaching tests following the method of the Japanese Leaching Test (JLT-13) of the Ministry of Environment were carried out in order to determine the leaching potential of heavy metals such as lead (Pb), cadmium (Cd), chromium (Cr), and zinc (Zn), other cations such as calcium (Ca^{2+}), sodium (Na^{+}), potassium (K^{+}), and magnesium (Mg^{2+}), and the total organic content (TOC) from PS and CS. The chemical composition in the effluent collected from JLT-13 was analyzed using an ion coupled plasma spectrometer (Shimadzu ICPS-8000).

3.3.3. Cone Index and Unconfined Compressive Tests

Cone index test and unconfined compressive test were carried out in accordance with JGS

0716 and JGS 0511, respectively. These tests are useful evaluating the fundamental levels of dehydration-compaction treatments of PS and CS which are discharged as industrial wastes, and are applied to the barrier material in cover system. The PS and CS samples used in the cone index and unconfined compressive tests were prepared with different levels of molding water content. They were respectively compacted in the molds with a diameter of 10 cm and a height of 12.7 cm, and that with a diameter of 5 cm and a height of 10 cm, to achieve the same level of dry density as that obtained by standard compaction curve at each molding water content. This compaction procedure is the same as that used to prepare the specimens for the shear strength test, consolidation test, and hydraulic conductivity test mentioned in the later section. The cone index (cone penetration strength) obtained by the test is defined as average resistance pressure applied on the bottom of the cone when the cone penetrometer continuously penetrated 5.0, 7.5, and 10.0 cm in the depth of specimen at the penetration rate of 1 cm/s.

3.3.4. Shear Strength and Consolidation Tests

Consolidated undrained triaxial tests with pore pressure measurements following JGS 0523 were performed to examine the shear strength behavior of the two types of sludge. PS and CS samples with molding water contents of 148.8% and 21.5%, respectively, were used. These levels of molding water content for the PS and CS used in this test corresponded approximately to the levels of molding water content which can obtain the minimum hydraulic conductivity. The PS and CS samples with each molding water content were compacted to achieve the same level of dry density as that obtained by a standard compaction curve at each molding water content. Confining pressures of 30, 60, and 120 kPa were applied to evaluate the internal friction angle and cohesion.

Consolidation tests were conducted according to JGS 0411. PS and CS samples with molding water contents of 144.3% and 27.0%, respectively, were used. The compaction method of the PS and CS was the same in consolidated undrained triaxial tests. The compaction conditions of the PS and CS are the same in both tests, provided approximately the minimum hydraulic conductivity. The relationship between compaction condition and hydraulic conductivity is discussed in a later section. In the consolidation tests, a consolidation ring with a diameter of 6 cm and a height of 7 cm was used.

3.3.5. Hydraulic Conductivity Tests and Chemical Analysis of the Effluents

Hydraulic conductivity is a key issue for barrier layers. The hydraulic conductivity of PS and CS was evaluated using flexible-wall permeameters, and the falling-head procedure (*see* Fig. 3.1). The infiltration water used in the hydraulic conductivity tests was distilled water (a pH of 6.2). In the hydraulic conductivity tests, five PS samples were prepared with different levels of molding water content (53.0%, 75.7%, 114.5%, 136.1%, and 157.9%). In the case of

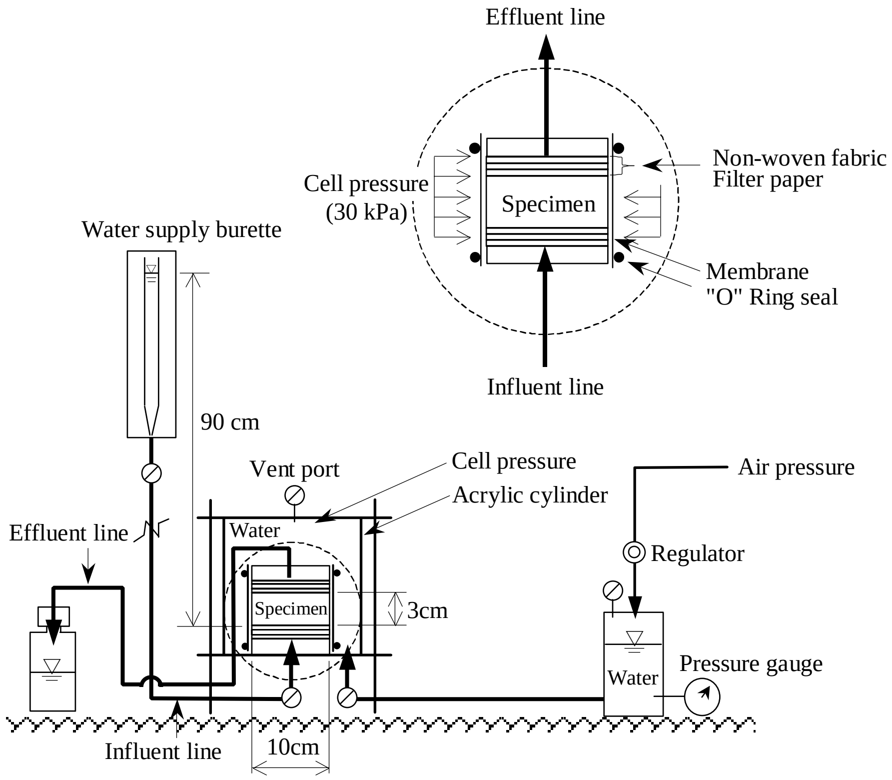

Fig. 3.1 Schematic diagram of flexible-wall hydraulic conductivity experimental setup

the CS samples, molding water contents of 10.5%, 20.1%, 26.6%, 32.7%, and 40.9% were used. The PS and CS samples with these molding water contents were compacted in a specially prepared mold (10 cm in diameter and 3 cm in height) to achieve the same level of dry density as that obtained by standard compaction curve at each molding water content. This compaction procedure is the same as in the shear strength and consolidation tests mentioned above. The compacted specimens were carefully placed in the hydraulic conductivity cell with a layer of non-woven fabric sandwiched between filter papers. A low confining stress of 30 kPa was applied to all compacted specimens in order to eliminate the sidewall leakage of the infiltration water. Moo-Young and Zimmie (1996a) reported that performing hydraulic conductivity tests at a low confining stress of 34.5 kPa for barrier materials provides the most practical approach to simulate the worst case, that is, the highest hydraulic conductivity. The confining pressure of 30 kPa used in this test is also referred to in their reports.

When waste sludge such as PS and CS are utilized for barrier materials in cover systems, the evaluations of their geotechnical compatibility as barrier materials are an indispensable task. However, as long as risks to the surrounding environment from reuse of waste sludge materials exist, their utilization is impossible. It is therefore necessary to carefully evaluate the environmental impact due to effective utilization of the waste sludge. In

Japan, before a waste is geotechnically utilized, the JLT-13 or JLT-46 methods have to be carried out as the official leaching tests for an environmental impact assessment on the leaching of toxic substances from the waste. In the present study, the JLT-13 method was conducted to understand the basic leaching properties of heavy metals such as lead (Pb), cadmium (Cd), chromium (Cr), and zinc (Zn) and the total organic content (TOC), The concentrations of the heavy metals, the other cations such as calcium (Ca^{2+}), sodium (Na^{+}), potassium (K^{+}), and magnesium (Mg^{2+}), and the total organic content (TOC) in the effluents collected from the hydraulic conductivity tests were measured using the ion coupled plasma spectrometer. The leaching behavior of a toxic substance from the compacted sludge obtained from the hydraulic conductivity tests rather than from the official leaching tests (JLT-13) can reproduce the in-situ conditions.

3.3.6. XRD and Microscopic Analysis

An x-ray diffraction (XRD) analysis was conducted on the dry samples of PS and CS using an x-ray diffractometer (Rigaku RAD-IIB) and a digital HD microscope (Keyence VH-7000) was used to observe the fabric view of the sludge samples, magnifications varied from 25 to 175.

3.3.7. Geotechnical Centrifuge Tests

The use of a geotechnical centrifuge allows the simulation of long prototype times, which in turn facilitates the prediction of the future behavior of PS and CS as barrier material in cover system. This will be an important consideration in obtaining regulatory agency approval for the use of sludge as landfill covers.

The hydraulic conductivity and settlement characteristics of the compacted PS and CS were measured under centrifugal field at 60 G to predict the long-term behavior of the PS and CS as a barrier material in cover systems. The geotechnical centrifuge, located at the Disaster Prevention Research Institute, Kyoto University, Japan, was used for the experiments. Geotechnical centrifuge tests were performed for 24 hours to simulate the long-term behavior of compacted PS and CS as a barrier material, which corresponds to 9.86 years according to the established scaling relationship (*see* Table 3.2). Whether or not a period of 9.86 years is a long term for a barrier material should be determined by taking the following into consideration. The long-term performance of barrier materials depends on the performance required at each landfill site.

Figure 3.2 shows a cross section of the model used in the centrifuge tests with details on the layout of the instruments. A stainless tank was suitable for the centrifuge test, since it can resist a 60 G centrifugal field. In addition, a stainless steel filter was installed at the bottom of the stainless tank, and this part was considered to be the drainage layer. In the centrifuge tests, the PS and CS with molding water contents of 145.2 and 25.3%, respectively,

Table 3.2 Scaling relationship for centrifugal loading tests (summarized from Zeng et al., 1998; Jessberger and Stone, 1991)

Parameter	Dimension	Prototype	Centrifuge model (at N G)
Linear	L	l	l / N
Area	L^2	A	A / N^2
Volume	L^3	V	V / N^3
Stress	$ML^{-1}T^{-2}$	ρ	ρ
Strain		ε	ε
Force	MLT^{-2}	F	F / N^2
Mass density	ML^{-3}	ρ_m	ρ_m
Unit weight	$ML^{-2}T^{-2}$	γ	$N\gamma$
Time*	T	t	t / N^2

* Applies to laminar flow processes such as consolidation

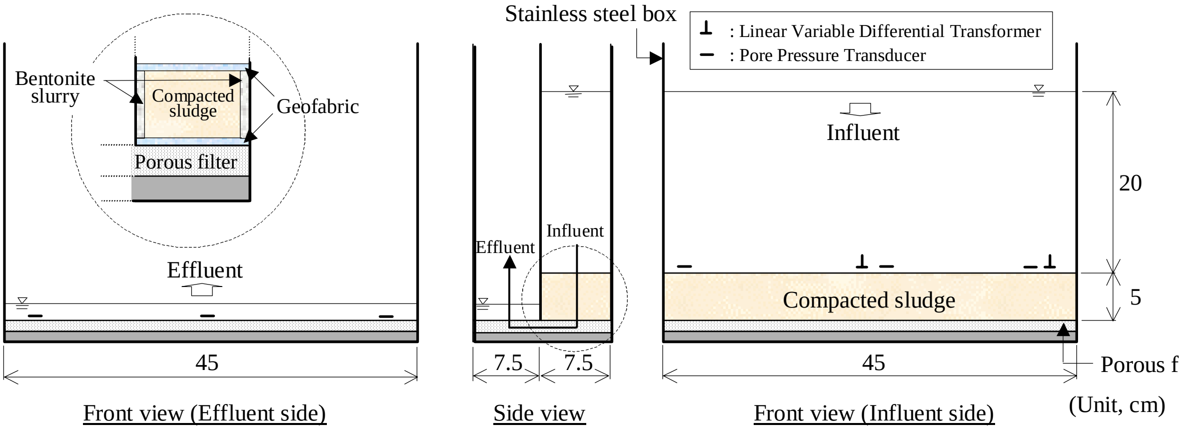

Fig. 3.2 Cross sections of tank and layout of instruments in centrifuge test

were compacted directly in the soil tank to achieve dry densities of 0.48 and 1.48 g/cm^3, which provide the same compaction conditions as the standard compaction curve at each molding water content. A water level of 20 cm on the surface of the compacted sludge was provided, and water was allowed to pass through the filter from the influent reservoir to effluent reservoir. In the centrifuge tests, distilled water with a pH of 6.2 was used, although using actual rainwater might have been more suitable. In order to calculate the hydraulic conductivity of the compacted sludge, changes in the influent and the effluent water levels were observed. Three pore pressure transducers (PPT) were positioned on the surface of the compacted sludge, and the decrease in influent water level was measured at different time intervals during the test period. Similarly, increases in effluent water level were monitored at various time intervals by positioning three PPTs on the bottom of the effluent reservoir. It was thereby possible to measure the change in the difference of water level between the influent and the effluent sides during the centrifuge tests. Additionally, two linear variable differential transformers (LVDT) were set at predetermined positions on the sludge surface to measure the settlement of the sludge during the centrifuge testing.

As mentioned above, in the centrifugal tests, the changes in the water levels of the influent and the effluent sides were measured at various time intervals, Δt. The hydraulic conductivity of compacted sludge was calculated using the changes in the influent and effluent water levels (*see* Fig. 3.3). As shown in Fig. 3.3, both of the influent and the effluent levels are changed in this test, therefore, Equation (3.1) is established for the evaluation of the hydraulic conductivity in this test. Hydraulic conductivity tests in which the influent and effluent water levels change at the same time are generally regulated by ASTM D 5084 as *Tests with an Increasing Tailwater Level (Method C)*.

$$k = \frac{L a_{in} a_{ef}}{At(a_{in} + a_{ef})} \ln\left(\frac{h_{it1}}{h_{it2}}\right) \qquad (3.1)$$

where a_{in} is the cross-sectional area containing the influent liquid, a_{ef} is the cross-sectional area containing the effluent liquid, L is the thickness of the specimen, A is the cross-sectional area of the specimen, t is the elapsed time between the determination of h_{it1} and h_{it2}, h_1 is the head loss across the specimen at time t_1, and h_2 is the head loss across the specimen at time t_2.

For evaluation of hydraulic conductivity obtained from the centrifuge tests using a scaling relationship (*see* Table 3.2), the hydraulic conductivity calculated in the centrifugal field at N G in N times larger than hydraulic conductivity obtained in the prototype (1 G field).

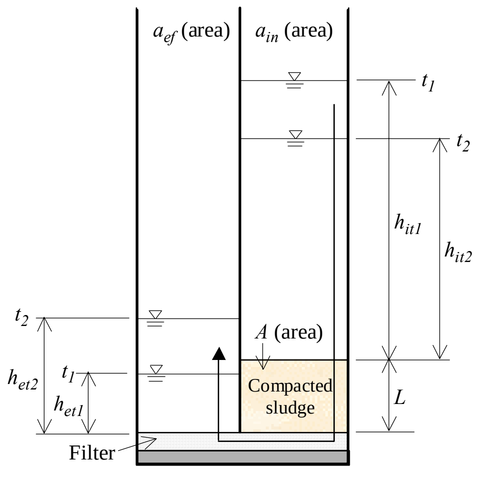

Fig. 3.3 Schematic diagram for calculation of hydraulic conductivity in centrifuge test

In the present study, the hydraulic conductivity obtained from the centrifugal field at *N* G was reduced by *N* in order to convert to the hydraulic conductivity in the prototype (1 G field).

As listed in Table 3.2, the stress affecting the centrifugal model and prototype are equal. Consequently, settlement *S* measured in the centrifugal field corresponds to that of *1/N* in the prototype. That is, the generated excess pore water pressure becomes equal to that of the prototype.

The barrier layer is usually constructed with a thickness of more than or equal to 60 cm in the field (prototype) in the United States, which corresponds to a layer thickness of more than or equal to 1 cm in the 60 G centrifugal field. For such a thin layer of sludge, uniform compaction is difficult and the risk of water leakage during centrifugal loading is increased. The sludge layer thickness of 5 cm was applied in the centrifugal field at 60 G, which corresponds to a prototype 300 cm in thickness. The values of hydraulic conductivity and settlement for the compacted sludge obtained in the centrifuge tests will not depend on the difference of the corresponding layer thickness in the prototype (1 G field). This is because the hydraulic conductivity and settlement obtained by the centrifuge test is converted to the field (prototype), using the scaling relationship considering the layer thickness and stress distribution with elapsed time (*see* Table 3.2). A thin layer of bentonite slurry was applied between the compacted sludge layer and the sidewall of the tank in order to prevent sidewall leakage during the tests. This technique was applied also in the centrifuge test conducted by Zimmie et al. (1994).

3.4. Results and Discussions

3.4.1. Basic Properties

The basic properties of PS and CS are given in Table 3.3. Previous research results for the basic engineering properties of PS generated in the United States are also summarized in Table 3.3. Figure 3.4 shows the compaction curves for PS and CS. The liquid limit of PS is as high as 132.5% (*see* Table 3.3), and the compaction test results indicate an optimum water content of 74% (*see* Fig. 3.4(a)). Because PS contains an organic content as high as 63.7%, the optimum water content of PS is higher, and the maximum dry density of it is lower than those of typical compacted clays. When the basic properties of the PS obtained in the present study are compared with previous research results for PS, the PS used in the present study has almost the same properties as the PS which was reported previously, in terms of the particle density, the consistency limits, the optimum water content, the maximum dry density, and the organic content (*see* Table 3.3).

The CS has a wide range of grain size, and is classified as a silty soil, according to the Japanese unified soil classification system (*see* Table 3.3). However, the natural water content of the original CS is very high with 337.2%, while the measured values of the liquid limit and optimum water content provide 46.5% and 18.2%, respectively (*see* Table 3.3 and

Table 3.3 Basic properties of PS and CS used in this research

Properties	CS	PS	Previous results of PS*
Natural water content (%)	337.2	132.5	150 - 268
Particle density (g/cm^3)	2.68	1.79	1.80 - 2.08
Liquid limit (%)	46.5	330.0	218 - 294
Plastic limit (%)	28.0	111.5	94 - 147
Optimum water content (%)	18.2	74.0	40 -100
Maximum dry density (g/cm^3)	1.637	0.553	0.52 - 0.82
Grain size distribution			
Gravel fraction (%)	0	-	
Sand fraction (%)	32.5	-	
Silt fraction (%)	53.3	-	
Cray fraction (%)	14.2	-	
pH in sludge	7.34	7.57	
Ignition loss (%)	6.2	63.7	35 - 60
Leachate composition**			
Pb^{2+} (mg/L)	≤ 0.01	≤ 0.01	≤ 0.05
Cd^{2+} (mg/L)	≤ 0.01	≤ 0.01	≤ 0.01
Tr-Cr (mg/L)	≤ 0.01	≤ 0.01	≤ 0.05
Zn^{2+} (mg/L)	≤ 0.01	≤ 0.01	≤ 0.05
Ca^{2+} (mg/L)	118.2	256.7	
Na^{+} (mg/L)	152.1	20.4	
K^{+} (mg/L)	192.6	2.32	
Mg^{2+} (mg/L)	116.6	10.7	
TOC (mg/L)	60.8	171.6	61.6 - 293
EC (mS/cm)	1.872	0.912	
pH	7.21	7.37	
Exchangeable cations	27.03	14.63	
Ca^{2+} (meq/100g)	5.89	12.80	
Na^{+} (meq/100g)	6.61	0.89	
K^{+} (meq/100g)	4.93	0.06	
Mg^{2+} (meq/100g)	9.60	0.88	

* Summarized from Moo-Young and Zimmie (1996a) and Izu et al. (1997)
** Following the Japanese Leaching Test (JLT-13)

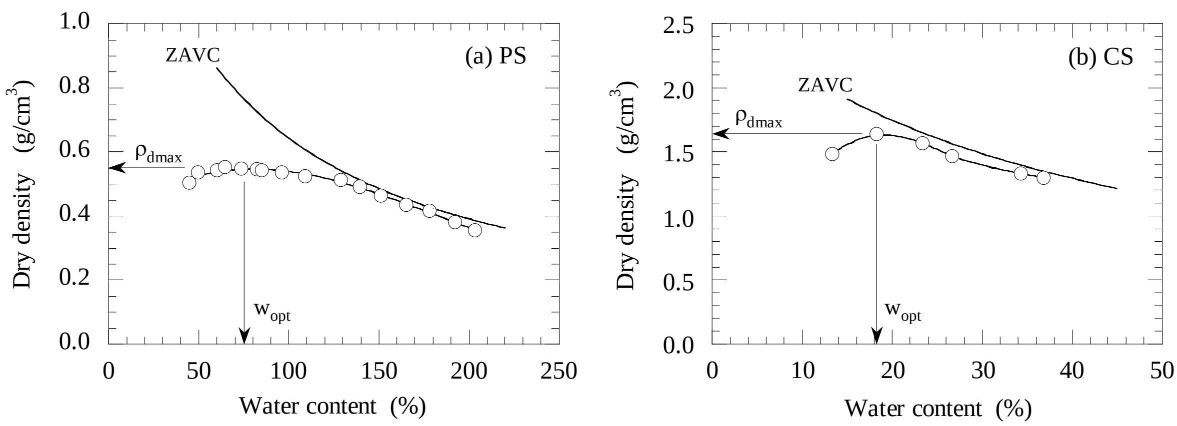

Fig. 3.4 Compaction curves for (a) PS and (b) CS

Fig. 3.4(b)). Thus, a considerable dehydration is necessary in reusing the CS as barrier material in cover system.

The chemical composition in the effluent collected from the batch-type leaching test (JLT-13) is also listed in Table 3.3. The concentrations of heavy metals such as Cr, Cd, and Pb leached out from the CS and the PS are less than the environmental quality standard value stipulated in the Japanese Governmental requirements. The leaching of cations such as Ca^{2+}, Na^{+}, Mg^{2+}, and K^{+} is more remarkable with the CS than with the PS. In the PS case, the leaching of Ca^{2+} is relatively higher than that of other cations.

3.4.2. Cone Penetration and Unconfined Compressive Strength

Just discharged PS and CS from paper factory and construction site, respectively, are classified into industrial waste as types of sludge. The PS and CS with remaining the discharge condition practically contain a large quantity of water, and are considered to be no possible for direct reuse of them. An intermediate treatment is usually conducted for reducing the water content of the PS and CS, under responsibility of the enterprises which discharged the PS and CS. Generally, the dehydration using the belt press is carried out for the intermediate treatment of PS, while the treatment of CS is performed by mechanical dehydration with mixing a coagulant. The PS used for this research is that after the dehydration using the belt press, while the CS is that before performing mechanical dehydration treatment with mixing a coagulant. The natural dehydration treatment by drying in a 35°C thermostatic chamber, therefore, is applied for reducing the water content of CS, in this research. If the PS and CS are used for barrier material in cover systems, the effect of the dehydration treatment on the bearing capacity of PS and CS during the construction works should be clarified. For the purpose, the cone index and unconfined compressive tests are carried out for the PS and CS compacted at different molding water contents. The cone index test or unconfined compressive test is commonly used to provide an index for classification of surplus soil from construction works. Verification between the cone penetration strength obtained by the cone index test and Table 2.2, which is shown the basis of selection for soil material and applicability of the soil in various uses, provides the estimation of execution management as a soil material in the construction works, such as a trafficability, and the possibility of reuse for the surplus soils can be predicted.

Figure 3.5(a) shows the cone penetration and unconfined compressive strengths with respect to dry density of PS compacted at different molding water contents. The cone penetration and unconfined compressive strengths of PS range from 200 to 450 kPa and from 50 to 100 kPa, respectively, in the different molding water contents from 70 to 180%, and large variation in the both strengths with changing in the molding water content is not found. This tendency may be because there is no large difference in the dry density of PS compacted at each molding water content. In addition, the PS exhibits higher cone penetration strength than that of other naturally occurred soil material in the same high water content regions. The organic matter, especially the organic fibers, contained in the PS as reinforcement element is contributed to this high strength. Comparing Fig. 3.5(a) with Table 2.2 shows that the PS, which is originally classified as waste, satisfies the cone penetration strength required for the 4th class soil due to the dehydration-compaction treatments. The PS, after being improved to the 4th class soil, is applicable as reclamation material. It is also considered to be applicable as barrier material in cover system, from a viewpoint of the execution management using the cone index as shown in Table 2.2. Considering the fact that the strength of PS is without large variation in the cone index and that the unconfined compressive strength at the different molding water contents is higher, “soil spray method”, which constructs a flat layer by spraying sludge on a waste layer using compressed air, is considered to be applicable. The soil spray method is also effective in construction of the layer even on an inclined plane in landfill.

The cone penetration strength, unconfined compressive strength, and dry density of CS compacted at different molding water contents are shown in Fig. 3.5(b). The CS compacted at around its optimum water content exhibits a cone penetration strength as high as 780 kPa, implying that the same treatment as the 3rd class soil is possible (*see* Table 2.2). Also, considering the hydraulic performance of the CS at its optimum water content, CS is applicable for the barrier material in cover system. However, the cone penetration and unconfined compressive strengths of CS compacted at various molding water contents shows significant decrease with increase in the molding water content. The CS compacted at a molding water content around its liquid limit cannot demonstrate sufficient cone index to give the trafficability for construction works. However, the CS demonstrates sufficient low hydraulic conductivity to be used as barrier material in cover system, even when it compacted at its liquid limit. The hydraulic performance of CS and PS is discussed in a later section. If the reuse of CS with as a water content as high as its liquid limit is achieved, the barrier layer by the CS in cover system would be attractive even from a viewpoint of costs for reusing. For

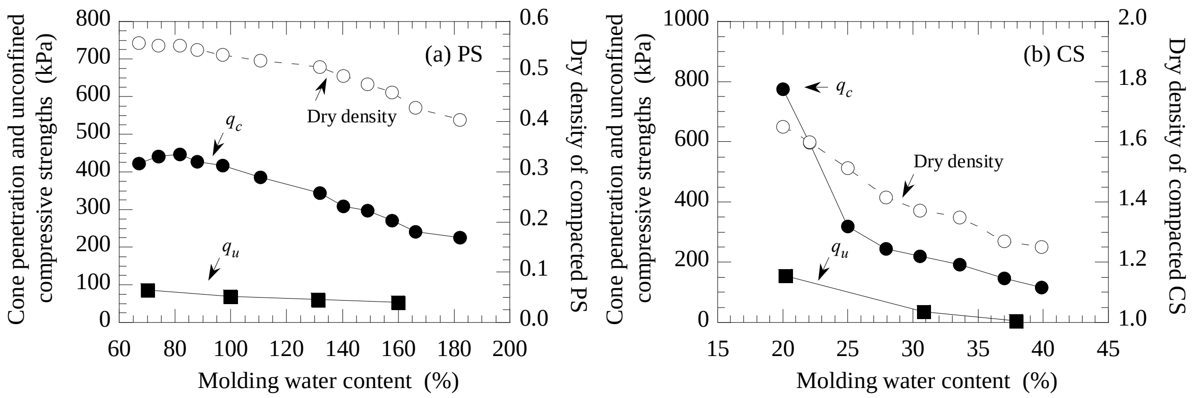

Fig. 3.5 Cone penetration strength, unconfined compressive strengths, and dry density of (a) PS and (b) CS compacted at different levels of molding water content

this purpose, the soil spray method mentioned above is considered to be applicable. Commonly, soil stabilization by cement is also considered to be effective to improve the strength properties of CS dehydrated to around its liquid limit by the dehydration treatment. For example, Kamon et al. (1998) investigated the treatment technique of waste sludge using dehydration-solidification treatments system. For the purposes of this study to evaluate the potential feasibility of PS and CS as barrier layer in cover system, the stabilization of CS by the dehydration-solidification treatment is not mentioned. However, applicability evaluations of CS stabilized by the dehydration-solidification treatments as barrier layer in cover system should be placed as practical subject.

When the waste sludge is applied as barrier material in cover system, dehydration-compaction treatment is needed to obtain the minimum hydraulic conductivity. The present research discusses the feasibility of PS and CS as barrier material in cover system, which have already dehydrated by the natural dehydration treatment. The term of "compacted sludge" used in this study means the PS and CS whose water contents have already decreased by some dehydration treatments.

3.4.3. Shear Strength and Consolidation

If sludge or soil materials are used for barrier materials in cover systems, their slope stability and compression characteristics must be evaluated, the slope stability in particular can become a major issue in overall landfill cover integrity (Quiroz and Zimmie, 1998). For this reason, in the present study, consolidated undrained shear and consolidation tests are conducted to evaluate the strength properties for compacted PS and CS as barrier materials in cover systems.

The values of cohesion and internal friction angle obtained from the consolidated undrained shear test for the compacted PS (with a molding water content of 148.8% and a dry density of 0.48 g/cm^3) and the compacted CS (with a molding water content of 21.6% and a dry density of 1.53 g/cm^3) are shown in Fig. 3.6. As noted, the determination of a failure point is difficult, because a yield point can not be found in the stress-strain curves obtained from the consolidated undrained shear test for the compacted PS. Thus, a reasonable level of 15% strain has been arbitrarily selected as the failure point for compacted PS. Cohesion and the internal friction angle for the compacted PS are 3.9 kPa and 40.5°, respectively (*see* Fig. 3.6(a)), while the results for compacted CS indicate cohesion and an internal friction angle of 3.6 kPa and 34.1°, respectively (*see* Fig. 3.6(b)). Although the compacted PS has a higher molding water content and a lower dry density than those of the compacted CS, the test results indicate that the compacted PS has higher levels of cohesion and internal friction angle than those of the compacted CS. The higher value for the internal friction angle may be mainly attributed to the presence of organic fibers in the compacted PS. MacFarlane (1966) reported that the internal friction angle of compacted PS shows a high value of 45 to 65° according to the increase in organic content.

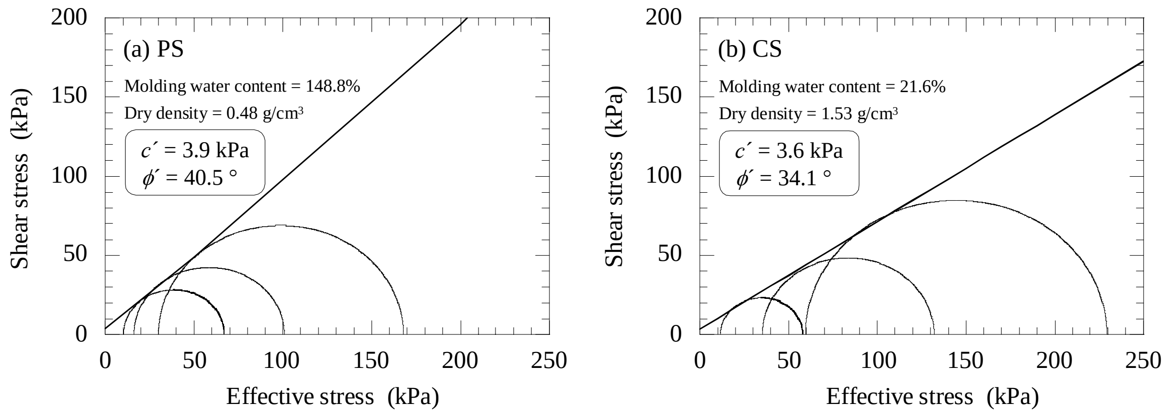

Fig. 3.6 Shear strength of (a) PS and (b) CS

When the obtained results for the compacted PS used in this test are compared with results of compacted PS obtained by Wang et al. (1991), and Moo-Young and Zimmie (1996b), it is found that the shearing behaviors of both compacted PS are similar to the behavior of organic soil or slightly over-consolidated clay. Quiroz and Zimmie (1998) carried out an extensive study on the variations in undrained strength for compacted PS covers using field vane shear tests. Their results indicated that the compacted PS with water content of 150% exhibits an undrained shear strength of 23 kPa for stable slopes not steeper than 3H: 1V. The compacted PS used in the present study showed an undrained shear strength of 32 kPa under an effective stress of 30 kPa estimated to be the smallest stress affecting the barrier layer. Taking the results of Quiroz and Zimmie (1998) and this test into consideration, the compacted PS used in the present study will be able to practically form the stable landfill slopes. If it considers more in detail, Fig. 3.7 shows the relationship between the slope angle and the safety factor of slopes for PS and CS barriers, in which the barrier layers are assumed to have a thickness of 50 cm, using the parameters obtained from the consolidated undrained shear tests. The stability analysis of slopes on an infinite plane-sliding surface is applied for calculating the safety factor of slopes for the PS and CS barriers. In the analysis, the seepage flow in the sludge barrier is assumed. The safety factor of slope, F_S, is estimated using following Eq. (3.2):

$$F_s = \frac{2c'}{\gamma_{sat} \cdot z \cdot \sin 2\beta} + \frac{\gamma'}{\gamma_{sat}} \frac{\tan \phi'}{\tan \beta} \tag{3.2}$$

where c' is the cohesion (kPa), ϕ' is the internal friction angle (°), γ' is the submerged unit weight of sludge barrier (kN/m^3), γ_{sat} is the saturated unit weight of sludge barrier (kN/m^3), z is the thickness of sludge barrier (m), and β is the angle of slope (°). Each parameter used for the slope stability analysis of the PS and CS barriers is shown in the Fig. 3.7. The slope

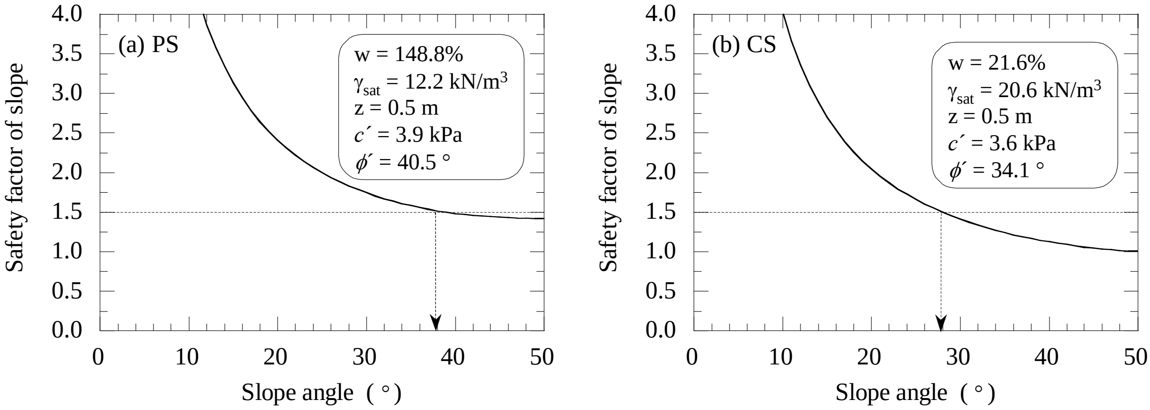

Fig. 3.7 Safety factor of slope composed of compacted (a) PS or (b) CS as barrier layer material

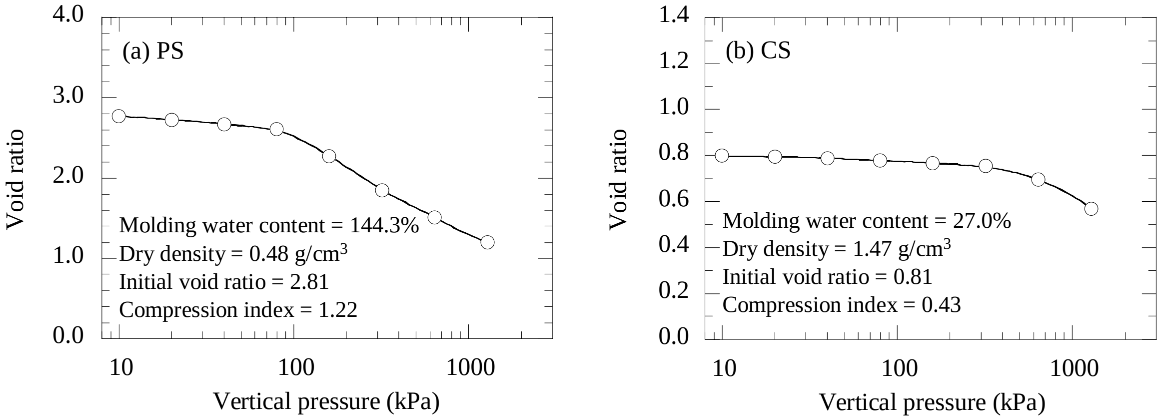

Fig. 3.8 Consolidation properties of (a) PS and (b) CS

stability of the PS barrier achieves a safety factor of greater than 1.5 at a slope angle less than or equal to 39° when the PS barrier has a layer thickness of 50 cm. The slope angle satisfying a safety factor of 1.5 provides a slope angle at less than or equal to 28° in the case of CS barrier. Slopes for cover systems which apply clay barriers with a low level of hydraulic conductivity are generally recommended to be 25 to 30% (14 to 16.7°) in the United States (Quiroz and Zimmei, 1998).

The results of one-dimensional consolidation tests conducted on compacted PS and CS are shown in Fig. 3.8. The molding water content and dry density levels of each compacted specimen are also indicated in Fig. 3.8. Compacted PS shows a large reduction in void ratio from 2.81 to 1.20 with an increase in consolidation stress from 9.8 to 1254 kPa, and this indicates a compression index, C_c, of 1.22 (*see* Fig. 3.8(a)). When the results for compacted PS obtained from this test are compared with the previous test results for compacted PS by Wang et al. (1991), and Moo-Young and Zimmie (1996a), each compacted PS shows highly compressive behavior. Moo-Young and Zimmie (1996a) performed consolidation tests on several kinds of PS generated in the United States. In their results, the average compression index, C_c, of those compacted PS type was 1.24. This average value agrees well with the compression index, C_c, of the compacted PS used in the present study. Although, the compacted PS is highly compressive, as shown in a compression index, C_c, of 1.22, it exhibits over-consolidation state under an overburden pressure of 20 to 30 kPa, which is the typical load subjected to the barrier layer. In the case of the compacted CS used in this test, the initial void ratio decreases from 0.81 to 0.60. The compression index C_c of the compacted CS is estimated at 0.43 (*see* Fig. 3.8(b)). The test results indicate that the compacted CS does not show high compressibility when compared with compacted PS.

3.4.4. Saturated Hydraulic Conductivity

The main criteria for using PS and CS for landfill cover applications can be evaluated using hydraulic conductivity. Table 3.4 shows the molding water content, dry density, and the other characteristic values for compacted PS and CS specimens used in the hydraulic conductivity tests.

The relationship between hydraulic conductivity and molding water content for compacted PS and CS, under a confining pressure of 30 kPa, are shown in Fig. 3.9. The plots in Fig. 3.9 correspond to the stable hydraulic conductivities which are obtained for the elapsed time. Compacted CS with molding water contents of 10.5%, 20.1%, 26.6%, 32.7%, and 40.9%, respectively, shows hydraulic conductivity ranging from 1.2 x 10^{-8} to 6.5 x 10^{-8} cm/s. Especially, compacted CS with a molding water contents of 20.1% and 26.6% exhibit the lowest hydraulic conductivity of approximately 1.2 x 10^{-8} cm/s. The lowest hydraulic conductivity is obtained with CS compacted with a water content 2 to 8% wetter than the

Table 3.4 Initial conditions of compacted PS and CS used for hydraulic conductivity tests

	Molding water content (%)	Wet density (g/cm^3)	Dry density (g/cm^3)	Void ratio	Volume of void (cm^3)	Degree of saturation (%)
PS	53.0	0.80	0.53	2.41	166.6	39.4
	75.7	0.97	0.55	2.26	163.3	60.1
	114.5	1.12	0.52	2.42	166.7	84.8
	136.1	1.14	0.48	2.72	172.2	89.7
	157.9	1.16	0.45	2.98	176.5	94.9
CS	10.5	1.50	1.36	0.97	116.2	28.8
	20.1	1.95	1.63	0.65	92.9	83.0
	26.6	1.85	1.46	0.84	107.4	85.2
	32.7	1.78	1.34	0.99	117.6	88.1
	40.9	1.83	1.30	1.06	121.5	103.2

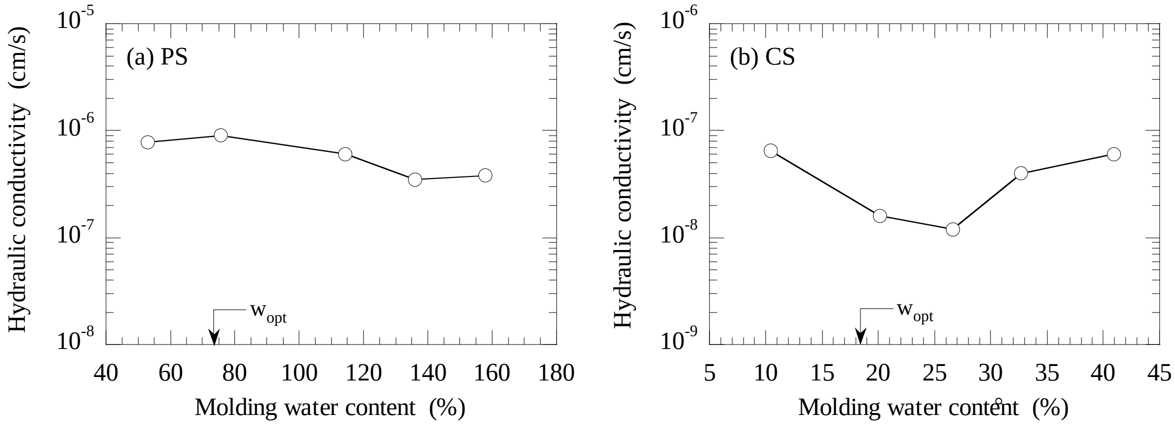

Fig. 3.9 Hydraulic conductivity versus molding water content for (a) PS and (b) CS

optimum water content of 18.2%. The compacted CS with several molding water contents used in the test satisfies the requirement for hydraulic conductivity of landfill covers issued by the United States Environmental Protection Agency (US EPA).

Compacted PS with molding water contents of 136.7% and 157.9%, which correspond to 62 to 83% the wetness of optimum water content of 74%, shows the lowest hydraulic conductivity. However, the remarkable difference in the hydraulic conductivity of the range of 3.5 x 10^{-7} to 9.0 x 10^{-7} cm/s for molding water contents with wide range of 50 to 200% cannot be observed. In other words, the hydraulic conductivity of compacted PS is not sensitive to the molding water content, which may be advantageous from the viewpoint of quality control during construction. In addition, Kraus et al. (1997) showed that compacted PS exhibits low hydraulic conductivity in a wide range of molding water contents, and this behavior is considered the same as the compacted PS used in this test. When the hydraulic conductivities of compacted PS obtained from the tests are compared with the construction criteria for cover systems issued by the US EPA, although a hydraulic conductivity of 1 x 10^{-5} cm/s can be satisfied, that of 1 x 10^{-7} cm/s cannot be ensured. Thus, when PS is applied as a barrier material, a performance evaluation which sufficiently considers the weather conditions at each landfill site is necessary. The performance evaluation of cover systems using a compacted PS barrier is conducted by a water balance analysis in Chapter 4.

Evaluations of the hydraulic conductivity of PS as a barrier material have been performed by several researchers. For example, the field test results presented by Kraus et al. (1997) and Quiroz and Zimmie (1998) showed a decrease in void ratio of PS due to consolidation under an effective stress of 70 kPa. This consolidation might result in a decrease in the hydraulic conductivity up to an acceptable level of 10^{-7} or 10^{-8} cm/s order of magnitude; furthermore, this level of hydraulic conductivity is maintained even after 8 years. They concluded that the consolidation effect significantly decreases the hydraulic conductivity of compacted PS, and that the organic decomposition of PS does not lead to an increase in the hydraulic conductivity of compacted PS over the long-term. In the present study, although the relationship between the concentration of total organic carbon (TOC) in the effluents collected from the hydraulic conductivity tests and the hydraulic conductivity is discussed in a later section, the effect of organic decomposition over the long-term on the hydraulic conductivity could not be examined, since the period of hydraulic conductivity test is too short to evaluate the effect of organic decomposition. However, if the research results of Quiroz and Zimmie (1998) for organic decomposition versus the hydraulic conductivity of compacted PS, which are mentioned above, are applied to this study, it is supposed that the organic decomposition of compacted PS does not have any adverse effect on the hydraulic conductivity or the long-term performance of PS as barrier material in cover systems.

3.4.5. Chemical Analysis of the Effluents from Hydraulic conductivity Tests

Column tests are deemed necessary and useful in order to assess the environmental impacts of the effective utilization of wastes (Kamon and Katsumi, 1999). In the present study, the concentration of cations including the heavy metals, pH, electric conductivity (EC), and total organic content (TOC) in effluents collected from the hydraulic conductivity tests are measured.

Figure 3.10(a) indicates the variations in the pH and EC of the effluents collected from the hydraulic conductivity test for compacted PS with a molding water content of 136.1%. As noted, the liquid-solid ratio, L/S, shown in the X-axis is defined as the ratio of infiltration volume to dry weight of a compacted sludge specimen. Although the initial pH of the effluents shows a range of 7.3 to 8.3, it tends to become stable around 7.4 with an increasing liquid-solid ratio, L/S. In addition, the EC, as the response of all the ions in a solution, indicates a high value during the initial stage (less than L/S of 5), and then decreases to almost equivalent to that of distilled water when the EC of the effluents achieves a stable

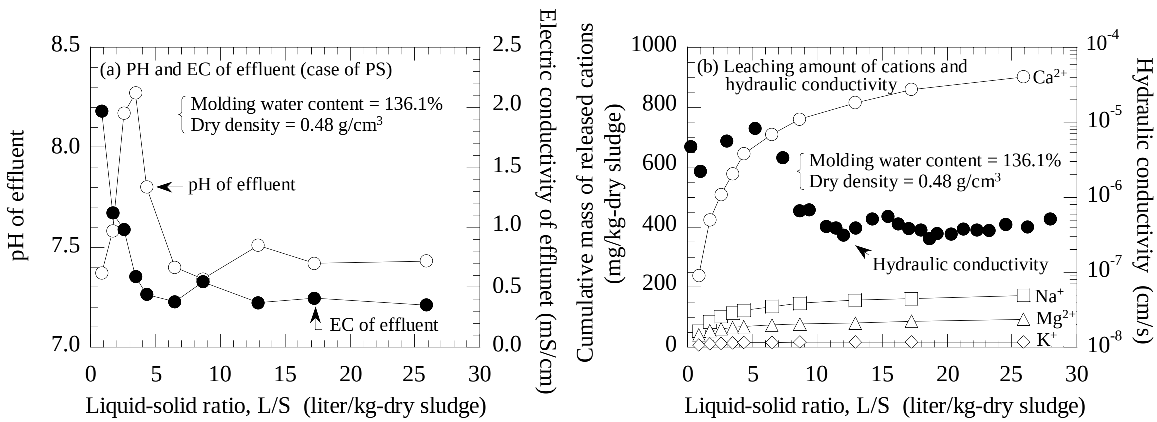

Fig. 3.10 pH, EC, and concentration of cations in effluents from hydraulic conductivity tests

value. Consequently, most of the ionized chemical substances in the PS seem to be dissolved in a range of less than 5 in the liquid-solid ratio. Figure 3.10(b) shows the cumulative leaching amount of Ca^{2+}, Na^{+}, K^{+}, and Mg^{2+} from the PS, and the hydraulic conductivity with liquid-solid ratio. Although the L/S for which the leaching of each cation achieves a stable tendency is different, each cation rapidly dissolves in approximately less than L/S of 5, and the leaching amount of Ca^{2+} is especially remarkable. This is because a large amount of calcium carbonate ($CaCO_3$) is contained in the PS, which reacts with the dissolved carbon dioxide (CO_2) in the influent, leading to the leaching out of Ca^{2+}. Representative reactions are as follows (Stumm and Morgan, 1996):

$$\mathrm{CaCO_3 + CO_2 + H_2O \Leftrightarrow Ca^{2+} + 2HCO_3^-}$$

Comparing Fig. 3.10(a) with Fig. 3.10(b), leaching tendency of these cations in the PS seems to affect the EC of effluents. Within the liquid-solid ratio, where the chemical substance abundantly dissolves from the PS, the soil fabric in the PS may change and the hydraulic conductivity seems to fluctuate. The leaching of cations from the PS has already been completed in the L/S where stable hydraulic conductivity is obtained.

Figure 3.11 shows the cumulative mass of released heavy metals in the effluents collected from the hydraulic conductivity tests. Even though the gradual release of a concentration of heavy metals in PS shows a sharp increase in pH at the initial stage, a sudden decrease in pH at L/S of 3 is noticed (see Fig. 3.11(a)). After the pH indicates a stable value of 7.4, the release of heavy metals shows a stable trend, and the concentrations are less than the environmental quality standard values, which are regulated by the concentration of each heavy metal in leachate at L/S=10, which is based on JLT-13, known as the official leaching test. In the case of CS, the measured concentrations of heavy metals are low when compared with the PS, and the pH of the effluents is close to a neutral condition (*see* Fig. 3.11(b)). The

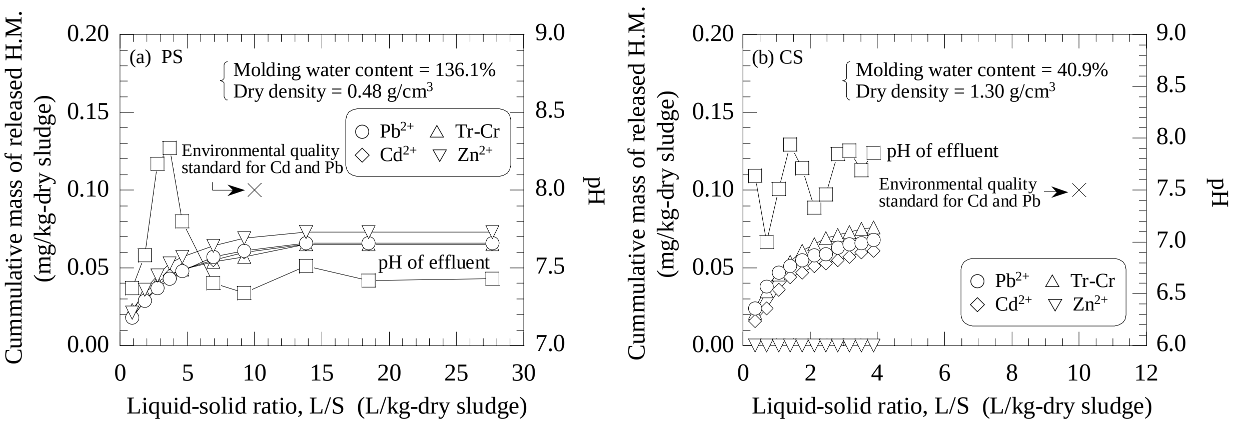

Fig. 3.11 Heavy metals released from (a) PS and (b) CS during hydraulic conductivity tests

above test results reveal that the concentrations of chemical constituents leached out from the PS and CS samples are within the environmental quality standards, and the application of both PS and CS for barrier materials in the field does not cause a serious environmental risk. Furthermore, the utilization of waste materials such as the PS and CS used in the present study for the barrier materials in cover systems is encouraged by Daniel (1995) discussed in the following. Daniel (1995) pointed out that heavy metal leaching is not as critical in the case of landfill cover as it is with landfill liners. His opinion is that since cover systems are placed over waste layers, if the barrier materials in cover system only satisfy the safety criterion for waste disposal, those materials will be applicable because the bottom liners may attenuate the leached chemicals.

In the case of PS, additional tests for total organic carbon (TOC) are carried out to evaluate its impact on hydraulic conductivity. It has been reported that the organic decomposition of PS results in an increase with time in the content ratio of the clay content (kaolinite) that originally exists in the PS, and a decrease in hydraulic conductivity (Moo-Young and Zimmie, 1996b). Figure 3.12 displays the variations in hydraulic conductivity and cumulative mass of released TOC with liquid-solid ratio. The cumulative mass of released TOC from the PS tends to increase with an increase in the liquid-solid ratio, L/S. This tendency may be due to the beginning of organic decomposition of PS under aerobic condition. The hydraulic conductivity of the PS maintains the almost stable value of 10^{-7} cm/s order of magnitude after the L/S reaches approximately 7, even if the cumulative mass of released TOC from the PS increased. The dispersion and fluctuation of hydraulic conductivity at the initial stage for L/S would be caused by the leaching out of chemical substances such as cations from the compacted PS discussed in Fig. 3.10, as well as the effect of consolidation. Thus, the release of TOC from PS does not contribute to the increase in its hydraulic conductivity.

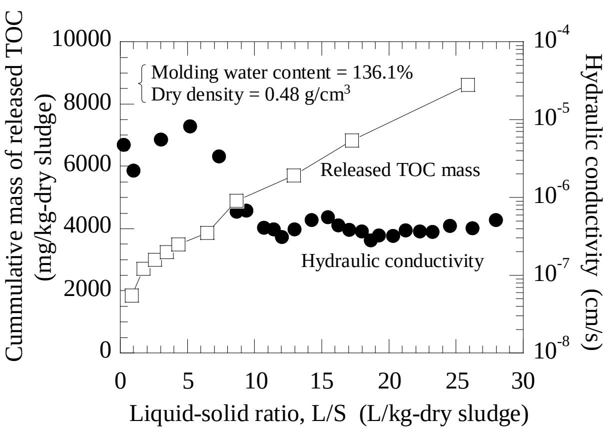

Fig. 3.12 Relationship between hydraulic conductivity and TOC in effluents from hydraulic conductivity tests of PS

3.4.6. XRD and Microscopic Analysis

The characteristics of sludge types and their hydraulic conductivity performance mainly depend on the mineral composition. Thus, the mineral composition of PS and CS should be evaluated. Air-dried PS and CS samples are used for the x-ray analysis, while wet and dry PS samples are further analyzed by microscopic study. Figure 3.13(a) shows the XRD analysis of the PS, which indicated the presence of kaolinite and calcium carbonate as dominant minerals. Moo-Young and Zimmie (1996a) also reported the same minerals are contained in the PS generated in the United States. The XRD pattern of the CS indicates the dominance of montmorillonite, illite, and quartz minerals (*see* Fig. 3.13(b)).

The PS contains a great deal of organic matter, as shown in Table 3.3. Thus, the views of the organic matter contained in the PS under wet and dry conditions are observed. Figure 3.14(a) shows a microscopic view of PS under wet conditions after the hydraulic conductivity tests, and the presence of homogeneous lumps consisting of organic fibers and tissues can be seen in this micrograph. The organic fibers and tissues embedded in the compacted PS would contribute to improve the engineering behavior of PS as the reinforcing elements. Franklin et al. (1973) drew a similar conclusion for organic soils, and they reported that the organic fibers act as reinforcing elements and result in a higher shear strength. A microscopic view of PS under dry conditions, which air-dried the PS after hydraulic conductivity tests, indicates shrinkage of the fiber lumps (*see* Fig. 3.14(b)). Moo-Young and Zimmie (1996b) also pointed out the shrinkage behavior of organic fibers contained in the PS generated in the United States. The microscopic view of the PS samples confirms the presence of well-knit organic fibers and organic tissues. It has been reported that the behavior of organic matter contained in PS is similar to that of peaty soils (Edil and Dhowian, 1981; Al-Khafaji, 1979). However, the author of this dissertation recognize the necessity of further understanding of the role of organic fibers and the effect of their decomposition on long-term

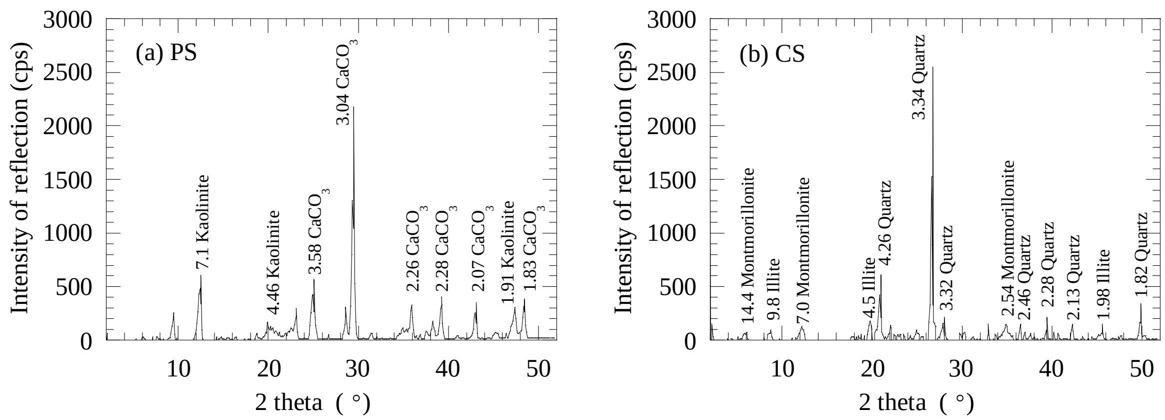

Fig. 3.13 X-ray diffraction patterns of (a) PS and (b) CS

(a) Wet condition

(b) Dry condition

Fig. 3.14 Scanning microscopic views of PS under (a) wet and (b) dry conditions

hydraulic conductivity of PS, when the long-term performance of PS as a barrier material in cover systems is evaluated.

3.4.7. Consolidation in Centrifugal Field

The initial conditions of compacted PS and CS used for the centrifuge test are shown in Table 3.5. Figure 3.15 shows the settlement measured at the center and side positions of the sludge surface in a centrifugal field of 60 G with centrifugal force loading time. No large difference in the settlement tendency of the PS and CS due the measurement positions is found. The comparison of settlement characteristics obtained from centrifuge tests and laboratory consolidation tests are as follows. As noted, the compaction conditions of these tests for each sludge are almost the same from the viewpoint of a molding water content and a dry density. For example, the molding water contents under compaction of PS used for the centrifuge test and laboratory consolidation test are 145.2% and 144.3%, respectively, and the dry densities of compacted PS are respectively the same as 0.48 g/cm^3. In the case of compacted PS, the void ratio at the end of the laboratory consolidation tests is about 2.27 under surcharge of 120 to 150 kPa. In addition, a settlement of about 1 cm is measured under a surcharge of 160 kPa for compacted PS 7 cm in height, which is equivalent to a compression of 14.3%. In the centrifuge tests, a surcharge of about 120 kPa is loaded on the compacted sludge, because a 20 cm water level is applied to the surface of compacted sludge and a centrifugal force of 60 G is loaded. Compacted PS with a layer thickness of 5 cm in the centrifugal field at 60 G indicated a settlement of about 0.7 cm by the centrifugal tests, which is equivalent to a compression of 14%. The consolidation behavior of the compacted PS in the centrifugal field indicates a void ratio of 2.28 after the centrifugal tests, which agrees well with the laboratory consolidation test results.

In the case of compacted CS, the initial void ratio of 0.81 is decreased to 0.76 under a surcharge of 120 kPa in the laboratory consolidation test. The settlement that occurred is about 1.5 mm, which is equal to a compression of 2.1% for the thickness of sample. In the

Table 3.5 Initial conditions of compacted PS and CS used for centrifugal loading tests

Index	PS	CS
Molding water content (%)	145.2	25.3
Wet density (g/cm^3)	1.18	1.86
Dry density (g/cm^3)	0.48	1.48
Void ratio	2.73	0.81
Porosity (%)	73.2	44.7
Degree of saturation (%)	95.5	83.9

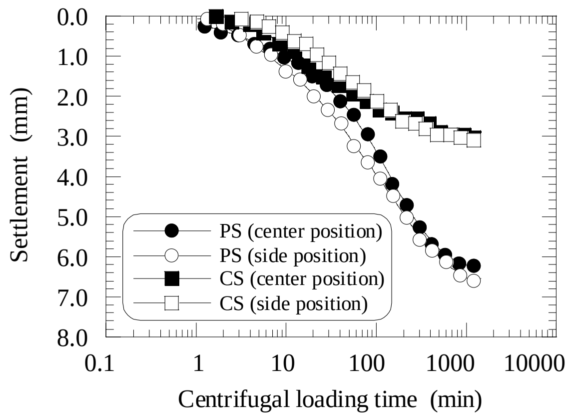

Fig. 3.15 Settlement of PS and CS versus time in centrifuge test at 60 G

centrifuge test, the recorded settlement is about 3 mm under a surcharge of 120 kPa, and the compression of 6% occurs for the total layer thickness of compacted CS. The void ratio of compacted CS at the end of the centrifuge test is 0.70. When results obtained from the two types of tests are compared, larger compressibility occurs in the centrifuge tests. For this reason, the author thought the following. The centrifuge tests provide the simulation of long-term settlement behavior under a loading surcharge of about 120 kPa over 9.86 years. The first consolidation due to the dissipation of excess pore water pressure occurs, and the secondary consolidation with the rearrangement of soil particles will progress after the first consolidation under a loading surcharge of about 120 kPa over 9.86 years. Consequently, to consider the effect of first and secondary consolidation will be more desired for predicting the long-term settlement of compacted CS as a barrier material.

Figure 3.16 shows the estimated consolidation behavior of compacted PS and CS from the results of the centrifuge tests. As noted, the consolidation behavior of the compacted PS and CS is assumed to follow Terzaghi's consolidation theory. The coefficient of consolidation of the PS obtained in the centrifuge tests is 1.03 x 10^{-3} cm^2/s, which is equal to the coefficient of consolidation in its prototype, based on the scaling relationship shown in Table 3.2. From these results, the case in which a surcharge of 120 kPa is applied to the compacted PS having a thickness of 50 cm needs to about 600 hours (about 25 days) to reach

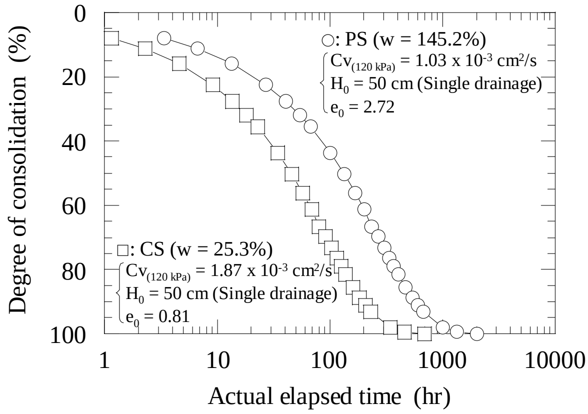

Fig. 3.16 Consolidation behavior of PS and CS in centrifuge test

a 90% degree of consolidation. The measured coefficient of consolidation for the compacted CS by the centrifuge tests is 1.87 x 10^{-3} cm^2/s. When a surcharge of 120 kPa is applied on compacted CS having a 50 cm layer thickness, about 206 hours (8.6 days) is required to achieve a 90% degree of consolidation.

3.4.8. Hydraulic Conductivity in Centrifugal Field

In the centrifuge tests, PS and CS are compacted in a 5 cm-thick layer in a stainless steel tank to estimate their long-term hydraulic conductivity. Initial conditions of compacted PS and CS are shown in Table 3.5. The water level is measured at three places on both the influent and effluent sides during the centrifugal loading period. This is to clarify the formation of the meniscus water surface by the centrifugal loading force.

Figure 3.17 shows the time variation in the influent and the effluent water levels of compacted PS and CS with molding water contents of 145.2% and 25.3%, respectively, under centrifugal loading time. In this figure, it is not possible to confirm the meniscus surface because there is no difference in the changes in the water levels measured at the three places on the influent and effluent sides, respectively. Accordingly, in the calculations of the influent and the effluent water levels, average water levels are used. The changes in the water level are very dull for the compacted PS and CS, while the decreasing ratio of the influent water level to elapsed time and the increasing rate of the effluent water level for elapsed time matched well.

Figure 3.18 shows the time variation in the hydraulic conductivity of compacted PS and CS in the centrifuge tests. The hydraulic conductivity calculated in the centrifugal field at 60 G is 60 times larger than the hydraulic conductivity obtained for the prototype (1 G fierld). Thus, the hydraulic conductivity calculated from the centrifugal field at 60 G should be divided by 60 in order to convert the hydraulic conductivity calculated from the centrifugal field at 60 G into the hydraulic conductivity in the prototype (1 G field). The values of

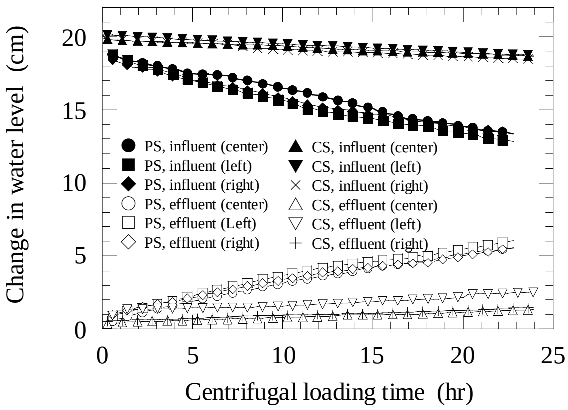

Fig. 3.17 Changes in water level with the centrifugal loading time in centrifuge test

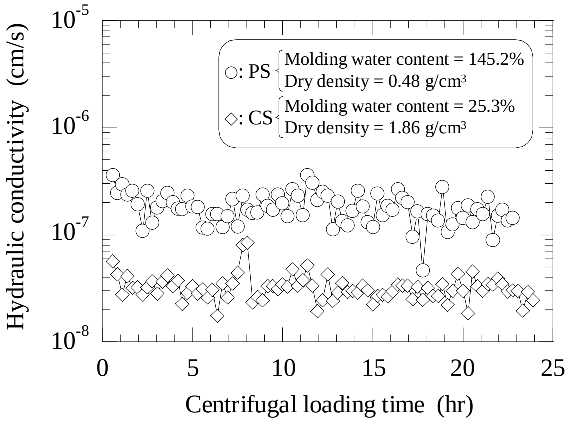

Fig. 3.18 Changes in hydraulic conductivity with centrifugal loading time in centrifuge test

hydraulic conductivity shown in Fig. 3.18 have already been converted to the 1 G field. In the case of compacted PS, the hydraulic conductivity ensures 10^{-7} cm/s order of magnitude over the centrifugal loading of 24 hours. When the hydraulic conductivity obtained by the centrifuge tests is evaluated in the prototype, compacted PS (a layer thickness of 300 cm) maintains a hydraulic conductivity of 4.0 x 10^{-7} cm/s as a barrier material in cover system just after the installation. The hydraulic conductivity decreases to about 1.0 to 2.0 x 10^{-7} cm/s after 600 days from the installation with progress of the consolidation, and it can be assumed that the hydraulic conductivity has almost been stabilized within the 9.86 years. When the hydraulic conductivity obtained from the centrifuge test for compacted PS with a molding water content of 145.2% and a dry density of 0.48 g/cm^3 (*see* Fig. 3.18) is compared with the results obtained from the flexible-wall hydraulic conductivity test for compacted PS with a molding water content of 157.9% and a dry density of 0.45 g/cm^3 (*see* Fig. 3.9(a)), an almost equivalent level of hydraulic conductivity is found in both tests. Consequently, the hydraulic conductivity measured by the flexible-wall hydraulic conductivity tests can be maintained throughout the 9.86 years.

The compacted CS with a molding water content of 25.3% and a dry density of 1.48 g/cm^3 maintains the hydraulic conductivity of 2.0 to 5.0 x 10^{-8} cm/s during the centrifugal loading period (*see* Fig. 3.18). However, the hydraulic conductivity of compacted CS obtained from the flexible-wall hydraulic conductivity test (*see* Fig. 3.9(b)) provides 1.4 x 10^{-8} cm/s under the compaction condition of a molding water content of 26.6% and a dry density of 1.46 g/cm^3. This is almost the same level as the hydraulic conductivity measured in the centrifuge test for compacted CS. When the hydraulic conductivity of CS obtained from the centrifuge test is converted to a prototype, compacted CS can demonstrate a hydraulic conductivity less than the application standards of 1 x 10^{-7} cm/s just after the installation. Furthermore, the low hydraulic conductivity will be maintained throughout 9.86 years.

3.5. Summary and Conclusions

The utilization of PS and CS materials for the landfill cover systems will not only save landfill space, but serve to the cost reduction of landfill construction. Since the PS and CS are considered to be waste products, the construction costs for landfill facilities can be reduced due to reuse of the PS and CS for barrier materials in cover system. The chemical analysis of PS and CS, and the results of hydraulic conductivity tests on effluents indicated a low concentration of heavy metals (i.e., within the Japanese environmental quality standards). Furthermore, the reuse of PS and CS as barrier materials in cover system provides significant pollution control, as well as ecological and economical benefits, in comparison to the utilization of traditional barrier materials such as compacted clay and geosynthetic materials. The test results strongly encourage PS and CS utilization for the barrier materials in cover systems, and the above experimental work leads to the following conclusions:

1) The leaching of heavy metals such as Pb, Cd, and Cr from the PS and CS samples is within the environmental quality standards. Thus, the applications of both PS and CS for barrier materials in the field will not cause serious environmental impacts.
2) The results of consolidated undrained shear tests for compacted PS indicate a higher cohesion and a higher internal friction angle than those of the compacted CS, although the compacted PS has a higher molding water content and a lower dry density than those of the compacted CS. The higher value for the internal friction angle may be mainly attributed to the presence of organic fibers and tissues in the compacted PS. According to the shearing behaviors of compacted PS and CS obtained from the consolidated undrained shear tests, the compacted PS and CS used in the present study can expect to practically form the stable landfill slopes.
3) Compacted PS shows highly compressive behavior. The consolidation of compacted PS due to overburden pressure contributes to a decrease in the hydraulic conductivity of PS with time. The compacted CS does not show high compressibility when compared with

compacted PS. These two types of compacted sludge exhibit over-consolidation states under the overburden pressure of 20 to 30 kPa, which is a typical load applied to the barrier layer.

4) The compacted CS shows the hydraulic conductivities of the range of 1.2 x 10^{-8} to 6.5 x 10^{-8} cm/s. Furthermore, the minimum hydraulic conductivity is obtained in the CS compacted at the water content of wetter by 2 to 8% than the optimum water content of 18.2%. The compacted CS can satisfy the requirements for hydraulic conductivity of landfill covers distributed by the US EPA.
5) The compacted PS shows very little variation in hydraulic conductivity, 3.5 x 10^{-7} to 9.0 x 10^{-7} cm/s, even though the molding water contents range from 50 to 150%. The hydraulic conductivity of compacted PS is not sensitive to the molding water content, which may be advantageous from the viewpoint of quality control during the construction.
6) The settlement of 14% for compacted PS with a layer thickness of 5 cm is observed in the centrifuge test. This agrees well with the results of laboratory consolidation tests. When compacted PS 50 cm in layer thickness is constructed, 25 days will be required to complete the consolidation under a surcharge of 120 kPa.
7) The settlement of compacted CS is small throughout the centrifuge test at 60 G, and is estimated as 6% of the layer thickness. When constructing a barrier layer thickness of 50 cm using compacted CS, 8.6 days will be required to complete the consolidation under a surcharge of 120 kPa.
8) Compacted PS maintains a hydraulic conductivity of 10^{-7} cm/s order of magnitude under a centrifugal field of 60 G over 24 hours, and the test results indicate that PS can maintain this level of hydraulic conductivity for about 9.86 years when used as a barrier material.
9) The hydraulic conductivity of CS maintains 10^{-8} cm/s order of magnitude under a centrifugal field of 60 G over 24 hours. Thus, the compacted CS can exhibit a hydraulic conductivity less than the application standards of 1 x 10^{-7} cm/s just after installation. Furthermore, this low level of hydraulic conductivity will be maintained throughout 9.86 years.
10) The potential utilization of PS (paper sludge) and CS (construction sludge) for barrier materials in landfill cover systems has been examined. Although the long-term behavior of these two types of sludge has been evaluated based on the centrifugal simulation for the consolidation and the hydraulic conductivity, the degradation of the sludge and its durability as a water interception material during at least the service life should be investigated in future.

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CHAPTER 4

Water Balance in Cover System for Minimizing Geo-Environmental Impacts

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4.1. General Remarks

Engineered waste containment facilities are required to have a surface cover and a bottom liner in order to minimize the geo-environmental contamination caused by the leakage of waste leachate. The purpose of a landfill cover system is to reduce the infiltration of rainwater into the underlying waste layer, and consequently, to reduce the generation of waste leachate. The purpose of a bottom liner is to serve as a final defense against the leaking of waste leachate which might contain toxic chemicals. The United States and the most European countries have regulations for surface covers as well as for bottom liners. In Japan, however, no such regulations have been determined for surface covers. In fact, a semi-aerobic landfill approach is adopted in the Final Waste Disposal Guideline in 1979 issued by the Ministry of Health and Welfare, Japan. Most of the landfills designed by this technology in Japan are characterized by no usage of cover systems. It is explained that a partial aerobic digest in addition to the anaerobic digest within these landfills without cover systems may promote the decomposition of wastes at a higher rate and, therefore, to stabilize the landfills in a shorter time than the traditional anaerobic landfills with cover systems (Hanashima and Furuichi, 2000). Anaerobic decomposition provides a reducing condition within the landfills, which is favorable to the fixation of heavy metals, tending to decrease the pollution risk of leachate to the nearby environment. Thus, the importance of cover systems in Japan is stressed in the present research. The feasibility of waste sludge as an alternative to compacted clays with low hydraulic conductivity is also proposed from laboratory experiments. Compacted clays have generally been used for the barrier layer materials of cover systems in the United States and Europe. If waste sludge can be applied as a barrier layer material, it would be a great response to social requests to reduce landfill site construction costs and to rationally reduce the waste to be landfilled.

In waste management practices, the primary function of a landfill cover system is to minimize the rainwater penetration into the landfill during the inactive (post-closure) period of the landfill. Estimating the amount of leachate produced after the closure of a landfill is an important task for landfill designers. The answer ultimately depends on the quantity of water percolating from the cover system (Khire et al., 1997). Investigators have carried out extensive water balance analyses for landfill cover systems (e.g., Benson et al., 1994; Chiu and Shackelford, 1998). The quantity of water percolating to an underlying waste layer from a cover system depends on several factors, such as the hydrological conditions of the landfill site, the cross sectional composition of the cover system, the top slope of the closed landfill, and the existence or nonexistence of vegetation. Predictions of the migration of rainwater and melted snow, which generally pass through the cover system, can be simulated using hydrological models. A few models are commercially available, for example, the Hydrologic Evaluation of Landfill Performance (HELP) model has been widely used in the United States to evaluate the performance of rainwater interception in landfill cover systems.

This Chapter discusses the performance of cover systems with sludge barriers at several sites in Japan. Kamon et al. (2000, 2001a, 2001b, 2002) have reported the same discussions as this Chapter. By using a water balance analysis, the following items are evaluated: (1) the effects of final and daily cover systems on reducing the quantity of rainwater percolating to the waste layers from the cover systems, under the humid climatic conditions in Japan, and (2) the performance of rainwater interception of final cover systems on the changes in the hydraulic conductivity and thickness of the sludge barrier layers. An evaluation of the effects of the installation of daily and final cover systems on leachate reduction is also performed during the post-closure period of a landfill from the beginning of the waste reclamation. For the water balance analysis, an analytical method of the HELP model, proposed by Thornthwaite and Mather (1955), has been applied, which can evaluate simply the rainwater interception ratio of daily and final cover systems.

4.2. Water Balance Analysis

4.2.1. Definition and Significance of Water Balance Analysis

One of the most important functions of a landfill cover system is to limit or eliminate the production of leachate in underlying waste by minimizing or eliminating percolation of water through the cover system. The analysis of water routing in cover systems is called water balance analysis. The reasons why designers or regulators analyze water balance in covers may include one or more of the following: (1) to compare alternative design profile and materials, (2) to help to understand how the cover will function and which water routing mechanisms are most important, (3) to estimate flow rates so that components of the system can be sized properly, and (4) to estimate the amount of contaminated liquid the will be generated. This value can be used as input to a fate-and-transport model of impacts on ground water. Fate-and-transport modeling is often a critical component of risk-based corrective action for site remediation projects. The fourth objective listed above is probably the most underutilized one, especially in landfill cover systems.

4.2.2. Water Balance Analysis for Cover Systems

To evaluate the effectiveness of cover systems and also to design facilities for leachate treatment, it is essential to analyze the water balance in landfills, especially to predict the quantity of water percolating into the waste layer from the cover system. Figure 4.1 conceptually shows the migratory pathway of rainwater when it passes through a cover system at a waste landfill site (Khire et al., 1997). Infiltration precipitation (rainwater) into landfill can be minimized by: (1) the effect of the evapotranspiration in the surface layer, (2) the effect of water retention in the surface layer, (3) the effect of the surface runoff of precipitation due to surface inclination, (4) the effect of drainage from the drainage layer, and (5) the effect of the low hydraulic conductivity of the barrier layer in the final cover system.

Estimations of the infiltration of rainwater or melted snow into a cover system are usually accomplished using one of the many available water balance analysis models. The best known one is the Hydrologic Evaluation of Landfill Performance (HELP) model (Schroeder et al., 1984a, 1984b). The HELP model simulates the hydrologic processes for active or closed landfills by performing sequential water balance calculations using a quasi two-dimensional approach, which considers all flow to be vertical, except at the lateral drainage layer, where the flow can be vertical or lateral. The simulation progresses with time, and the water balance process is thought to be steady within each time step. A conceptualization of the HELP model is shown in Fig. 4.2. The model can be used to evaluate each subsurface profile separately, as shown in Fig. 4.2. The hydrologic processes considered in the model include precipitation, surface water storage, the interception of precipitation by

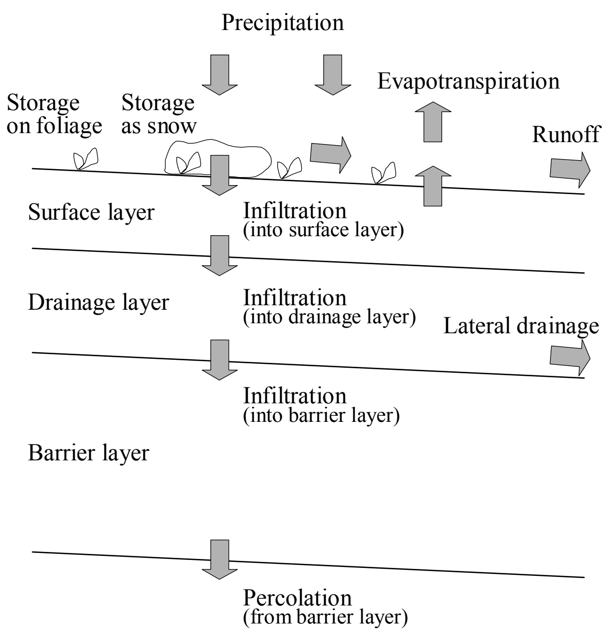

Fig. 4.1 Identification of water movement and storage in a cover system

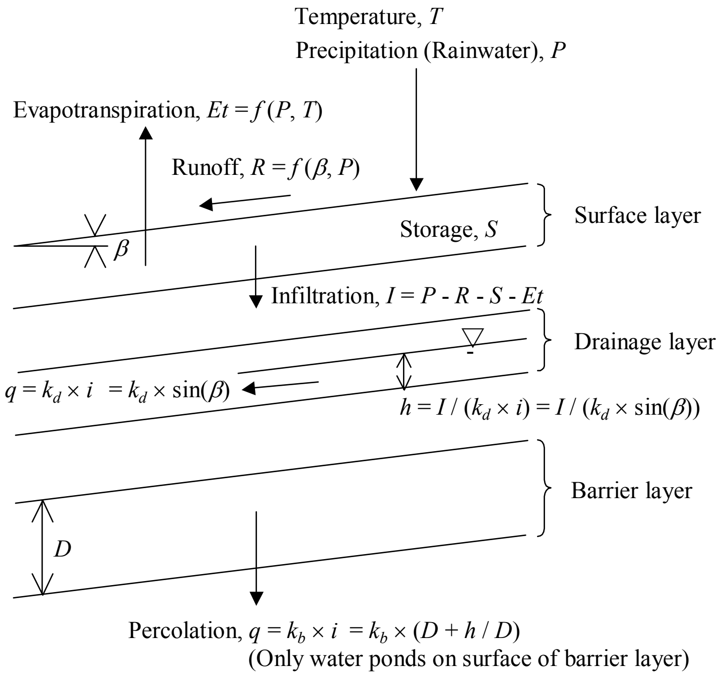

Fig. 4.2 Schematic representation of the HELP model

foliage, surface water evaporation, runoff, melted snow, infiltration, plant transpiration, soil water evaporation, soil water storage, vertical flow (saturated and unsaturated) through non-barrier soil layers, vertical percolation (saturated) through soil barrier layers, vertical percolation (saturated) through geomembrane and geomembrane-soil composite barrier layers, and lateral or vertical flow (saturated) through drainage layers (Rowe, 2000). Water balance analyses for landfill cover systems, using the HELP model, have been performed in several studies (e.g., Benson et al., 1994; Khire et al., 1997). For instance, Khire et al. (1997) used the HELP model to predict the percolation through the cover system at Live Oak Landfill in Atlanta, Georgia.

A water balance analysis using the HELP model may be performed by hand or with a computer. In the present study, a water balance analysis by hand, using an algorithm of the HELP model, is conducted to predict the performance of the rainwater interception of a cover system with sludge barriers. Details of the procedure are discussed in a later section. This procedure has also been recommended by Thornthwaite and Mather (1955), Fenn et al. (1975), and Kmet (1982).

4.3. Analysis Methods of Water Balance in Cover Systems

The fundamental aspect in evaluating water transport is defined by the mass conservation law. When precipitation passes through the earthen cover surface layer to the drainage layer, the mass conservation law and water balance can be expressed by Eq. (4.1) (Daniel 1993):

$$I = P - R - S - Et \tag{4.1}$$

where I is the water quantity infiltrating into the drainage layer from the surface layer (mm), P is the average monthly precipitation at the landfill site (mm), R is the quantity of surface runoff at the landfill site (mm), S is the quantity of water that can be retained in the surface layer (mm), and Et is the quantity of water loss due to evapotranspiration (mm).

Runoff quantity, R, overflowing onto the surface of a cover system, can be estimated according to the following Eq. (4.2):

$$R = P \times C \tag{4.2}$$

where C is the surface runoff coefficient. Surface runoff coefficient, C, is defined by the type of composed soil and the angle of the surface layer in the cover system. Typical runoff coefficients for completed landfill covers are given in Table 4.1.

Evapotranspiration, Et, provides the direct water loss and the transpiration by the plants on the surface layer. Since the determination of evapotranspiration is complicated, it can be simplified by the empirical formula expressed in Eqs. (4.3a), (4.3b), and (4.3c) (Thornthwaite and Mather, 1955), and can be defined as a function of the average temperature and the duration of sunlight:

$$Et = 0 \qquad (T \le 0°\text{C}) \tag{4.3a}$$

$$Et = N \times 0.53 \left(\frac{10T}{H_a} \right)^a \qquad (0°\text{C} < T < 27°\text{C}) \tag{4.3b}$$

$$Et = N \times (-0.015T^2 + 1.093T - 14.208) \qquad (T \ge 27°\text{C}) \tag{4.3c}$$

Table 4.1 Typical runoff coefficients for storms occurring at 5- to 10-year frequencies (Developed in part by Frevert et al., 1955; Fenn et al., 1975; Linsley, 1991)

Types of soil	Slope (%)	Runoff coefficient C			
		With grass		Without grass	
		Range	Typical	Range	Typical
Sandy loam	2	0.05 - 0.10	0.06	0.06 - 0.14	0.10
	3 - 6	0.10 - 0.15	0.12	0.14 - 0.24	0.18
	7	0.15 - 0.20	0.17	0.20 - 0.30	0.24
Silty loam	2	0.12 - 0.17	0.14	0.25 - 0.35	0.30
	3 - 6	0.17 - 0.25	0.22	0.35 - 0.45	0.40
	7	0.25 - 0.36	0.30	0.45 - 0.55	0.50
Tight clay	2	0.22 - 0.33	0.25	0.45 - 0.55	0.50
	3 - 6	0.30 - 0.40	0.35	0.55 - 0.65	0.60
	7	0.40 - 0.50	0.45	0.65 - 0.75	0.70

$$a = (6.75 \times 10^{-7}) H_a^3 - (7.71 \times 10^{-5}) H_a^2 + 0.01792 H_a + 0.49239$$

$$H_a = \sum_{i=0}^{i=n} (H_i)^{1.514}$$

$$H_i = (0.2T)^{1.514} \qquad (T \geq 0°\text{C})$$

$$H_i = 0 \qquad (T \leq 0°\text{C})$$

where T is the average monthly temperature (°C) and N is the duration index of sunlight which is assumed from the latitude at which the landfill site is located. Details on the relationship between N and the latitude are given in Koerner and Daniel (1997).

The total quantity of water that can be retained in a unit volume of surface soil layer, S, depends on the field capacity (FC) and permanent wilting percentage (PWP) of them. The soil-water suction at FC is typically between 100 kPa and 333 kPa (Tchobanoglous et al., 1993; Hansen et al., 1979). The PWP is defined as the quantity of water left in the soil when plants are no longer able to extract any more. The soil-water suction at the PWP is approximately 1500 kPa (Tchobanoglous et al., 1993; Hansen et al., 1979). Typical FC and PWP values based on the volumetric water content for representative soils are given in Table 4.2. In the water balance analysis, the maximum quantity of water that can be retained in a unit volume of the surface layer, $S_{\max}$ (mm), is determined by Eq. (4.4), in other words:

Table 4.2 Typical volumetric water content values at field capacity and permanent wilting points for various soils (Adapted from Thornthwaite et al., 1955; U.S. Army Corps of Engineers, 1956; Fenn et al., 1975; Hansen et al., 1979; Linsley et al., 1991)

Soil classification	Value (%)			
	Field capacity		Permanent wilting point	
	Range	Typical	Range	Typical
Sand	0.06 - 0.12	0.06	0.02 - 0.04	0.04
Fine sand	0.08 - 0.16	0.08	0.03 - 0.06	0.05
Sandy loam	0.10 - 0.18	0.14	0.04 - 0.08	0.06
Fine sandy loam	0.14 - 0.22	0.18	0.06 – 0.10	0.08
Loam	0.18 - 0.26	0.22	0.08 - 0.12	0.10
Silty loam	0.19 - 0.28	0.24	0.09 - 0.14	0.10
Light clay loam	0.20 - 0.30	0.26	0.10 - 0.15	0.11
Clay loam	0.23 - 0.31	0.27	0.11 - 0.15	0.12
Silty clay	0.27 - 0.35	0.31	0.12 - 0.17	0.15
Heavy clay loam	0.29 - 0.36	0.32	0.14 - 0.18	0.16
Clay	0.31 - 0.39	0.35	0.15 - 0.19	0.17

$$S_{\max} = (\theta_{FC}) \times H \times 1000 \tag{4.4}$$

where θ_{FC} is the field capacity of the volumetric water content and H is the thickness of the surface layer (m). When the precipitation is not infiltrated into the surface layer due to the quantity of rainwater, the runoff effect, and the evapotranspiration effect, the water retained in the surface layer evaporates, and the quantity of water retained in the surface layer is decreased, as shown in Eq. (4.5a) given by Koerner and Daniel (1997). When the precipitation infiltrates into the surface layer, the quantity of water retained in the surface layer is added to the quantity of water retained in the previous month and the quantity of infiltration of the concerned month (*see* Eq. (4.5b)) (Koerner and Daniel, 1997). However, the quantity of water that exceeds the maximum quantity of water that can be retained in the surface layer, $S_{\max}$, cannot be stored (Koerner and Daniel, 1997), in other words:

$$S_i = (S_{\max}) \times 10^{b(P-R-Et)} \qquad (P - R - Et \leq 0) \tag{4.5a}$$

$$b = \frac{0.455}{S_{\max}}$$

$$S_i = (P - R - Et)_i + S_{i-1} \qquad (P - R - Et > 0) \tag{4.5b}$$

where S_i is the possible water retention quantity of the surface layer in the *i*th month (mm).

Using the above equations, the quantity of water infiltrating into the drainage layer from the surface layer can be obtained. In the drainage layer, some of the water can be drained in an inclined direction as per Darcy's rule, as shown in Eq. (4.6), under the assumption that the barrier layer is fully saturated (Othman et al., 1995), namely:

$$h = \frac{q}{k_d \cdot \sin(\beta)} \tag{4.6}$$

where h is the water level in the drainage layer (cm), q is the quantity of infiltration flow of the drainage layer (cm^3/s), k_d is the saturated hydraulic conductivity of the drainage layer (cm/s), and β is the angle of the drainage layer (°).

According to Darcy's law, the quantity of water leaking out of the barrier layer, which is the same as the quantity of water percolating from the cover system, can be obtained with the following Eq. (4.7):

$$q = k_b \frac{h + D}{D} \tag{4.7}$$

where q is the quantity of water percolating from the barrier layer (cm^3/s), k_b is the saturated hydraulic conductivity of the barrier layer (cm/s), and D is the thickness of the barrier layer (cm).

For this analysis, the average monthly precipitation, mean temperature, and average solar radiation over the past thirty years at each site were used as the input data. The data were taken from the weather databases of the Japan Weather Association in 1998.

4.4. Performance of Final Cover Systems with Sludge Barrier

4.4.1. Geographical effect

An evaluation of the performance of rainwater interception for final cover systems that applied PS and CS materials as the barrier layer materials is performed, considering the changes in weather at each landfill site. The assumed structural profile and the characteristic values of the constituent layers of the final cover systems are shown in Fig. 4.3. Sludge barriers with a hydraulic conductivity of 1 x 10^{-7} cm/s are assumed, because the hydraulic conductivities of the CS can be maintained at levels lower than or equal to 1 x 10^{-7} cm/s from the discussion in Chapter 3. In addition, the structural profile and the characteristic values of the constituent layers are recommended for landfill cover systems by the United States Environmental Protection Agency (US EPA).

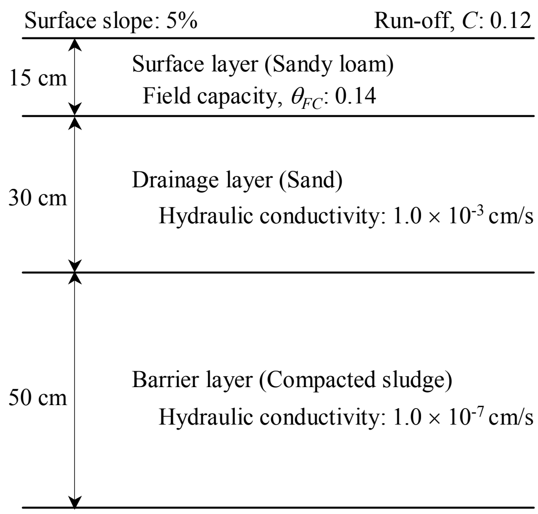

Fig. 4.3 Assumed structural profile and characteristic values for constituent layers of final cover systems

Table 4.3 and Fig. 4.4 summarize the annual cumulative precipitation, infiltration, and percolation for the assumed final cover systems based on a water balance analysis at some specific locations. In the present state of landfill management in Japan, only a surface layer (top soil layer) is constructed. The calculated results indicate a rainwater interception ratio of 32 to 72% when only a surface layer is installed. The rainwater interception ratio is defined as the ratio of the infiltration or the percolation water to the precipitation, as expressed in Eq. (4.8), namely:

$$W.I.E. = \left(1 - \frac{\text{Infiltration or Percolation}}{\text{Precipitation}} \right) \times 100 \qquad (4.8)$$

where *W.I.E.* is the ratio of the rainwater interception (%). The ratio of the rainwater interception, when only a surface layer is used, directly depends on the changes in weather conditions, especially the changes in precipitation at each landfill site. Furthermore, the ratio of the rainwater interception decreases at sites where heavy precipitation occurs. When a structural type of final cover system is established, in which the sand drainage layer and the sludge barrier layer are placed on the bottom of the surface layer, the ratio of the rainwater interception by the final cover system is at a very high level, namely, from 97 to 99%. This shows an almost complete sealing off of the entire quantity of precipitation at each site. The ratio of the rainwater interception of the final cover system tends to increase in regions where there is a heavy precipitation. It can be found that the highest ratio of the rainwater interception of a final cover system in Japan appears in Owase City, which has 2.2 times more precipitation (3930 mm/y) than average (1760 mm/y). The quantity of water percolating from

Table 4.3 Effects of rainwater interception of final cover systems with a sludge barrier installed at assumed landfill sites in Japan

Site	Latitude (A)	Temperature (B) (°C)	Precipitation (C) (mm/y)	Runoff (D) (mm/y)	Evapo-transpiration (E) (mm/y)	Infiltration into drainage layer (F) (mm/y)	Percolation from cover system (G) (mm/y)	Rainwater interception by surface layer (H) (%)	Rainwater interception by cover system (I) (%)
Sapporo	43°03′	8.43	1106.3	132.8	473.8	499.7	33.1	54.8	97.0
Sendai	38°16′	12.0	1205.1	144.6	700.7	359.8	32.5	70.1	97.3
Tokyo	35°41′	15.8	1439.6	172.8	793.1	473.8	33.0	67.1	97.7
Kyoto	35°01′	15.5	1544.7	185.4	823.5	535.8	33.2	65.3	97.9
Osaka	34°41′	16.4	1296.0	155.5	786.1	354.4	32.5	72.7	97.5
Owase	34°04′	15.7	3929.6	471.6	821.6	2636.5	41.7	32.9	98.9
Kochi	33°33′	16.5	2515.8	301.9	872.6	1341.3	36.5	46.7	98.5
Kumamoto	32°49′	16.3	1990.1	238.8	873.1	878.2	34.6	55.9	98.3
Naha	26°12′	22.6	1973.0	236.8	1130.5	605.7	33.5	69.3	98.3

H = (1 - F/C) × 100 , I = (1 - G/C) × 100

Barrier layer: $k = 1.0 \times 10^{-7}$ cm/s, $h = 50$ cm

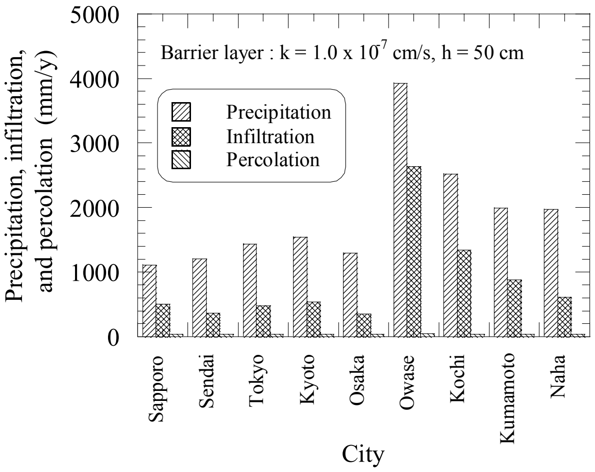

Fig. 4.4 Results of water balance analysis for final cover systems with a sludge barrier at each site

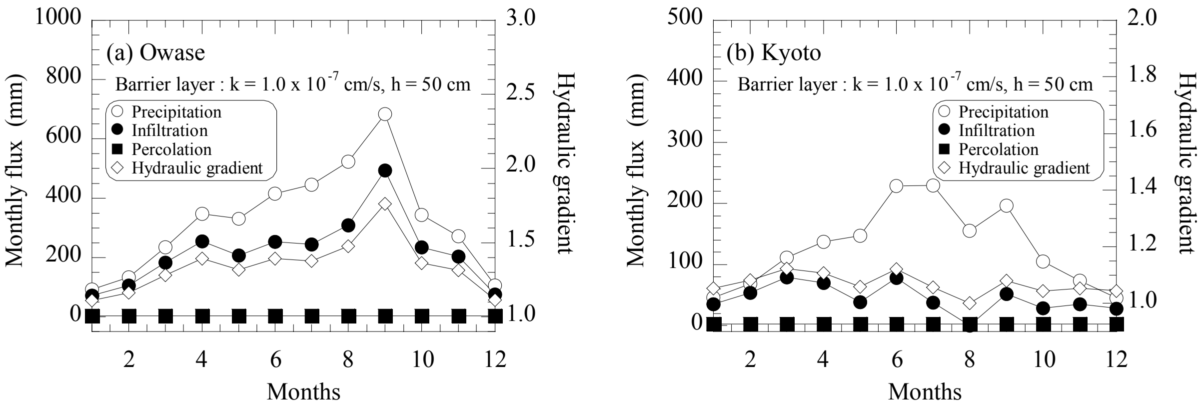

Fig. 4.5 Changes in water balance through final cover systems, with elapsed time, in (a) Owase and (b) Kyoto

the barrier layer is almost constant, regardless of the regional differences in precipitation. This tendency is clearly seen from Fig. 4.4. The ratio of the rainwater interception of the final cover system increases at sites with heavier precipitation.

Figure 4.5 shows the changes in water balance in the final cover systems, with elapsed time, in Owase and Kyoto. The hydraulic conductivity and the thickness of the sludge barrier layers in both cities are assumed to be 1 x 10^{-7} cm/s and 50 cm, respectively. The precipitation in Kyoto is heavier in summer (August) than in winter. The quantity of infiltrated water that passed through the surface layer into the drainage layer at each site depends on the quantity of precipitation. However, the quantity of infiltration decreases with the increase in evapotranspiration during periods of high temperature. Even though a hydraulic gradient is changed in the range of 1.1 to 1.8 and a percolating water quantity is proportional to the change in the hydraulic gradient, the water quantity percolating from the sludge barrier layer is at a lower level of 2.8 to 3.8 mm/month throughout the whole year in Owase; this level is almost equivalent to that of Kyoto. This can be explained by Darcy's rule, since the quantity of water percolating from the sludge barrier layer is determined by the hydraulic conductivity and the hydraulic gradient. Changes in hydraulic gradient in the range of 1.1 to 1.8, at a low hydraulic conductivity of 1 x 10^{-7} cm/s, hardly affect the quantity of the percolated water. The quantity of water percolating from the final cover system with a barrier layer thickness of 50 cm seems to depend on the hydraulic conductivity of the barrier layer. The assumed final cover system is able to intercept almost the entire quantity of precipitation at all specified sites in Japan. Furthermore, it has been indicated that the hydraulic conductivity and the layer thickness of a sludge barrier are a critical elements for the water interception performance of a cover system.

4.4.2. Effect of sludge barrier

The performance of rainwater interception of a final cover system with sludge barrier layer which has various conditions in the hydraulic conductivity and the layer thickness is evaluated. The assumed structural profile and the characteristic values of the surface and the drainage layers of a final cover system are similar, as seen in Fig. 4.3.

Figure 4.6 shows the relationship between annual average levels of hydraulic gradient in terms of the layer thickness of the sludge barrier layers which are usually constructed at a thickness of 50 to 90 cm. The variations in hydraulic gradient for Kyoto and Owase are 1.04 to 1.07 and 1.20 to 1.35, respectively. There is no significant variation in the level of hydraulic gradient with changing barrier layer thickness within a range of 50 to 90 cm. Figure 4.7 shows the annual average rainwater interception ratio for the changing in hydraulic conductivity of the sludge barrier layer in Owase. If the sludge barrier layer can ensure a

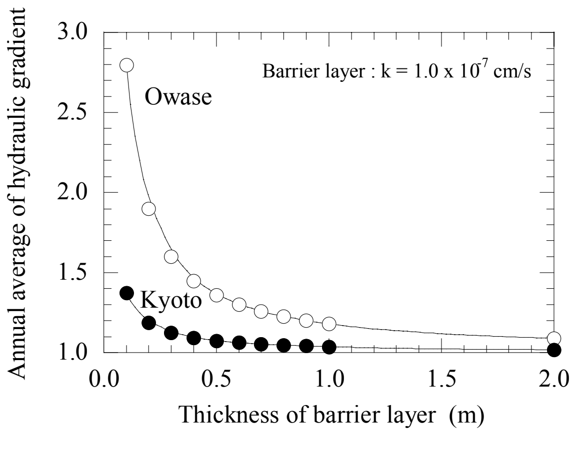

Fig. 4.6 Annual average hydraulic gradient for the thickness of sludge barriers in Owase and Kyoto

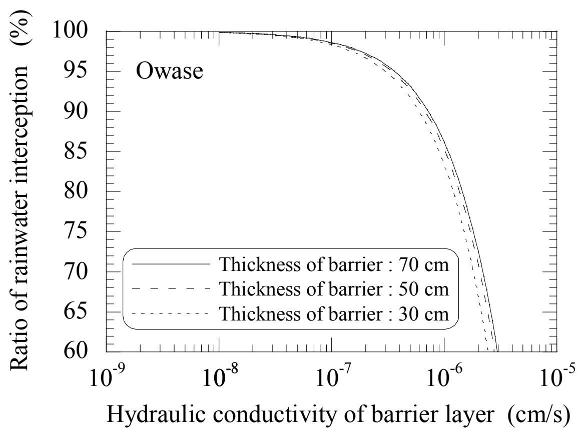

Fig. 4.7 Annual average ratio of rainwater interception for changing in the hydraulic conductivity of the sludge barrier layer in Owase

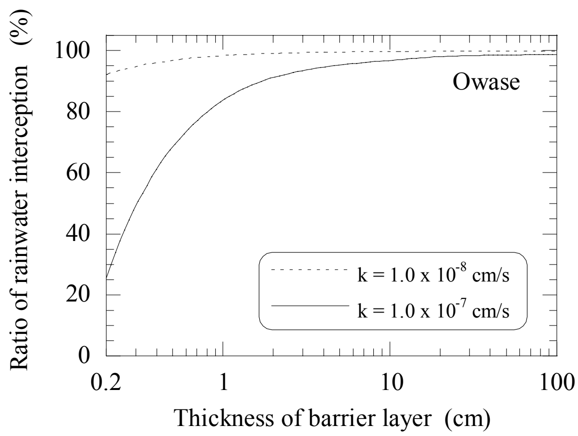

Fig. 4.8 Relationship between sludge barrier thickness and rainwater interception ratio in Owase

hydraulic conductivity of less than 1 x 10^{-7} cm/s, with a thickness of 30 to 70 cm, the ratio of rainwater interception can achieves at almost 100%. The relationship between the thickness of the sludge barrier layer and the rainwater interception ratio in Owase is shown in Fig. 4.8. If the thickness of the assumed layer is more than 10 cm, with a hydraulic conductivity of 1 x 10^{-7} cm/s, a ratio of water interception of more than 95% can be achieved. A level of hydraulic conductivity of 1 x 10^{-7} cm/s is exhibited for compacted CS through flexible wall hydraulic conductivity tests, which were discussed in Chapter 3.

4.5. Performance of Daily Cover Systems with Sludge Barriers

In order to evaluate the effect of rainwater interception on daily cover systems, daily changes in the weather conditions have to be considered. The procedure for analyzing the water balance is basically the same as that for final cover systems. The input data of the weather conditions used are the daily precipitation, the daily mean temperature, and the daily average amount of solar radiation in 1998. The analysis period selected is one month in length, which is considered to be the longest period of exposure for a daily cover system. The assumed structural profile for the daily cover system is shown in Fig. 4.9. The ratio of rainwater interception of the daily cover system is evaluated for the assumed structural type in which the sludge barrier layer is placed on the bottom of the surface layer.

Figure 4.10 shows the changes in the quantity of water percolating from the daily cover systems in Owase and Kyoto, where the sludge barrier layers are assumed to have a thickness of 30 cm and to have hydraulic conductivity levels of 1 x 10^{-5} cm/s and 1 x 10^{-6} cm/s, respectively. In addition, the variations in precipitation are also shown in Fig. 4.10. As noted, the assumed hydraulic conductivity of the sludge barrier layers is higher than that of the final cover systems. This is because ensuring a hydraulic conductivity of 1 x 10^{-7} cm/s for the sludge barrier layer may be difficult during the short period of field construction of a daily

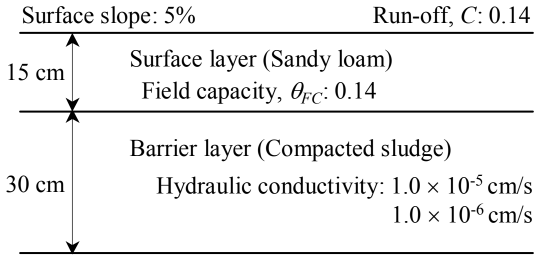

Fig. 4.9 Assumed structural profile and characteristic values for the constituent layers of a daily cover system

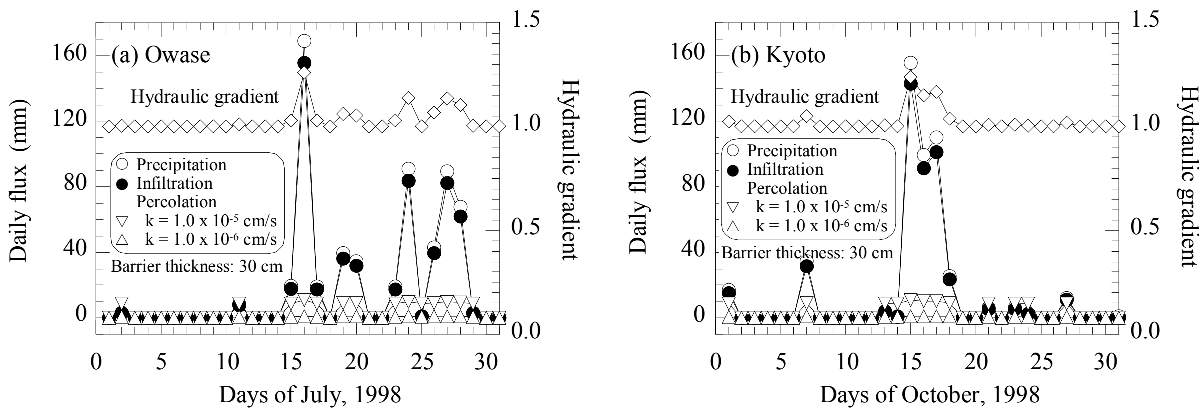

Fig. 4.10 Daily changes in the water balance in the daily cover systems with sludge barriers in (a) Owase and (b) Kyoto

cover system. Although the daily variations in precipitation and in the infiltration into the sludge barrier layer are considerable, the quantity of water percolating from the sludge barrier (i.e., from the daily cover system) is verified very little when the sludge barrier layer has a hydraulic conductivity of 1 x 10^{-6} cm/s or 1 x 10^{-5} cm/s. A water interception ratio of more than 98%, as a cumulative flux over one month, can be obtained if the sludge barrier layer can maintain a hydraulic conductivity of less than 1 x 10^{-5} cm/s. From the test results obtained, such a low hydraulic conductivity for compacted PS and CS is possible to be achieved.

4.6. Effect of Cover Systems on Leachate Reduction

4.6.1. Analysis Method

(a) Water balance in waste layer

To evaluate the effect of installation of cover systems on leachate reduction, a quantification of the leachate generated in the waste layer during the post-closure period of the landfill from the beginning of the waste reclamation is needed.

The water infiltrating into the waste layer, which is the same as the quantity of water percolating from the cover system, is either retained in the waste layer or generated as leachate. Any soil is capable to retain the water against the force of gravity. The quantity of water that can be retained against the force of gravity is referred to as a field capacity (FC). If the quantity of water in soil exceeds the maximum water retention capacity (i.e., the field capacity), excess water, referred to as free water, is generated. As to the waste layer discussed here, if the quantity of water percolating from cover system is greater than the maximum quantity of water that can be retained in the waste layer, excess water is generated as leachate from the waste layer.

The degree of field capacity of waste varies depending on the type of waste and the reclamation methods of the waste and is difficult to estimate completely. In the present study, a homogeneous waste layer, which composed of solid waste, is assumed, and the following Eq. (4.9) proposed by Huitric et al. (1980) is used for estimating the field capacity of the homogeneous solid waste:

$$FC = 0.6 - 0.55 \times \left(\frac{M}{180000 + M} \right) \tag{4.9}$$

where FC is the field capacity (i.e., the fraction of water in the waste based on the dry weight of the waste) and M is the overburden weight calculated at the mid-height of the waste (kPa). The effect of cover system on leachate reduction in waste layer is evaluated, in the present study, under the assumption in which the Eq. (4.9) is applicable to estimate the field capacity of solid waste.

The maximum quantity of water that can be retained in the waste layer, $S_{\max}$ (mm), then can be predicted by of the Eq. (4.10):

$$S_{max} = \frac{FC \cdot \gamma_t \cdot (1 - w/100) \cdot H}{\gamma_w} \times 1000 \tag{4.10}$$

where γ_t is the unit wet weight of the waste (kN/m^3), w is the water content (i.e., liquid mass / total mass) of the solid waste (%), γ_w is the unit weight of water (kN/m^3), and H is the thickness of the waste layer (m).

(b) Case study

During the initial and the final stages of waste reclamation, the application of both daily and final cover systems is assumed for the calculation. From the first day until the operation of the landfill site reached an equilibrium condition, the quantity of leachate that is generated in the waste layer is calculated on the basis of the water balance (Tchobanoglous et al., 1993). An equilibrium condition means a condition at that the quantity of water which infiltrates into the waste layer equals the quantity of water that leaches out. All the conditions assumed for this case study of the landfill site are shown in Table 4.4. Waste is reclaimed at a thickness of 3 m per lift, and a daily cover system composed of a sludge barrier layer and a surface layer, expressed in Fig. 4.9, is installed at each lift. The capacity of the landfill is assumed to be five lifts of waste and duration of one lift to be one year so that the reclamation of waste is completed in five years. The final cover system, composed of a sludge barrier, a drainage layer, and a surface layer, as expressed in Fig. 4.3, is placed at the final stage. Figure 4.11 presents the definitions for the landfill site model and the water balance model used in this

Table 4.4 Assumed conditions for the landfill site used in the case study

	Solid waste	Daily cover system		Final cover system		
		Barrier layer (PS)	Surface layer (sandy loam)	Barrier layer (PS)	Drainage layer (sand)	Surface layer (sandy loam)
Thickness (cm)	300*	30	15	50	30	15
Hydraulic conductivity (cm/s)	-	1×10^{-6}	-	1×10^{-7}	1×10^{-2}	-
Unit wet weight (kN/m^3)	6.0	17.8	16.0	17.5	17.5	16.0
Water content (%)	20**	-	-	-	-	-
Assumed site	Owase					
Total number of lift	5 (one corresponding to each year)					
Duration of waste filling	5 years					

* Height of one lift

** Water content = (Liquid mass / Total mass) x 100

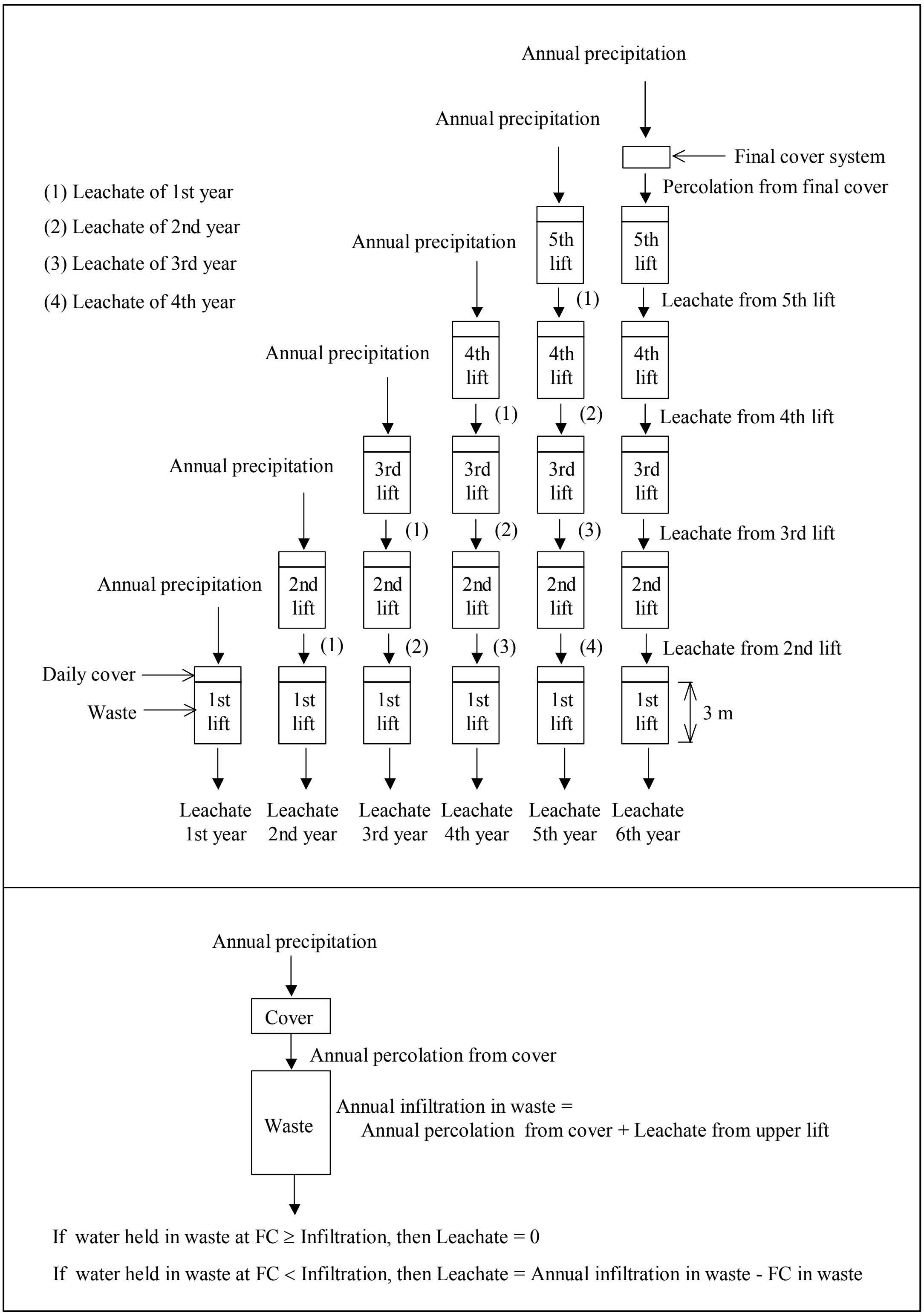

Fig. 4.11 Definitions of the landfill site and the water balance models used in the case study

case study. The annual cumulative quantity of water percolating from the daily and the final cover systems were used for the quantity of water infiltrating into the waste layer in each year.

4.6.2. Estimation of the Leachate Reduction

The quantity of water infiltrating into the waste layer corresponds to the quantity of water percolating from the daily cover system during the waste reclamation stage. It also corresponds to the quantity of water percolating from the final cover system after the completion of the reclamation. Thus, it is necessary to determine the quantity of water percolating from both daily and final cover systems. Figure 4.12 shows the cumulative monthly precipitation, the infiltration into the surface layer, and the percolation from each cover system in Owase. A sludge barrier layer with a thickness of 30 cm and a hydraulic conductivity of 1 x 10^{-6} cm/s is assumed here for the daily cover system. In addition, the application of a sludge barrier layer with a hydraulic conductivity of 1 x 10^{-7} cm/s and a layer thickness of 50 cm is assumed for the final cover system. The quantity of water percolating from the daily cover system, which infiltrates into the waste layer during the waste reclamation stage (i.e., five years from the time the operation of the landfill begins), is quantified as 431.7 mm/y. After five years, when the waste reclamation is complete, the quantity of water infiltrating into the waste layer is 41.7 mm/y due to the installation of the final cover system. However, if only a surface layer is installed and neither a daily nor a final cover system is adopted, 2636.7 mm/y of water will infiltrate into the waste layer during the operating years.

The quantity of leachate that is generated in the waste layer, with or without the installation of daily and final cover systems, is shown in Fig. 4.13. For the first five years before the waste reclamation is completed, the quantity of leachate generated in the waste layer tends to increase with the progression of time. The absolute quantity of leachate

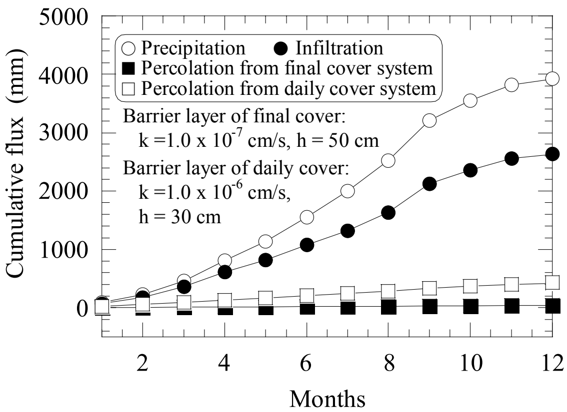

Fig. 4.12 Cumulative flux of percolation from daily and final cover systems with a sludge barrier installed in Owase

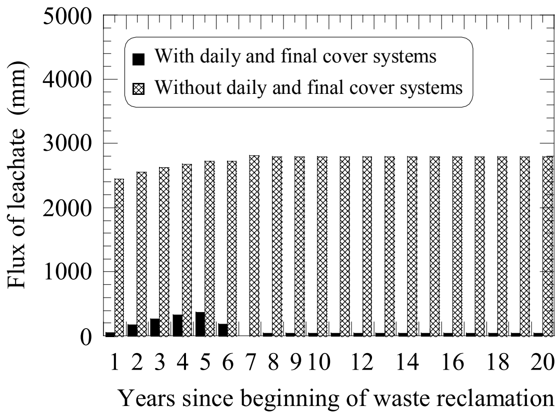

Fig. 4.13 Amount of leachate generated in waste layer with/without the installation of daily and final cover systems in Owase

generated in the waste layer when a daily cover system is used during the waste reclamation stage, can be decreased to 10% of that when only a surface layer is used. Furthermore, the quantity of leachate generated in the waste layer through the installation of a final cover system decreases to 1.7% of cases when only a surface layer is installed, when the landfill site reaches an equilibrium condition. Consequently, the installation of an assumed cover system can significantly contribute to a reduction in the generation of leachate in the waste layer.

The installation effect of daily and final cover systems on the reduction of leachate generated in waste layer is indicated quantitatively, under assuming the waste layer composed of solid waste. However, it is thought that the water retention characteristics of waste, which is mainly represented by the field capacity, is different with depending on the type of waste as well as the reclamation methods. When the present state of waste management in Japan is especially taken into consideration, it should be fully considered that the type of waste reclaimed in the landfill has been changing into mainly incinerator ash. Thus, after completely understanding the water retention characteristics of incinerator ash, further, the installation effect of the daily and final cover systems on the reduction of leachate generated in the incinerator ash will have to discuss.

4.7. Summary and Conclusions

In this Chapter, the effects of final and daily cover systems with sludge barriers on reducing the quantity of rainwater percolating to the waste layers from the cover systems, under Japanese climatic conditions, have been evaluated using a water balance analysis. The effects of installation of daily and final cover systems with sludge barriers on leachate reduction during the post-closure period of a landfill from the beginning of the waste reclamation has also been evaluated. The conclusive results of this study are as follows:

1) Under the protective surface layer, if a final cover system, which is composed of sand

drainage layer and the sludge barrier layer (a hydraulic conductivity of 1 x 10^{-7} cm/s and 50 cm in thickness), is installed, the rainwater can be significantly intercepted, namely, from 97 to 99%. This shows an almost complete sealing off of the entire annual precipitation at each site in Japan.

2) When sludge barriers have a layer thickness of more than at least 10 cm with a hydraulic conductivity of 1 x 10^{-7} cm/s, a rate of water interception of more than 95% can be achieved.
3) The quantity of the water percolating from sludge barriers (i.e., from daily cover system) cannot be verified if the sludge barrier layers have a hydraulic conductivity of 1 x 10^{-6} cm/s or 1 x 10^{-5} cm/s and a thickness of 30 cm. An effect of water interception of more than 98%, as a cumulative flux for one month, can be exhibited when the sludge barrier layer can maintain a hydraulic conductivity less than 1 x 10^{-5} cm/s.
4) The quantity of leachate generated in the waste layer after the installation of final cover system decreases to 1.7% of cases when only a surface layer is installed. Consequently, the installation of an assumed cover system can contribute to a significant reduction in the generation of leachate in the waste layer.

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CHAPTER 5

Water Interception Performance of Cover Systems under Unsaturated Conditions

5.1. General Remarks

The construction of landfill facilities to contain enormous quantities of waste is an important task for minimizing the adverse impact of leachate that generates from waste on the peripheral geo-environment. The final defense against leachate leaking out to the peripheral geo-environment is the construction of bottom liner systems in containment facilities. Cover systems overlying landfilled waste masses minimize the infiltration of rainwater into underlying waste layers and control the emission of landfill gas (Daniel and Koerner, 1995). Thus, cover systems are effective in reducing leachate generation from waste layers.

In evaluating the water interception of cover systems, it is needed to estimate the amount of rainfall infiltrating into the cover systems and the quantity percolating from the cover systems to the underlying waste layers. The United States Environmental Protection Agency (US EPA) has distributed some hydrological simulation models such as the Hydrologic Evaluation of Landfill Performance model (HELP model). Most of these models have mainly been used to evaluate the water interception of cover systems based on the saturated hydraulic conductivity of the barrier layers; but the unsaturated behavior of the constituent layers of the cover systems has not been sufficiently considered. The constituent layers of cover systems are generally constructed using cohesive soil with low hydraulic conductivity. It is usually compacted under an unsaturated condition during the construction stage. Thus, the compacted layers exhibit water interception based on the unsaturated infiltration behavior until they reach the saturated condition under specified weather conditions. Consequently, an evaluation of the water interception of cover systems, which can consider both the unsaturated infiltration characteristics of the constituent layers and the weather conditions at each construction site, is more desirable. When cover systems are constructed in Japan, where there is a monsoon climate, it is believed that the impact of rainfall on the cover systems will be much larger than that the United States and European countries. Furthermore, cover systems installed in Japan will be subjected to the intense annual change in weather throughout the four seasons.

Khire et al. (1997) have carried out research on unsaturated water transport in cover systems, and have demonstrated cover systems which can sufficiently exhibit water interception based on the unsaturated infiltration theory for aired regions. However, the effects of the water interception of cover systems in areas with heavy rainfalls, such as Japan, have never been evaluated.

In this chapter, the effects of the water interception of cover systems which apply paper sludge (PS) and construction sludge (CS) barriers are predicted using the unsaturated infiltration characteristics of the compacted PS and CS obtained experimentally, and meteorological data on each place in Japan. The soil-water characteristic curves (unsaturated infiltration characteristics) of PS and CS are examined by conducting the proposed soil-water retention tests to estimate the unsaturated infiltration parameters (e.g., unsaturated hydraulic conductivity). The data obtained from the soil-water retention tests are identified using the representative fitting models (van Genuchten and Brooks-Corey models). The applicability of the fitting models is evaluated, and the relevance of the fitting parameters to the molding water content and the dry density of the compacted sludge is evaluated. Finally, predictions of rainwater interception, based on each unsaturated infiltration characteristic of the cover systems, are performed using the UNSAT-H model which introduces the unsaturated infiltration theory into the water balance analysis (Fayer and Jones, 1990). In addition, the contents in this Chapter have been reported by Kamon et al. (2001, 2003).

5.2. Properties of Unsaturated Soil

5.2.1. Soil-Water Characteristic Curve

There is a correlation between the matric potential and the water content of soil under equilibrium conditions, and it can be expressed by plotting water content versus suction which is known as a soil-water characteristic curve. In soil science, this curve is more commonly referred to as a soil-water retention curve (Leong and Rahardjo, 1997). The term soil-water characteristic curve is used in this paper. Soil-water characteristic curves clarify the relationship between the gravimetric water content, w, the volumetric water content, θ, or the degree of saturation, S_r, and the matric suction, ϕ. Soil-water characteristic curves are highly controlled by the compositions of the soil particles and the soil pores because the matric suction is an absolute value of the potential energy arising from the curvature of the interface between the pore water and the pore air. In addition, the hysteresis is observed in the typical soil-water characteristic curve as shown in Fig. 5.1. The relationship between ϕ and θ, obtained by the drying process, is different from the relationship obtained by the wetting process. Generally speaking, the drying process provides a higher volumetric water content than the wetting process at the same level of matric suction. The hysteresis by size

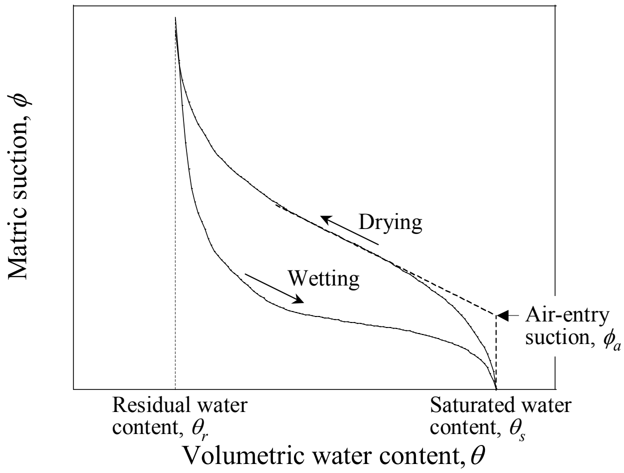

Fig. 5.1 Concept of soil-water characteristic curve

between the primary pores and the interconnecting pore throats, and is sometimes called the "ink bottle effect". Experimental measurements of the soil-water characteristic curves in the wetting process are essentially difficult to conduct, and usually only the curves in the drying process are measured (Meerdink et al., 1996). Thus, soil-water characteristic curves, only for the drying process of the compacted sludge, are discussed in this study, without giving consideration to the effects of the hysteresis. The shape of the soil-water characteristic curves depends on the soil type. Soils with a wide range of pore sizes exhibit greater changes in matric suction according to the water content (Tinjum et al., 1997).

As shown in Fig. 5.1, the air-entry suction, ϕ_a, indicates the smallest suction value necessary to remove water from the water-saturated soil pores. When microscopic soil pores exist within such soil as cohesive soil, ϕ_a provides a larger value. The water content corresponding to the asymptote of the soil-water characteristic curve is called the residual volumetric water content, θ_r, which is the amount of water that cannot be removed from the soil. θ_s is the saturated volumetric water content, and this value should be used to indicate the total porosity of the soil. Practically, however, θ_s shows approximately 90% of the total porosity, because entrapped air may remain in the soil pores when they are saturated during the wetting process.

5.2.2. Soil-Water Characteristic Models

Soil-water characteristic curve data, obtained from the soil-water retention tests, provide important unsaturated infiltration properties such as the unsaturated hydraulic conductivity. Therefore, it is necessary to identify the soil-water characteristic curve data according to the fitting models. The representative curve-fitting models are the van Genuchten model and the Brooks-Corey model. These models are usually intended for use with sand, silt, and agricultural soil (Meerdink et al., 1996). Examples of the models being applied to compacted clay are few (Tinjum et al., 1997). An application of the models is performed in this study on the soil-water characteristic curves of the compacted sludge materials.

The van Genuchten model can be expressed as Eq. (5.1):

$$Se = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \left\{ \frac{1}{1 + (\alpha\phi)^n} \right\}^m \tag{5.1}$$

where Se is the effective saturation ratio, α, n, and m are the fitting parameters, and m is frequently set at $1 - n^{-1}$. The Brooks-Corey model is described by Eqs. (5.2a) and (5.2b):

$$Se = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \left\{ \frac{\phi_a}{\phi} \right\}^\lambda \qquad \phi \geq \phi_a \tag{5.2a}$$

$$Se = 1, \quad \theta = \theta_s \qquad \phi < \phi_a \tag{5.2b}$$

where λ is a fitting parameter that is called the pore-size-distribution index, which is a function of the distribution of pores in the soil. The fitting parameters define the shape of the soil-water characteristic curves for both models. Soils with steeper soil-water characteristic curves are characterized by smaller λ or n. A higher level of air-entry suction is characterized by the greater ϕ_a seen in Eq. (5.2) or the smaller α seen in Eq. (5.1).

5.2.3. Estimation of Unsaturated Hydraulic Conductivity

Unsaturated hydraulic conductivity indicates the resistance during water migration through unsaturated soil. Hence, the unsaturated hydraulic conductivity depends on the viscosity of the water, the soil-pore structure, and the water content of the soil. The unsaturated hydraulic conductivity is generally expressed as a function of the water content of the soil, and it significantly decreases from an approximately saturated condition to the air-dried state. In addition, the unsaturated hydraulic conductivity can also be displayed as a function of the matric potential, because the volumetric water content can be functionally determined from the matric potential, as shown in the soil-water characteristic models. The function of the unsaturated hydraulic conductivity is frequently estimated using semi-empirical models, most of which are based on the saturated hydraulic conductivity, k_s, and the fitting parameters which describe the shape of the soil-water characteristic curves. van Genuchten (1980) indicated that the relative hydraulic conductivity function, $Kr = K_\phi / k_s$, can be estimated using the relationship between the matric potential and the unsaturated hydraulic conductivity proposed theoretically by Mualem (1978), if the soil-water characteristic curves can be identified by the van Genuchten model expressed by Eq. (5.1). This van Genuchten-Mualem model, which estimates the unsaturated hydraulic conductivity function from the fitting parameters, is shown in the following Eq. (5.3) (Meerdink et al., 1996):

$$Kr(\phi) = \frac{\left\{ 1-(\alpha\phi)^{n-1}\left[1+(\alpha\phi)^{n} \right]^{-m} \right\}^{2}}{\left[1+(\alpha\phi)^{n} \right]^{m/2}} \tag{5.3}$$

Brooks and Corey have also suggested that the relative hydraulic conductivity function could be empirically estimated by Eq. (5.4) (Corey, 1994):

$$Kr(\phi) = \left(\frac{\phi_a}{\phi} \right) \qquad \phi > \phi_a \tag{5.4a}$$

$$Kr(\phi) = 1 \qquad \phi \le \phi_a \tag{5.4b}$$

5.3. Experimental Approaches to Soil-Water Characteristic of Sludge

5.3.1. Materials Used

The sludge materials used in this study are paper sludge (PS), generated from the dehydration process of paper factory effluent treatment facilities using a belt press, and construction sludge (CS), generated from shield tunnel construction sites. The basic properties of these two types of sludge are shown in Table 3.3.

5.3.2. Soil-Water Retention Test

Two different methods, referred to as Procedure-A and Procedure-B, were applied for conducting soil-water retention tests. Both methods are categorized as pressure-plate methods, which are considered to be the most suitable for clayey soils. A schematic diagram of the soil-water retention test equipment used for the Procedure-A is shown in Fig. 5.2. The equipment was originally developed by Tinjum et al. (1997), and realizes perfect contact by two o-rings between the ceramic disk and the suction ring to maintain the hydraulic contact between the ceramic disk and the soil specimen. The equipment has been modified for use inside the triaxial compression test apparatus in this study. The suction ring has a diameter of 4 cm and a height of 2 cm. Procedure-A was conducted as follows. (1) The PS and CS

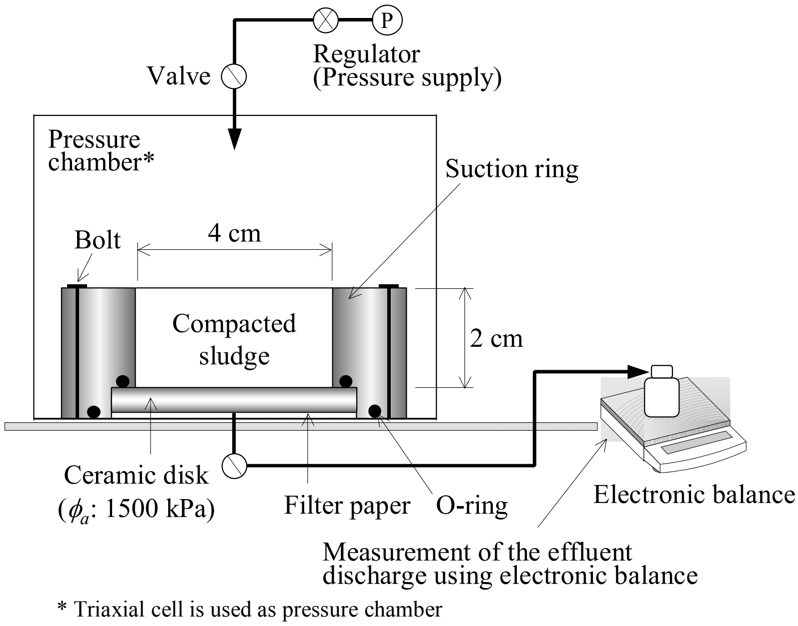

Fig. 5.2 Schematic diagram of equipment for the soil-water retention tests (Procedure-A)

with different levels of molding water content were compacted directly in the suction rings, respectively, to achieve the same compaction conditions as their compaction curves obtained from the test method for soil compaction using a rammer (JGS 0711) at each molding water content; (2) The compaction using a wooden hammer with a weight of 0.12 kg, was carried out so that the divided three layers have the equal density each other; (3) Porous stones were placed at the top and the bottom of the specimen in the suction ring, and the degree of saturation of the specimen was increased by a vacuum; (4) The mass of the specimen plus the suction ring was periodically measured until there were no more increases in the mass, then, the saturation step was terminated (the degree of saturation can be determined with the final mass of the specimen); (5) The two porous stones were removed, the ceramic disk with an air-entry pressure of 1500 kPa was placed on the bottom of the specimen, and the suction ring was fixed with bolts in the pressure chamber; (6) Various levels of pressure were applied in steps to the pressure chamber; (7) The water expelled from the soil specimen through the ceramic disk was collected in a graduated cylinder. The graduated cylinder was placed on an electric balance with a sensitivity of 0.01 g to periodically measure the mass of the expelled water. Each pressure level was maintained until there were no more increases in mass of the expelled water (thus, the degree of saturation for each pressure can be calculated using the mass of the expelled water); and (8) After the final pressure level, the total mass of the soil specimen was directly measured. In addition, the mass of the oven-dried specimen was measured to obtain the final water content, which can be checked with the amount of expelled water.

For Procedure-B, four to five compacted and saturated specimens with suction rings were placed on a ceramic disk with an air-entry pressure of 1500 kPa in a suction chamber. Various levels of pressure were applied until an equilibrium for the expelled water was achieved. For each pressure level, one specimen was removed to directly measure the water content. Thus, the same number of specimens is required as the number of pressure stages. Another difference of Procedure B from Procedure-A is that the suction ring is not fixed on the ceramic disk and perfect contact between the soil specimen and the ceramic disk is not necessarily assumed. However, one advantage of Procedure-B is that as many as twelve specimens under specified pressure levels can be measured simultaneously.

5.4. Results and Discussions

5.4.1. Results of Soil-Water Retention Test

Soil-water retention tests using the above-mentioned pressure plate methods (Procedures-A and -B) are carried out on paper sludge (PS) and construction sludge (CS) with several levels of initial water content. Both materials are compacted in the suction ring to achieve the same conditions of water content and dry density as those in the standard compaction curves. Figure 5.3 shows the soil-water characteristic curves for the compacted PS and CS obtained by the soil-water retention tests, in comparing with that for Toyoura sand from Committee on Permeability Assessment of Unsaturated Ground (1997). The degree of saturation for each specimen tends to decrease gradually with an increase in the matric suction. The data for each initial water content from Procedure-A provide smoother curves for the degree of saturation and the matric suction than from Procedure-B. This is because, Procedure-A requires one specimen with an initial water content for continuously obtaining the data of the degree of saturation and the matric suction. However, only one plot for the degree of saturation and the matric suction is obtained from one specimen in Procedure-B. It is difficult to provide exactly the same conditions for each specimen. However, there is no

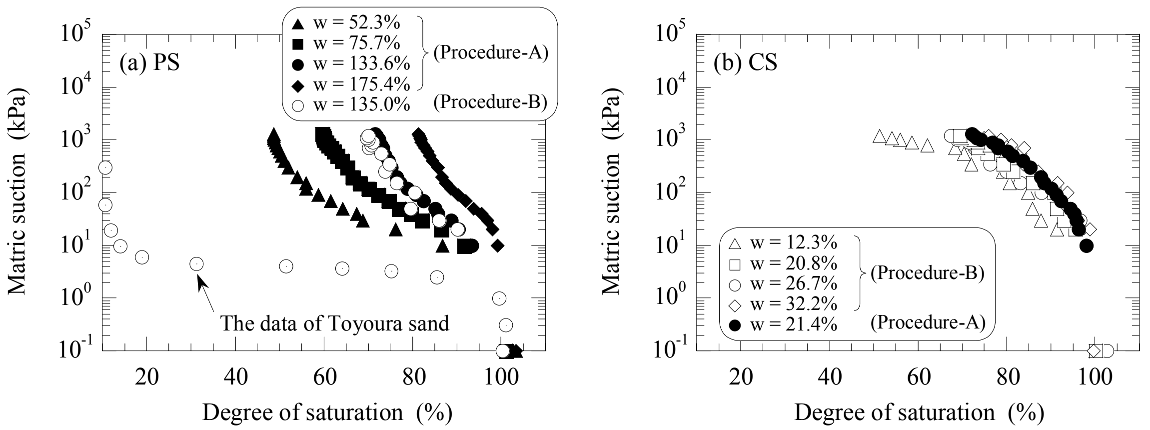

Fig. 5.3 Soil-water characteristic curves for compacted (a) PS and (b) CS

great difference between Procedure-A and -B in regard to trends of the data obtained.

For all of the compacted PS specimens, an asymptotic tendency is observed at matric suction levels higher than 200 kPa. Thus, the residual volumetric water content can be obtained using the fitting equations. In addition, the residual volumetric water content and the matric suction which provides the residual volumetric water content for the compacted PS is larger than that for typical sandy soil such as the Toyoura sand. This fact is thought to clarify that the pore water in the organic substances contained in the PS can be retained even at a high level of matric suction. Consequently, most of the water existing in the pores of the organic substances of PS can be regarded as strongly retained adsorption water. Unlike the compacted PS, the asymptotic tendency is not observed for a high range of matric suction, and the residual volumetric water content cannot be obtained from this experiment for the compacted CS.

The shape of the obtained soil-water characteristic curves is considered to depend on the pore properties of the materials. The compacted PS contains a large amount of water, and the water retained in the compacted PS exists in inter-particle pores or in pores of the organic substances. Pores in organic substances are generally very small, and the pore water in organic substances exhibits higher viscosity than inter-particle pore water (Adams, 1963; De Jong, 1968; Fredlund and Rahardjo, 1993). Thus, the effect of pore water in organic substances on the soil-water characteristic curves should be considered, especially in the case of the compacted PS. In contrast, the soil-pore structures in the compacted CS specimen are categorized into the pores formed in the inter-particles and those formed in the intra-particles, such as those in organic substances. As the CS particles are practically aggregated during water adjustment and compaction, the CS particles form clod structures, and the soil pores of the compacted CS exist in inter-clods or intra-clods. This is discussed in detail in a later section using the fitting equations.

5.4.2. Fitting for Soil-Water Characteristic Curves

The fitting for the data on the soil-water characteristic curves obtained from the experiments is conducted using the van Genuchten model (Eq. (5.1)) and the Brooks-Corey model (Eq. (5.2)). The inverse analysis, which is an indirect method recommended by Daniel and Wood (1971), is applied as a fitting technique (Hollenbeck et al., 2000; Miller et al., 2000). The technique calculates parameter vector b, which minimizes the residual sum of squares $O(b)$, defined in Eq. (5.5). Thus, the non-linear least-squares method is applied for Eq. (5.5) as the objective function:

$$O(b) = \sum_{i=1}^{N} \left\{ w_i \left[\theta_i - \theta_{ci}(b) \right] \right\}^2 \tag{5.5}$$

where N is the number of data points, θ_i is the observed water content, $\theta_{ci}(b)$ is the fitted water content, and w_i refers to as the weighting coefficients. An equivalent error is assumed for each measured value, and weighting coefficient w_i is assumed as 1. For soils which have too high level of matric suction for the residual volumetric water content to measure, Tinjum et al. (1997) recommended that the residual volumetric water content θ_r should be assumed as 0 to improve the fitting accuracy. In the present study, this assumption is applied to the curve fitting for the compacted CS, because the soil-water characteristic data of the compacted CS for matric suction levels higher than 1500 kPa cannot be obtained.

Typical fitting results are shown in Fig. 5.4, and the obtained parameters are summarized in Table 5.1. The data fitted by the van Genuchten model can exhibit a slightly better correlation than the data fitted by the Brooks-Corey model, which is indicated by the r-square, r^2, in Table 5.1. The equation form of the van Genuchten model, as shown in Eq. (5.1), gives more flexibility than that of the Brooks-Corey model, as shown in Eq. (5.2). The correspondence between the van Genuchten and the Brooks-Corey fitting parameters is

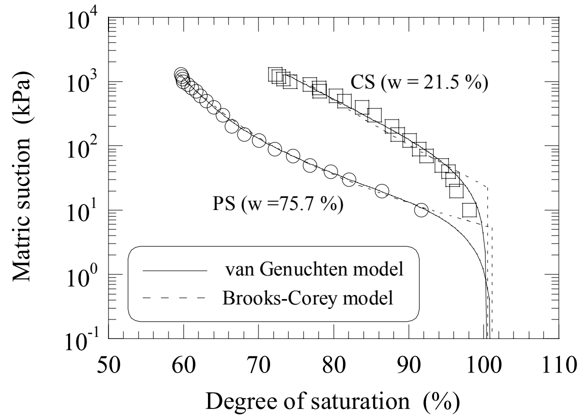

Fig. 5.4 Representative examples of the fitting results for compacted PS and CS

Table 5.1 Fitting results of applying the van Genuchten and the Brooks-Corey models

Soil	w	ρd	k_s*	θ_s	van Genuchten model				Brooks-Corey model				Test procedur
					θ_r	n	α	r^2	θ_r	λ	ϕ_a	r^2	
	(%)	(g/cm^3)	(cm/s)	(cm^3/cm^3)	(cm^3/cm^3)	(-)	(kPa^{-1})	(-)	(cm^3/cm^3)	(-)	(kPa)	(-)	
PS	52.3	0.530	7.8×10^{-7}	0.727	0.323	1.610	0.130	0.998	0.310	0.480	0.512	0.998	A
PS	75.7	0.599	9.0×10^{-7}	0.673	0.353	1.409	0.103	0.998	0.305	0.022	5.274	0.994	A
PS	133.6	0.526	3.5×10^{-7}	0.732	0.476	1.378	0.115	0.990	0.432	0.231	4.558	0.984	A
PS	135.0	0.522	3.5×10^{-7}	0.711	0.459	1.394	0.111	0.964	0.452	0.347	6.963	0.968	B
PS	175.4	0.501	3.8×10^{-7}	0.785	0.570	1.350	0.058	0.992	0.359	0.096	6.143	0.978	A
CS	12.3	1.361	6.5×10^{-8}	0.492	0.000	1.172	0.021	0.859	0.000	0.116	13.770	0.792	B
CS	20.8	1.577	1.6×10^{-8}	0.423	0.000	1.107	0.036	0.976	0.000	0.088	14.026	0.964	B
CS	21.5	1.568	1.6×10^{-8}	0.417	0.000	1.097	0.019	0.972	0.000	0.074	22.332	0.918	A
CS	26.7	1.506	1.2×10^{-8}	0.439	0.000	1.097	0.029	0.966	0.000	0.075	14.106	0.933	B
CS	32.2	1.391	4.0×10^{-8}	0.479	0.000	1.093	0.013	0.937	0.000	0.069	31.935	0.887	B
CS	37.1	1.317	6.0×10^{-8}	0.507	0.000	1.057	0.042	0.964	0.000	0.047	13.037	0.943	B

* Saturated hydraulic conductivity, k_s, obtained by flexible-wall hydraulic conductivity tests.

shown in Fig. 5.5. Parameter α by the van Genuchten model and ϕ_a by the Brooks-Corey model are related to the level of air-entry suction. Parameters n and λ, which are obtained by the van Genuchten and the Brooks-Corey models, respectively, reflect the degree of steeper slope of curve for the degree of saturation and the matric suction. The Parameter α generally decreases with an increase in the ϕ_a for both types of the compacted sludge. This inverse relationship is more apparent in the case of the compacted CS than in the case of the compacted PS (*see* Fig. 5.5(a)). The comparatively low correlation between α and ϕ_a for the compacted PS is probably because the lowest level of matric suction applied in the experiment is still higher than the true value of air-entry suction of the compacted PS. The lowest level of matric suction, 10 kPa, is applied in this experiment. In contrast, the correlation between n and λ for the compacted CS is stronger than that for the compacted PS, as shown in Fig. 5.5(b). This means that the slope of the soil-water characteristic curves using n and λ is better determined for the compacted CS than for the compacted PS. This is because the compacted CS has a higher level of air-entry suction compared with the lowest level of suction applied in these experiments (10 kPa), unlike the compacted PS. As a result, a high accuracy in the estimation of ϕ_a and α is obtained.

Since the van Genuchten model provides fitting with a higher accuracy than the Brooks-Corey model, which is clear in the r-square, r^2, in Table 5.1, a later section is discussed using the parameters regulated in the van Genuchten model.

5.4.3. Effects of Molding Water Content and Dry Density on the Soil-Water Characteristic Curves

The compacted PS and CS exhibit the different shapes of soil-water characteristic curves according to their levels of molding water content, respectively, as shown in Fig. 5.3. The relationship between the molding water content and the van Genuchten parameters such as

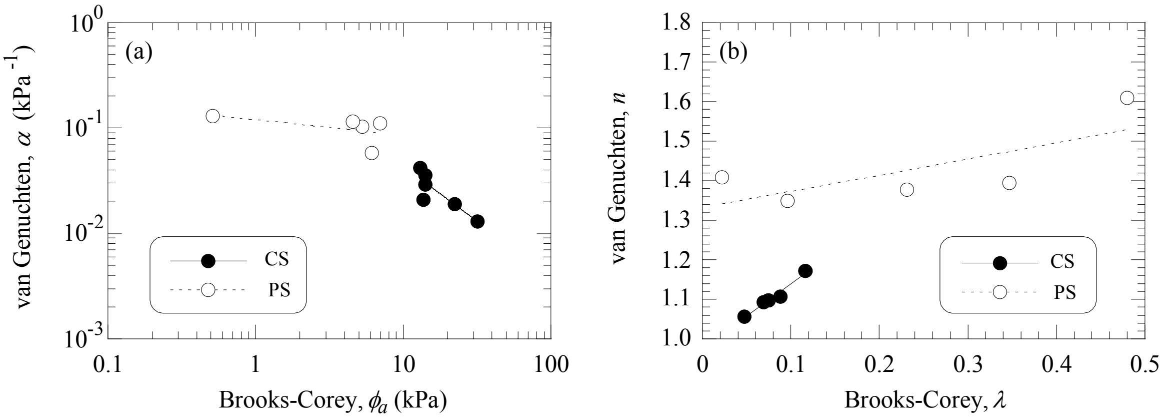

Fig. 5.5 Relationship between van Genuchten and Brooks-Corey parameters represented by (a) α versus ϕa and (b) n versus λ

α and n for the compacted PS and CS is shown in Fig. 5.6. Both α and n for the compacted PS decrease with an increase in the molding water content, as shown in Fig. 5.6(a). A decrease in α indicates an increase in the air-entry suction, and a decrease in n reflects the steeper slope of soil-water characteristic curve. Thus, the compacted PS at a higher molding water content retains more soil-water. The pore formations of the compacted PS are divided into inter-particle pores and pores in the organic substances. In the compacted PS with a higher molding water content, the microscopic pores of the organic substances may possess a larger amount of water than the inter-particle pores. The PS compacted at a higher molding water content indicates a higher level of air-entry suction and water retentiveness than the PS compacted at a lower molding water content even at a higher level of suction.

Although the parameters n and α of the compacted CS decrease with an increase in the molding water content (*see* Fig. 5.6(b)) and this tendency is the same as that for the compacted PS, the state of the soil-water in the pores is considered to be not the same as that for the compacted PS. The molding water content of CS affects the shapes of their soil-water characteristic curves because it affects the micro- and macro-scopic pore formations of the compacted CS. The CS prepared at a low water content contains clods of soil particles developing during the soil-water processing, and these clods are maintained during the compaction effort of CS. Such phenomenon is observed through the experiment. Thus, macroscopic pores as well as microscopic pores are formed in the inter-clods of the compacted CS with a low water content. In contrast, the CS with a high water content can be compacted without the formation of macroscopic pores in it, and only microscopic pores exist in the compacted CS. Benson and Daniel (1990) discussed the influence of the clods formed during soil processing and compaction with the standard Proctor effort on the soil-pore structures, and concluded that the soils compacted under drier than optimum conditions have large clods and visible inter-clod pores in them, while the soils compacted under wetter than optimum conditions have homogeneous small clods in them. These

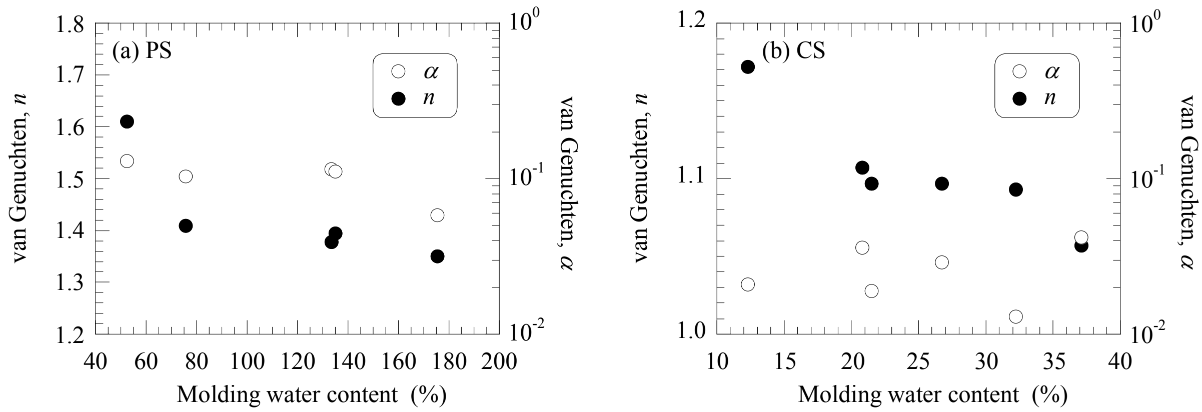

Fig. 5.6 Water content in compaction versus van Genuchten parameters for compacted (a) PS and (b) CS

results seem to be similar in the compaction property of CS observed in this study. Due to changes in the soil-pore structure depending on the molding water content of the compacted CS, the parameters n and α are considered to be decreased with an increase in the molding water content of the compacted CS.

Figure 5.7 shows the relationship between the van Genuchten parameter, n, and the dry density of the compacted PS and CS. It is obviously that the dry density of the compacted PS and CS is a factor which governs fitting parameters such as n and α. But besides dry density, the molding water content also influences the n. This can be found from the fact that the sludge compacted at a higher molding water content provides smaller n than that compacted at a lower molding water content even the both compacted sludge have the same level of dry density (*see* Fig. 5.7). The dry density governs the total amount of soil pores, while molding water content controls the pore-size distribution. The water retentiveness of the compacted PS and CS is controlled greatly by the pore-size distribution than by the amount of soil pores. Tinjum et al. (1997) reported experimentally the effect of pore size distribution in the different types of clay compacted with the standard Proctor effort on the shape of soil-water characteristic curve. They concluded that the clay specimens compacted to the same dry density under the compaction water contents with dry and wet of the optimum water content have radically different pore-size distributions. Therefore, significantly different α and n should be obtained for the same dry density, depending on the compaction water content. They also explained the effect of pore size distribution on the shape of soil-water characteristic curve as following: The compacted clays with dry of optimum water content contain stiff clods that are not remolded during the standard Proctor effort. As the result, the large macroscopic pores exist, which result in lower air-entry suction. The compacted clays with wet of optimum water content, however, are devoid of macroscopic pores, which result in a higher air-entry suction. Furthermore, Tinjum et al. (1997) succeeded in observation of the microscopic pore distributions of the

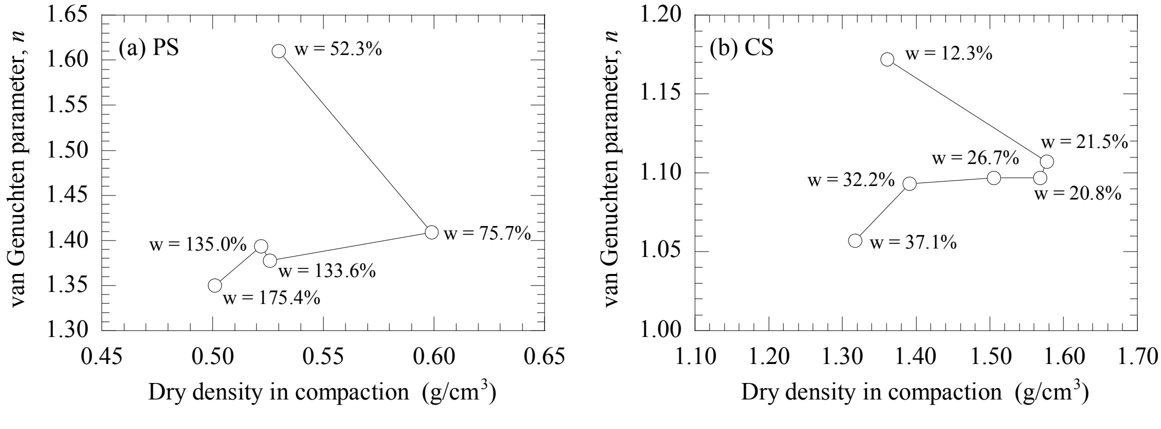

Fig. 5.7 Dry density in compaction versus van Genuchten parameter n for compacted (a) PS and (b) CS

clay specimens by photograph, and they showed that the steeper slopes of the soil-water characteristic curve of the compacted clays with the wet of optimum water content suggest the broader distribution of microscopic pores.

The PS and CS specimens used for the soil-water retention test are prepared according to the conditions of their standard compaction curves. Under such compaction conditions of specimen, the shape of soil-water characteristic curve depends on increasing in the molding water content of them. However, if the standard compaction effort is not achieved due to the field conditions, it is expected that the PS and CS compacted with lower dry density exhibit larger α and n than that of PS and CS compacted with the standard compaction effort, under the same level of molding water content. This is because, as Benson and Daniel (1990) reported, that the parameters n and α of the clay compacted using different compaction energy under the same level of compaction water content are increased with decrease in the compaction energy. This proves that the water retentiveness of the clay is dependent on the dry density in compaction, if the clay has the same molding water content. In this experiment, the relationships between different levels of molding water content and the shapes of soil-water characteristic curve are clarified on the PS and CS compacted according to their standard compaction curves. It should be noted that these relationships are discussed is based on the conditions of PS and CS in which the compaction is performed in accordance with the standard compaction effort. The effects of the dry density in compaction on the water retentiveness of the PS and CS compacted at different compaction efforts should be investigated in future.

5.4.4. Unsaturated Hydraulic Conductivity of the Compacted Sludge

The soil-water retention properties of compacted clay material for landfill barriers affect the unsaturated hydraulic conductivity of the compacted clay (Fredlund and Rahardjo, 1993; Meerdink et al., 1996). Hence, the unsaturated hydraulic conductivities of the two types of the compacted sludge are estimated by Eqs. (5.3) or (5.4) using saturated hydraulic conductivity k_s, measured with flexible-wall permeameters, and the soil-water characteristic curves, measured by the pressure plate extractor. The relationship between the unsaturated hydraulic conductivity and the degree of saturation for both types of the compacted sludge is shown in Fig. 5.8. For both types of the compacted sludge, the unsaturated hydraulic conductivity significantly decreases in orders of magnitude with a decrease in the degree of saturation from the fully saturated condition to the drying state. Such a significant decrease in the unsaturated hydraulic conductivity for variations in the water content is firstly due to the removed water from the largest soil pores. According to the law of Hagen-Poiseuille, fluid flux f_w flowing in a tube with a radius r and a length Δx is expressed as follows:

$$f_w = \left(r^2 / 8\upsilon \right)\left(\Delta P / \Delta x \right) \tag{5.6}$$

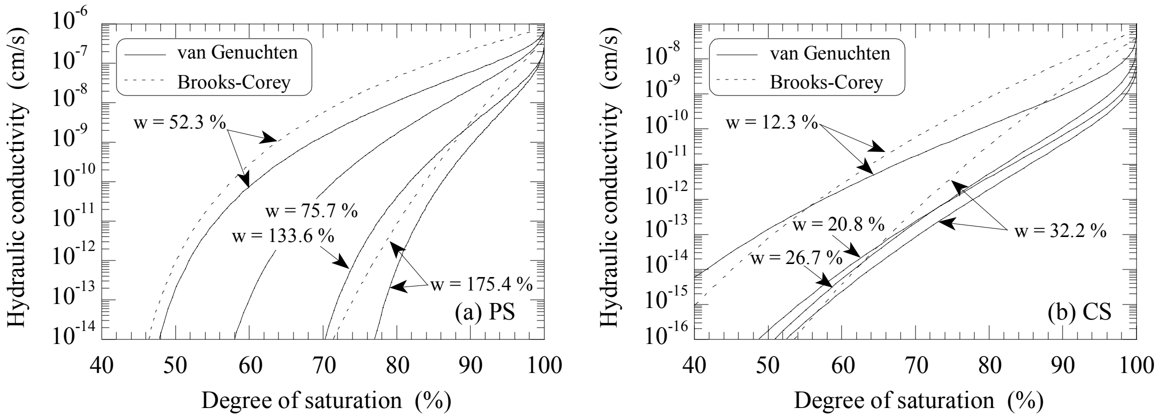

Fig. 5.8 Estimation of the unsaturated hydraulic conductivity of compacted (a) PS and (b) CS

where ν is the coefficient of the kinematic viscosity (m^2/s) and Δp is the pressure difference across Δx. Equation (5.6) implies that the passage of water through the smaller soil pores is more difficult than that through the larger pores. If the degree of saturation decreases, the refraction of the flow increases, and the waterway is lengthened. The unsaturated hydraulic conductivity of the compacted PS and CS, adjusted at higher levels of water content, is significantly decreased by a decrease in the degree of saturation, compared with that at lower levels of water content. Since the sludge materials with higher levels of water content can be compacted without the formation of clod structures, as discussed in the preceding section, the inside of the compacted specimen is composed of microscopic soil pores rather than macroscopic soil pores. Thus, the unsaturated hydraulic conductivity, which is greatly influenced by the distribution of soil pore size, decreases with an increase in the molding water content. Consequently, the compacted sludge at higher levels of molding water content has significant water retentiveness, which contributes to a significant decrease in the unsaturated hydraulic conductivity with a decrease in the degree of saturation.

However, there is a difference between the van Genuchten and the Brooks-Corey models in terms of estimating the unsaturated hydraulic conductivity, as shown in Fig. 5.8. The accuracy of estimating the unsaturated hydraulic conductivity from fitting parameters for the soil-water characteristic curves depends on several factors, in particular, the following two major ones. Firstly, the data available for defining the shape of the soil-water characteristic curves at lower levels of water content (at higher levels of matric suction) are insufficient. This may affect the estimation of θ_r and the gradient of each curve, which will in turn affect the predicted unsaturated hydraulic conductivity. Secondly, the van Genuchten-Mualem model is based on the statistical models of flow through a set of capillary tubes (Mualem, 1978), described by the law of Hagen-Poiseuille, which are unlikely to capture the complexities of the unsaturated flows through compacted clay where the flows occur through the inter-clod pores and the intra-clod pores. In addition, the mineral surface forces increasingly affect the flows through those pores as they desaturate (Meerdink et al., 1996).

5.5. Water Interception Performance of Landfill Cover Systems

5.5.1. Simulation Model

The most important function of landfill cover systems is to prevent water from percolating into waste layers. The amount of water percolating from cover systems depends on the hydrological conditions of the construction site, the components of the cover systems, the surface gradients of the landfill site, and the existence of vegetation. Some hydrological water balance simulation models are used to predict the amount of rainfall and snowfall percolating from the cover systems. The Hydrologic Evaluation of Landfill Performance (HELP) model is one of these representative models. The HELP model is based on a quasi-two-dimensional analysis, and the constituent layers of the cover systems are categorized into the vertical infiltration layer, the lateral drainage layer, and the barrier layer. The water which has percolated from the vertical infiltration layer is calculated using the given soil water retentiveness of the vertical infiltration layer. The percolated water from the lateral drainage and the barrier layers is then calculated applying Darcy's law to the drainage and the vertical directions, respectively (Schroeder et al., 1994). Thus, the drainage and the barrier layers are assumed to be in the saturated condition in the HELP model. The HELP model tends to overestimate the quantity of percolated water in the pluvial region in comparison to the value observed in the field (Khire et al., 1997), because the condition of the barrier layer is only assumed to be in a saturated condition rather than in an unsaturated condition.

Recently, an unsaturated water balance model has been developed as an alternative evaluation technique for the pluvial region. For example, the water interception performance of cover systems has been evaluated with the Unsaturated Soil Water and Heat Flow (UNSAT-H) model which considers the unsaturated condition of cover systems. In this paper, the UNSAT-H model is used to predict the water interception performance of cover systems using the compacted PS or CS as the barrier material, because cover systems are subjected to unsaturated condition which are affected by the weather and the compaction conditions.

The UNSAT-H model developed by Fayer and Jones (1990), solves the modified Richard's equation using a one-dimensional finite difference approximation and predicts the water seepage through cover systems using the governing equation as expressed in Eq. (5.7) (Khire et al., 1997, 2000):

$$\frac{\partial\theta}{\partial\phi}\frac{\partial\phi}{\partial t}=\frac{-\partial}{\partial z}\left\{\left(K_\phi+K_{v\phi}\right)\frac{\partial\varphi}{\partial z}+K_\phi+q_{vt}\right\}-S(z,t) \tag{5.7}$$

where ϕ is the matric potential head (negative) due to the capillary suction forces, t is the time, z is the vertical coordinate (positive downward), θ is the soil volumetric water content, K_ϕ is the unsaturated hydraulic conductivity, $K_{v\phi}$ is the isothermal vapor conductivity, q_{vt} is the thermal vapor flux density obtained by applying Fick's law to the vapor flux, and $S(z, t)$ is the sink term representing the water uptake by the vegetation, although this factor is not considered in the present study.

Figure 5.9 shows a simulation outline for the UNSAT-H model and the initial boundary conditions used in the analysis (Khire et al., 1995). The initial conditions for UNSAT-H are specified by assigning the initial head to each node in the finite difference nodal grid. The matric suction corresponding to the water content of each compacted sludge material is used to determine the initial head in this analysis. The simulation is conducted using the lower boundary as the unit gradient and the upper boundary as the specified infiltration or the evaporation flux boundary. Khire et al. (1997) used similar boundary conditions for analyzing the water balance at various existing landfill sites. Fayer et al. (1992) also simulated the hydrology of drainage lysimeters constructed at the Hanford site by applying boundary conditions similar to those of the UNSAT-H model. UNSAT-H separates the precipitated rainfall on a landfill cover into infiltration and runoff. Runoff occurs when water applied to the soil surface exceeds the infiltration capacity of the cover systems during a rainfall. Thus, the runoff of rainfall depends on the saturated and the unsaturated hydraulic conductivities of each constituent soil layer. Water that infiltrates into

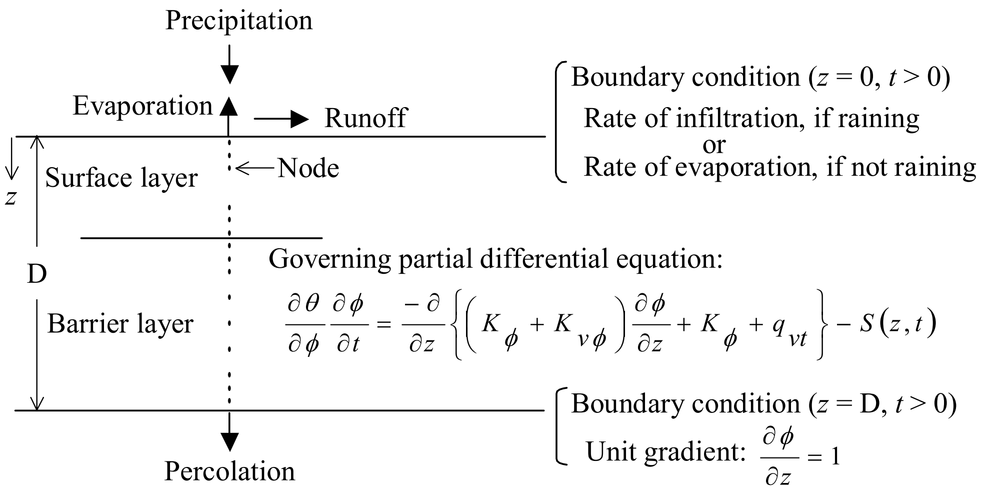

Fig. 5.9 Simulation outline for the UNSAT-H model and the initial boundary conditions used in the analysis (modified from Khire et al., 1995)

the cover systems moves upward due to evaporation or downward as a consequence of gravity and the matric potential. When the upper boundary is set as a flux boundary, infiltration and evaporation from the surface are the specified flux. Evaporation is computed using Fick's law. Water removed by the transpiration of plants is treated as a sink term in Eq. (5.7). The potential evapotranspiration (the upper limit on actual evapotranspiration) is computed from the daily relative humidity, the net solar radiation, the wind speed, and the daily minimum and maximum air temperatures using a modified form of Penman's equation given by Doorenbos and Pruitt (1997). The soil water storage is computed by the infiltrated water content profile. Flux from the lower boundary (the bottom of the cover system) is defined as percolation. UNSAT-H does not consider the adsorption or the interception of water by a plant canopy or the effect of the slope and the slope length.

5.5.2. Soil Properties under the Unsaturated condition

A water balance analysis using the UNSAT-H model requires the unsaturated infiltration characteristics of each constituent layer of the cover systems. Infiltration parameters such as the θ - ϕ relation, the unsaturated hydraulic conductivity function, and the saturated hydraulic conductivity are necessary for predicting the water interception of the cover systems. In this analysis, the fitting results by the van Genuchten model (Eq. (5.1)), as shown in Table 5.1, for the soil-water characteristic curves are used for the θ- ϕ relations of the compacted PS and CS as barrier materials. This is because the van Genuchten model resulted in a better fitting than the Brooks-Corey model. The unsaturated hydraulic conductivity has been estimated using the van Genuchten-Muallem model (Eq. (5.3)). These unsaturated infiltration parameters of the compacted sludge are obtained in the drying process, and the effects of the hysteresis are not considered in this analysis. Since the soil undergoing drying has a higher volumetric water content, it also has a larger cross-sectional area for flow, less tortuous flow paths, and consequently, a higher level of unsaturated hydraulic conductivity at the same suction level (Meerdink et al., 1996). Therefore, the drying process can provide more serious evaluation results for the analysis than the wetting process.

5.5.3. Cross Section of the Simulated Cover Systems and the Meteorological Data Input

The assumed structural profile for the analysis is shown in Table 5.2. A cover system composed of a surface layer of silty loam and a barrier layer of the compacted sludge is assumed. The unsaturated infiltration parameters of the silty loam are in accordance with the results by Fayer (2000). The parameters used in the water balance calculation are also shown in Table 5.2. The effects of the hydrological weather conditions and the unsaturated infiltration characteristics of the barrier layer with the compacted sludge on the water

Table 5.2 Assumed values for each composed layer in the cover system

	Surface layer	Barrier layer
Soil	Silty loam	Compacted sludge
Thickness, D (cm)	10	10, 20, 30, 40, and 50 cm
Infiltration parameters		
θ_s (cm^3/cm^3)	0.388	*see* Table 5.1
θ_r (cm^3/cm^3)	0.173	
n	0.047	
α (1/cm)	1.461	
k_s (cm/s)	1.5×10^{-3}	

interception performance are quantitatively evaluated.

The meteorological input for UNSAT-H includes the daily precipitation, the daily maximum and minimum air temperatures, the daily solar radiation, the average daily dew point, and the average daily wind speed. Weather databases in 1998, 1999, and 2000 obtained from the Japan Weather Association are used as input data.

5.6. Performance of the Cover Systems

5.6.1. Effects of Weather Condition on Water Interception

The water interception performance of the cover systems is greatly affected not only by the (unsaturated) infiltration characteristics of each constituent layer, but also by the weather conditions of the sites where the landfills are constructed. Hence, the water balance under various climatic environments is estimated for a cover system in which the compacted PS with a compaction water content of 133.6 % and a layer thickness of 30 cm is assumed to be used for the barrier layer. Figure 5.10 shows the results of the water balance analysis for the cover system subjected to the weather conditions of Owase City, Mie Prefecture, Japan, from January of 1998 to December of 2000. Owase City has the most precipitation in Japan. If the cover system is established in Owase, approximately 90% of the rainfall would be removed from the cover system as surface runoff or evaporation, and 10% would be infiltrated into the cover system. If the rainfall flux exceeds the infiltration capacity defined by the saturated and the unsaturated hydraulic conductivity of the cover system, the accumulated water quantity on the surface would be removed as surface runoff. Although the quantity of evaporated water, which depends on air temperature, solar radiation, and wind speed, tends to be actively generated in the rainy season, the absolute value of the quantity of water evaporated from the surface of the cover system is less than 10% of the rainfall. Consequently, the most of rainfall that is removed is caused by effects of the surface runoff. A quantity of water percolated from the cover system can be especially observed from May to September of each year, and during which there is comparatively heavy rainfall.

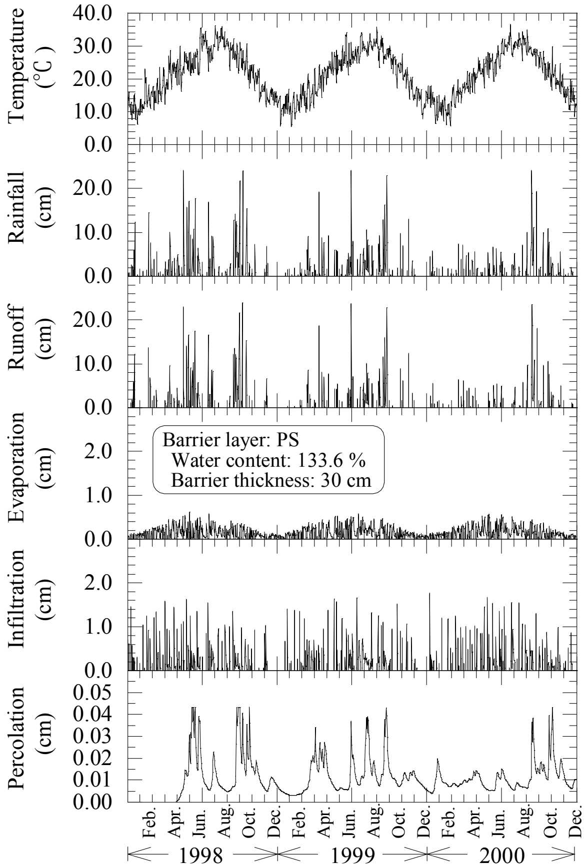

Fig. 5.10 Input data and results of the water balance analysis for a cover system with a PS barrier (water content of 133.6%, thickness of 30 cm) in Owase, Japan

Figure 5.11 indicates the results of a water balance analysis represented by the cumulative flux in Owase, Kyoto, and Tokyo, Japan. The quantity of surface runoff in Owase (*see* Fig. 5.11(a)) is more dominant than that in Kyoto (*see* Fig. 5.11(b)) or in Tokyo (*see* Fig. 5.11(c)). This is because the rainfall flux in Owase frequently surpasses the infiltration capacity of the cover system. The quantity of cumulative water percolated from the cover system is less than 15 cm, equaling approximately 1% of the total rainfall throughout the three-year period used for the analysis, even though the time at which the percolation begins depends on the weather conditions at each site. More than 99% of the rainfall is intercepted by the installation of the cover system in Owase, Kyoto, and Tokyo throughout the three-year period used for the analysis.

5.6.2. Effects of the Barrier Layer on Water Interception

The effects of the compaction properties of PS and CS as the barrier layer on the performance of cover system are evaluated for that constructed in Owase. The PS and CS as the barrier layer in cover system are assumed to be compacted at molding water contents of

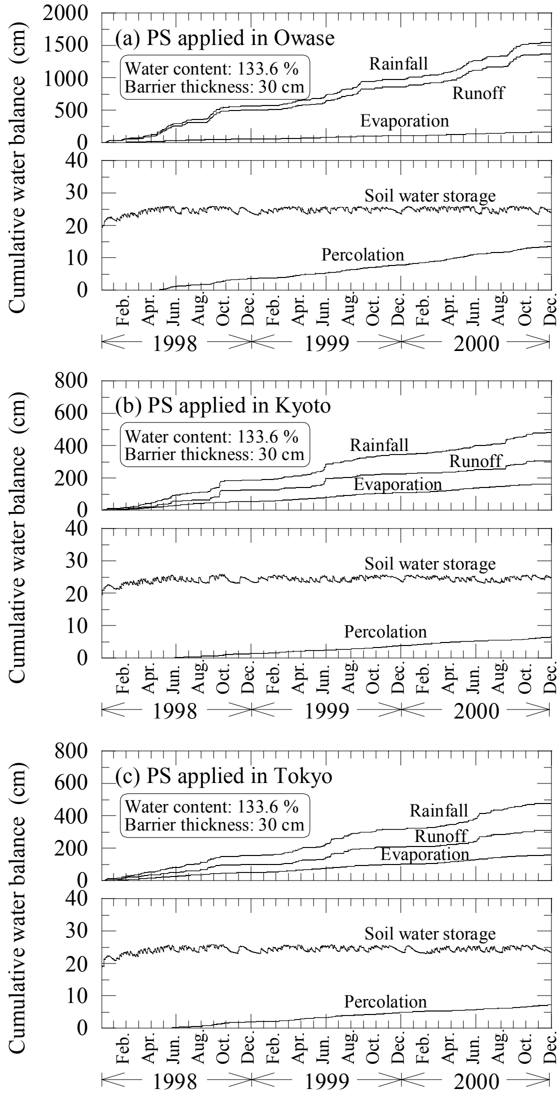

Fig. 5.11 Influence of weather conditions at each site on the results of the water balance analysis for cover systems with a PS barrier (water content of 133.6%, thickness of 30 cm)

133.6% and 21.5%, respectively, which provide the lowest saturated hydraulic conductivities of the compacted PS and CS. Figure 5.12(a) indicates the quantity of cumulative water percolated from the cover system. The quantity of water percolating from the barrier layer with the compacted PS increases during the period of relatively heavy rainfall (for example, from May to July of 1998). Namely, the compacted PS under unsaturated condition reaches the saturated condition during the period of heavy rainfall immediately after the installation. In addition, it can be assumed that the infiltration characteristics of the compacted PS, after the saturated condition is once reached, depend mainly on the saturated hydraulic conductivity. This is because the barrier layer of the compacted PS, which reached the saturated condition, cannot recover to the unsaturated

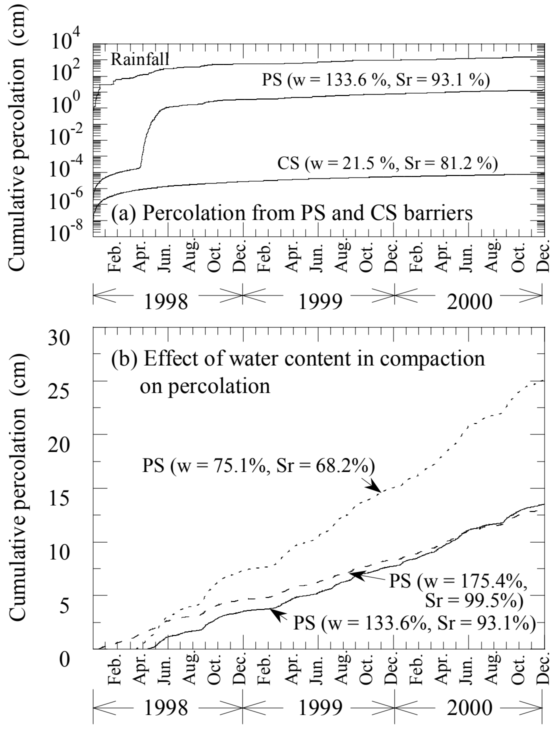

Fig. 5.12 Effects of sludge type and molding water content of the PS barrier on the quantity of percolation water

condition due to the weather conditions of heavy rainfall and the components of the cover system. The quantity of water percolated from the barrier layer of the compacted PS with different compaction water contents is shown in Fig. 5.12(b). The degree of saturation of the PS in the compaction greatly affects the water interception tendency of the cover system. Since the compacted PS with the highest molding water content (175.4%), in this study, reaches saturated condition after compaction, the percolation from the compacted PS barrier layer, due to rainfall immediately after the installation, occurred at the earliest time. In contrast, when the PS is compacted under unsaturated condition, such as at the compaction water contents of 133.6% and 75.7%, the infiltrated rainfall is retained in the compacted PS barrier layers for several months after the installation. The water percolated from the barrier layer is not noticed until the barrier layer reaches the fully saturated condition. Furthermore, although the evaporation of soil moisture is considered in this analysis, the barrier layer does not recover the unsaturated condition once the saturated condition is reached. This is probably because the surface layer prevents significant evaporation, and the infiltrated rainfall is strongly retained by the compacted PS layer. In a simulation case for which the barrier layer is constructed using the compacted CS, the quantity of water percolated from the compacted CS barrier equals almost 0 cm throughout three-year period used for the analysis (*see* Fig. 5.12(a)). This tendency clearly indicates that the compacted CS does not reach the saturated condition even under such weather conditions as those in Owase.

With the above explanation, it is estimated that fluctuations in the moisture profile of the cover system provide an effect on the rainwater interception performance of the cover system. Figure 5.13 shows the distribution of volumetric water content in the cover system when the quantity of water percolated from the compacted PS has significantly increased (for the period of May to July, 1998). The water content increases throughout the layer during this period and reaches a saturated volumetric water content of the barrier layer on the July, if the compacted PS is assumed to be installed as a barrier layer in the cover system. When the barrier layer is constructed with the compacted CS, the rainfall infiltrating into the cover system also accumulates at the interface of the surface layer and the barrier layer. However, the infiltrated water cannot reach the bottom of the compacted CS barrier layer. As a result, the bottom of the compacted CS barrier layer maintains at its initial compaction water content. Other calculated results indicate that it takes at least about five years for the compacted CS barrier layer to be fully saturated under the weather conditions that exist at Owase. This is clarified in Fig. 5.14, which predicts the quantity of

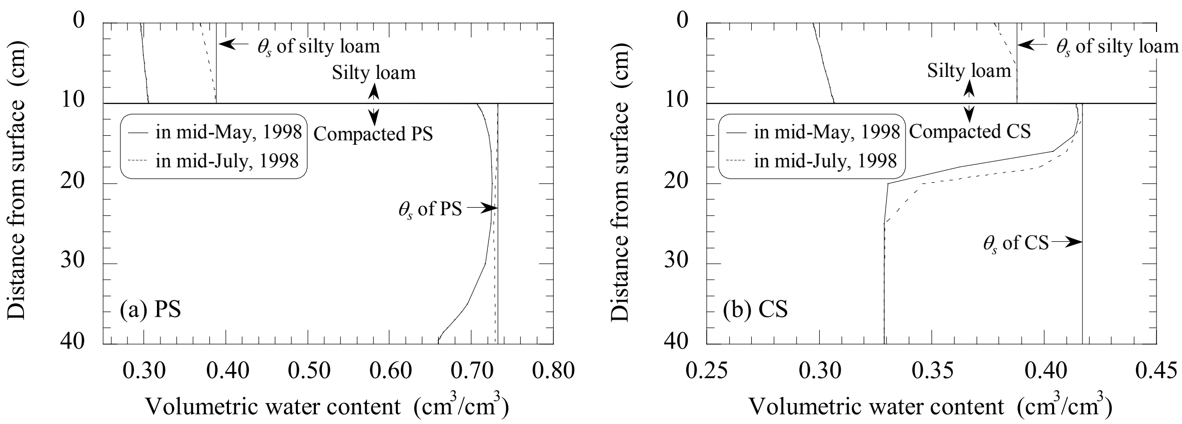

Fig. 5.13 Distribution of the volumetric water content in a cover system with (a) PS and (b) CS barriers

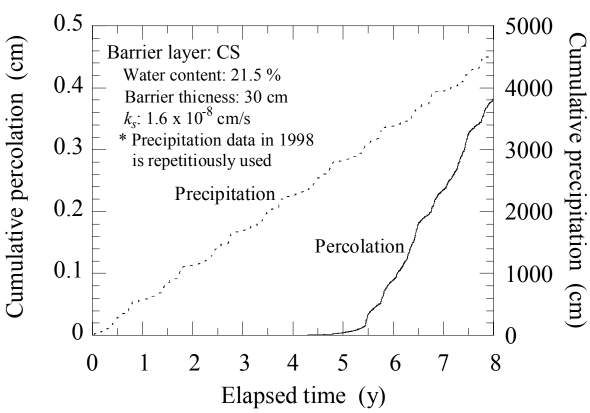

Fig. 5.14 Quantity of percolated water from the cover system with a CS barrier (water content of 21.5%, thickness of 30 cm) located in Owase

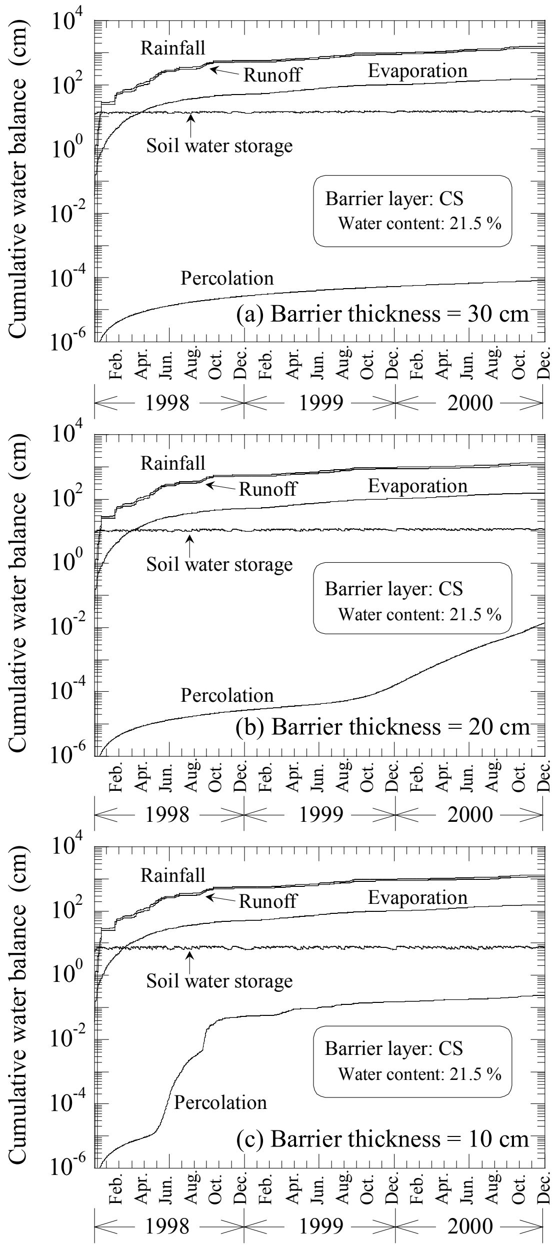

Fig. 5.15 Effects of barrier thickness on the results of the water balance analysis for a cover system with a CS barrier (water content of 21.5%)

water percolated from the CS barrier having a water content of 21.5% and a thickness of 30 cm for eight years using the meteorological data of 1998 at Owase repetitiously. Consequently, the cover system using the compacted CS as barrier layer shows a significant rainwater interception performance, because infiltration of the rainfall does not reach the bottom of the cover system in periods of heavy rainfall throughout the five years. In addition, the quantity of water percolated is significantly low in the five-year durations, because the compacted CS maintains a relatively low saturated hydraulic conductivity as 10^{-8} cm/s even after it reaches the saturated condition.

Figure 5.15 shows the effects of the thickness of the barrier layer on the water interception performance for a case in which the compacted CS with a water content of 21.5% is applied to the barrier layer. The quantity of cumulative water percolating from the barrier layer decreases with an increase in the layer thickness. For instance, the cumulative quantity from the compacted CS barrier layer with a thickness of 30 cm is over three orders of magnitude less than that from the barrier layer with a thickness of 10 cm, throughout the three years in the analysis. This is because the unsaturated zone increases with an increase in the barrier layer thickness. The quantity of percolating water is strongly controlled by the time it takes to reach a fully saturated condition.

The time it takes for the barrier layer to reach saturation depends on the weather conditions, and the unsaturated infiltration characteristics which are affected by the compaction conditions and the types of sludge. In the design of cover systems which use the compacted sludge, in terms of the water interception performance, a concern should be focused on whether or not the compacted sludge can retain the unsaturated condition over a long term. Evaluations of the water interception performance of cover systems, which consider the unsaturated and the saturated infiltration characteristics of the constituent layers, reproduces those in the field better than conventional evaluation techniques which depend on the saturated hydraulic conductivity represented by the HELP model. This is because the UNSAT-H model considers the changes in the water distribution within the cover systems. The UNSAT-H model also clearly indicates the beneficial effects of the cover systems, showing the unsaturated condition, on the water interception performance.

5.7. Summary and Conclusions

The generation of leachate containing toxic matters and the leakage of leachate from bottom liners is normally caused by the accumulating rainwater in the waste layers. Landfill cover systems can significantly cut off the infiltration of rainfall. This study clarifies the application of two types of sludge such as the PS and CS for barrier materials in landfill cover systems from the viewpoint of unsaturated infiltration characteristics. Soil-water characteristic curves for the two types of the compacted sludge are obtained experimentally using the proposed soil-water retention tests. The experimental results are identified using the van Genuchten and the Brooks-Corey models, which are representative fitting models. It is possible for these models to fit the soil-water characteristic curves of the compacted sludge. The feasibility of the two types of the compacted sludge as barrier materials is verified from the viewpoint of water retentiveness. While the unsaturated infiltration characteristics of the compacted PS and CS are evaluated experimentally, a water balance analysis which considered the unsaturated infiltration behavior of the constituent layers is carried out to predict the performance of the rainwater interception of landfill cover systems by the compacted PS and CS. From this analysis, the cover systems which apply the compacted PS and CS as barrier layer can maintain the unsaturated condition over several months and several years, respectively, even in so humid weather conditions as existed in

Japan, and the performance of the rainwater interception by the cover systems exhibiting the unsaturated condition is significant. The conclusive results of this study are as follows:

(1) The van Genuchten and the Brooks-Corey models are available for soil-water characteristic curves of the compacted PS and CS. In addition, the van Genuchten model provides more flexible nonlinearity and has a higher adaptability for the compacted PS and CS than the Brooks-Corey model.
(2) The PS, compacted at a higher level of water content, exhibits the microscopic pores of organic substances which may possess larger amounts of water than the inter-particle pores. The PS compacted at a higher level of water content can indicate a higher air-entry suction and water retentiveness than the PS compacted at a lower level of water content even at a higher level of suction.
(3) When the CS is compacted at a lower level of water content, the macroscopic pores formed in the inter-clods. In contrast, the CS compacted at a higher level of water content does not form the clod structures, then microscopic pores are produced. The compacted CS with microscopic pores can strongly retain the pore water. A higher level of molding water content of CS is considered to contribute to the water retentiveness.
(4) The unsaturated hydraulic conductivity of both PS and CS, compacted at higher levels of water content, significantly decreased with a decrease in the degree of saturation, compared with that compacted at a lower level of water content. This is because the unsaturated hydraulic conductivity of the compacted PS and CS is significantly affected by the soil-pore size distribution.
(5) As a result of the evaluation of the water interception performance using the UNSAT-H model, the quantity of the cumulative water percolating from cover systems is less than 15 cm, which equals approximately 1% of the total rainfall throughout the three years in the analysis. More than 99% of the rainfall could be intercepted by the installation of cover systems in Owase, Kyoto, and Tokyo, Japan throughout the three-year analyzing period.
(6) The time it takes for the barrier layer to reach saturation depends on the weather conditions and the unsaturated infiltration characteristics. In the design of cover systems using the compacted sludge, in terms of the water interception performance, it is most important whether the compacted sludge is under unsaturated condition or not.
(7) The UNSAT-H model clearly indicates the beneficial effects of cover systems, under the unsaturated condition, on the water interception performance.

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CHAPTER 6

Durability of Cover Systems with Sludge Barrier under Cyclic Wetting and Drying

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6.1. General Remarks

The main function of a landfill cover system is to minimize the infiltration of rainwater into underlying waste layer. For increasing the water interception by landfill cover system, it is important to maintain a low hydraulic conductivity of the barrier layer within its whole service duration. To retain the low hydraulic conductivity, cohesive soil is usually used. To evaluate the feasibility of a material as barrier layer in the cover system, its hydraulic performance is considered as an important factor. Another key issue is that the cover system is always exposed to a changing weather condition because it is generally close proximity to the atmospheric environment. The barrier material composed of the clay generally shows swelling and shrinkage behaviors under wetting-drying cycles. Cracks are caused by the excessive shrinkage, influenced by the drying velocity, soil particle composition, clay content, initial water content, and so on. It is obvious that the development of the cracks in the barrier material may cause a negative effect on the hydraulic performance of the barrier material. Practically, field studies reported by Brian and Benson (2001) showed that the desiccation induces severe cracking of unprotected clay barrier. Therefore, the long-term changes in the hydraulic conductivity of barrier material subjected to the wetting-drying cycles should be clarified in evaluation of the hydraulic durability.

This Chapter focuses on the hydraulic performance and desiccation shrinkage (cracking) behavior of paper sludge (PS) and construction sludge (CS) subjected to the wetting-drying cycles as the barrier material (i.e., sludge barrier) in cover system by conducting modified flexible-wall hydraulic conductivity tests. Furthermore, hydraulic conductivity tests with the wetting-drying cycles are carried out in a centrifugal loading field using geotechnical centrifuge in order to reveal the change in hydraulic conductivity of the sludge barrier under the wetting-drying cycles in a large time scale. The vertical depth of a crack generated is also predicted by analyzing the soil-water and suction distributions in the compacted sludge materials which is exposed to the drying process. The durability of the compacted sludge materials as a barrier layer in cover system is evaluated by calculating the changes in the suction distribution supposed that the sludge materials are exposed to the measured field weather conditions. In addition, the achievement performed in this Chapter has been reported by Kamon et al. (2001, 2002a, 2002b).

6.2. Desiccation of Soil

6.2.1. Shrinkage Characteristics of Soil

Shrinkage is one of the major causes for volume changes associated with variations of water content in soil. Haines (1923), Casagrande (1932), and Hogentogler (1937) have shown that when soil decreases its water content, the volume decreases. Based on the researches by Haines (1923) and Hogentogler (1937), the volume changes relating to soil consistency can be illustrated as shown in Fig. 6.1. Relationships among various soil moisture terms are approximate only. In examining Fig. 6.1, from point **a** to **b** is termed as "normal shrinkage", also called "primary shrinkage", and the volume changes linearly with decrease of water content. This linear shrinkage is due to surface tension forces of the capillary moisture. The

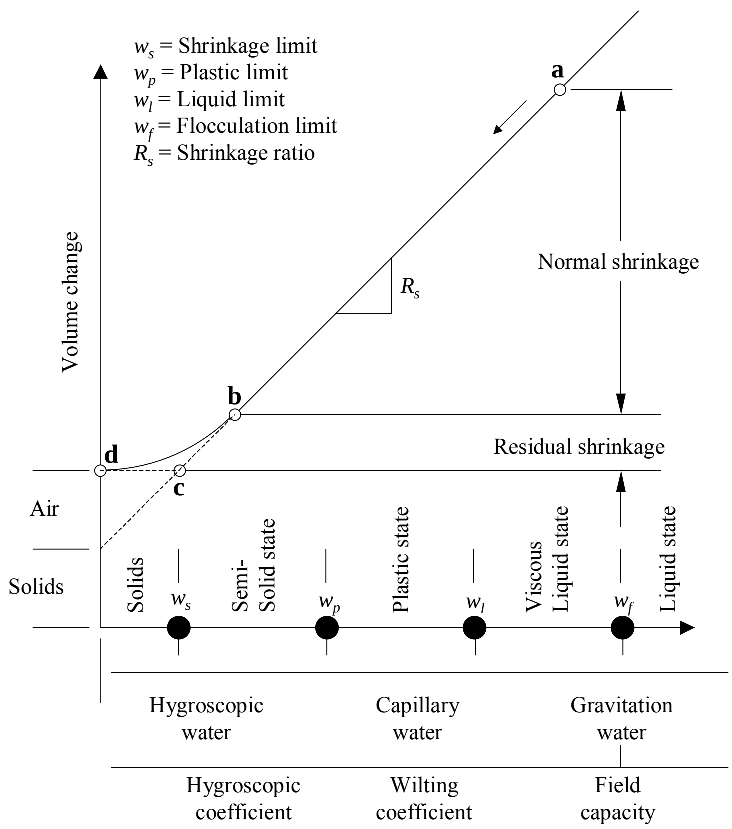

Fig. 6.1 Volume changes versus water content for soil (summarized from Haines, 1923; Hogentogler, 1937; Fang, 1997)

normal shrinkage of soil occurs as water leaves the soil without entry of air. Since air is not entering the soil, the volume change is equal to the volume of water leaving the soil. The majority of the total volume change occurs during the primary stage of drying. Water surrounding the individual soil particles is removed, allowing the soil particles to move closer together as the water retreats. When volume change reaches point **b** and the color of soil changes, a small amount of volume change from point **b** to **d** is termed as "residual shrinkage", also called "curvilinear shrinkage". At some points the soil particles contact each other, and the drying process slows as the structure of the soil begins to resist additional volume change. In this phase of drying, air enters the soil and replaces the water being removed because the particles are in contact. Therefore, the total amount of volume change is closely related to relative volumes of water and solids present in the soil as drying begin. Further decrease of water until there is no more change in volume as indicated at point **c** is called the shrinkage limit. The shrinkage limit can be used as a general index of clay content and will, in general, decrease with increases in clay content.

6.2.2. Literature Reviews for Desiccation Effect on Compacted Clays

Boynton and Daniel (1985) performed hydraulic conductivity tests on desiccated clay. Specimens were trimmed from plates of compacted soil prepared at three water contents, and placed in flexible-wall permeameters for testing. A specimen that had not been desiccated was also tested. At low effective stresses the hydraulic conductivity of the desiccated specimens was typically one-half to one order of magnitude greater than the hydraulic conductivity of the undesiccated specimen. The hydraulic conductivity of the desiccated specimens decreased rapidly with increasing effective stress from 30 up to 56 kPa, presumably due to the closure of cracks. The hydraulic conductivity decreased more gradually as the effective stress was raised beyond 56 kPa. At the highest effective stress, the hydraulic conductivity of each desiccated specimen was still greater than the hydraulic conductivity of the undesiccated specimen.

Sims et al. (1996) obtained similar results from tests on specimens collected in thin-walled sampling tubes from a desiccated natural deposit of clay. They also reported that the hydraulic conductivity decreased rapidly with increasing effective stress up to 120 kPa, and attributed the reduction in hydraulic conductivity to closure of cracks. Benson et al. (1993) performed hydraulic conductivity tests on two low-plasticity clays (Live Oak and Wenatchee clays) subjected to four cycles of wetting and drying. Specimens of each soil were prepared at 3% dry of optimum water content, optimum water content, and 3% wet of optimum water content. Specimens of Wenatchee clay compacted dry of optimum and at optimum water content showed no increase in hydraulic conductivity when desiccated, but the hydraulic conductivity the specimen prepared wet of optimum water content increased by a factor of three. For the Live Oak clay, the hydraulic conductivity of all specimens increased one order of magnitude within the first two wetting-drying cycles, but ceased increasing thereafter.

Phifer et al. (1994 and 1995) conducted tests on processed kaolinite and a natural kaolinitic soil to assess how desiccation affected hydraulic conductivity. Desiccation caused the specimens to shrink significantly (volumetric shrinkage strains: 20%) and the dry unit weight to increase. However, no cracks formed, and decreases in hydraulic conductivity were observed. Subsequently, Phifer et al. (1995) conducted laboratory-scale lysimeter tests to determine if similar behavior would be happened at larger scales as observed by Drumm et al. (1997). Results of the lysimeter tests showed that the soil cracked when desiccated, and that the bulk hydraulic conductivity (i.e., crack and matrix flow) increased as much as two orders of magnitude. Day (1997) performed hydraulic conductivity tests on a specimen prepared from a mixture of montmorillonite and sand, and a specimen of highly plastic clay from a natural deposit. Both specimens were subjected to five cycles of wetting and drying. At the end of one day of permeation, the hydraulic conductivity of each specimen was observed to be essentially the same as its initial hydraulic conductivity.

6.3. Experimental Approaches to Desiccation Effect of Sludge on Hydraulic Conductivity

6.3.1. Materials Used

The applicability of paper sludge (PS), generated from paper factory effluent treatment facility and dehydrated using a belt press, and construction sludge (CS), generated from the shield tunnel construction site, to the barrier material in cover systems through the laboratory experiments have been verified in the preceding Chapters. The sludge materials used in the following experiments are the same as those reported in preceding Chapters. The basic properties of these two types of sludge are shown at Table 3.3.

6.3.2. Shrinkage Tests

Each sludge material adjusted to the target molding water content was compacted in a compaction mould having a diameter of 6 cm and a height of 3 cm to achieve the same water content and the same compaction density as the ones obtained from the standard compaction curves. The degree of saturation of compacted sludge was elevated in suction chamber. The specimens were extruded from the compaction mould after they were fully saturated. Subsequently, the specimens were dried in a constant temperature drying room at 25°C. The weight of specimen was periodically measured using a electronic balance of 0.01 g sensitivity and the diameter and height of specimens were measured using a vernier caliper, until no further decrease in water contents were observed. The water content and volumetric shrinkage (change in volume/initial volume) of the specimens during drying were calculated from these measurements.

6.3.3. Hydraulic Conductivity Test under Wetting-Drying Cycles

Hydraulic conductivity tests were conducted using flexible-wall permeameters on the compacted PS and CS subjected to the wetting-drying cycles. The compaction method of specimens is as the same as that in the shrinkage tests. The specimens were directly placed in the hydraulic conductivity test apparatus; afterwards the degree of saturation of the specimen was increased by water infiltration. A confining pressure of 30 kPa was supplied for the specimens. Zimmie and Moo-Young (1995), and Quiroz and Zimmie (1998) have reported that 30 kPa is a typical confining pressure applied to the barrier layer in the cover system. Furthermore, Brian and Benson (2001) reported that the level of loading pressure existing typically in cover applications (i.e., less than 30 kPa) is not sufficient to close cracks caused by desiccation. The testing period of hydraulic conductivity for the compacted sludge materials is considered as a wetting stage. The details of the hydraulic conductivity test using the flexible-wall permeameter were discussed in Chapter 3. The specimens were removed from the test apparatus after a stable hydraulic conductivity was obtained during the wetting stage. Then, the whole surface areas of the specimens were uniformly dried in a dry room at a constant room temperature of 25°C (referred to as drying stage). Procedure and measuring factors in the drying stage are similar to those of the shrinkage tests. The specimens after the drying stage were reset into the hydraulic conductivity test apparatus, and the hydraulic conductivity test (second wetting stage) was conducted again. This process repeated until a desirable wetting-drying cycles reached.

6.3.4. Geotechnical Centrifuge Test under Wetting-Drying Cycles

The hydraulic evaluation of the compacted PS and CS subjected to the wetting-drying cycles was also carried out in centrifugal loading field. The measuring method of hydraulic conductivity for the compacted sludge using a geotechnical centrifuge is described in Chapter 3. In this experiment, the hydraulic conductivity test was carried out for 3 hours (wetting stage) under 30 G field on the PS and CS compacted at the height of 5 cm in a stainless-steel tank (45 cm x 7.5 cm). Subsequently, the surface of compacted PS and CS was dried using a dryer in 1 G field for 24 hours (drying stage). The drying temperature was almost 50°C. After the drying stage, the hydraulic conductivity test was carried out again under the centrifugal field. Above drying-hydrating cycles was repeated several times. After each drying stage ended, shrinkage settlement of the compacted PS and CS layer was measured using the vernier caliper, and crack depth was measured inserting into the crack by a wire of 1 mm in diameter. The outline of the procedure of geotechnical centrifuge test under Wetting-Drying Cycles is shown in Fig. 6.2.

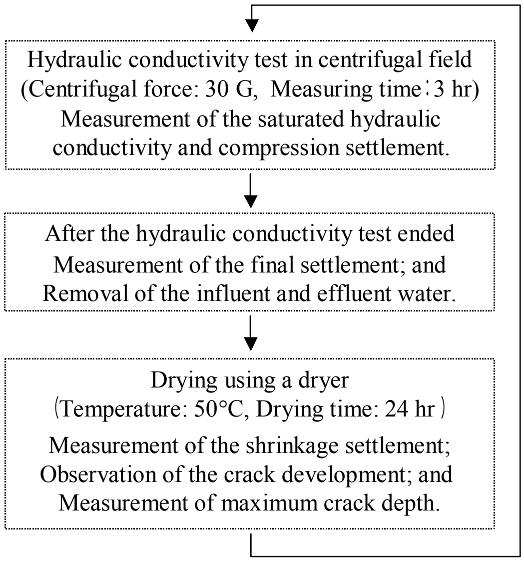

Fig. 6.2 Procedure of the centrifuge tests under the wetting-drying cycles

6.4. Results and Discussions

6.4.1. Shrinkage Properties

Figure 6.3 shows the change in a volumetric shrinkage strain on the PS and the CS with elapsed time during the shrinkage tests. The formation of the cracks caused by the desiccation shrinkage is not observed except for PS specimen compacted at the highest molding water content with 186.6 %. Although the significant cracks do not occur, the maximum volumetric shrinkage strains are 25% and 18% for the compacted PS and CS, respectively, which is thought to be significant. The dry density of the PS whose compaction is possible at relatively high water content is smaller than that of CS. It seems that the PS shows larger volumetric shrinkage strain than that of CS. The effect of the compacted water content and the dry density of each sludge on volumetric shrinkage strain is shown in Fig. 6.4. The volumetric shrinkage strain depends on the molding water content and dry density of the specimens. Both PS and CS specimens compacted around the optimum water content result in the smallest volumetric shrinkage strain. When soil is compacted dry or wet of optimum water content, the dry unit weight decreases and more water and fewer solid particles exist per unit volume. More room is available for the soil particles to collapse before contacting each other during drying, resulting in larger volumetric shrinkage strain, as described on compacted natural clays by Brian and Benson (2001).

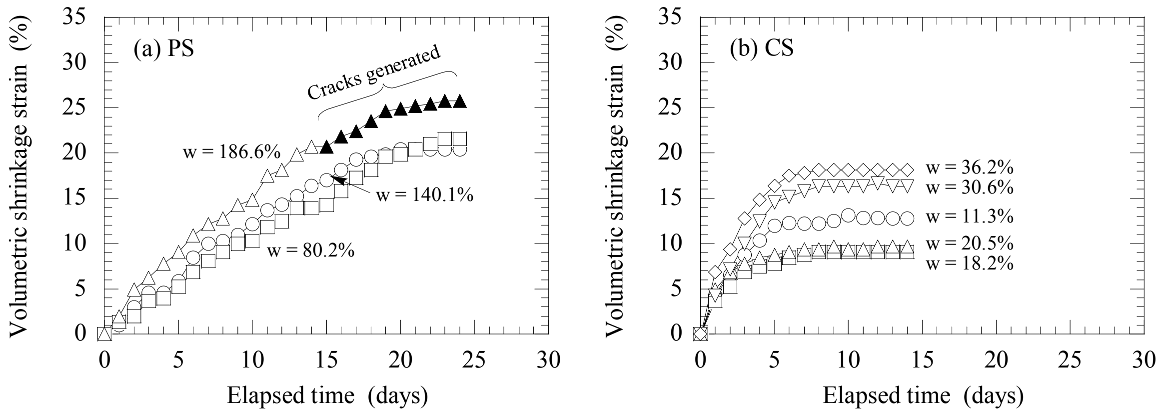

Fig. 6.3 Desiccation shrinkage strain of the compacted (a) PS and (b) CS

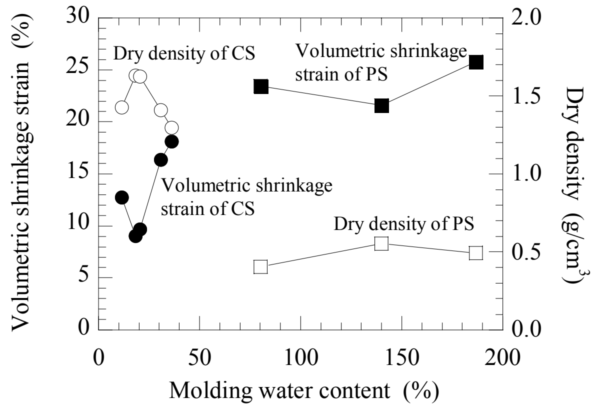

Fig. 6.4 Effect of molding water content and dry density on volumetric shrinkage strain

6.4.2. Saturated Hydraulic Conductivity under Wetting-Drying Cycles

Figure 6.5 indicates the change in specimen volume from the hydraulic conductivity tests under the wetting-drying cycles. The degree of shrinkage and swelling of the compacted PS and CS in one wetting-drying cycle are different from that in the previous cycle, because the immediately pasted drying stage affects subsequent swelling behavior. If the shrinkage of the compacted PS and CS are in normal shrinkage during drying stage, the pore space decreases and the saturation state is still maintained. Thereby, the compacted sludge subjected to the drying cycle can resist for subsequent intrusion of the water from outside. Although the specimens show shrinkage and swelling behavior under the drying-wetting cycles, cracks do not occur except for the PS compacted at the water content of 186.6% as well as the case of the shrinkage tests.

Figure 6.6 shows variation in the hydraulic conductivity and the void ratio for compacted PS and CS subjected to the wetting-drying cycles. The hydraulic conductivity of

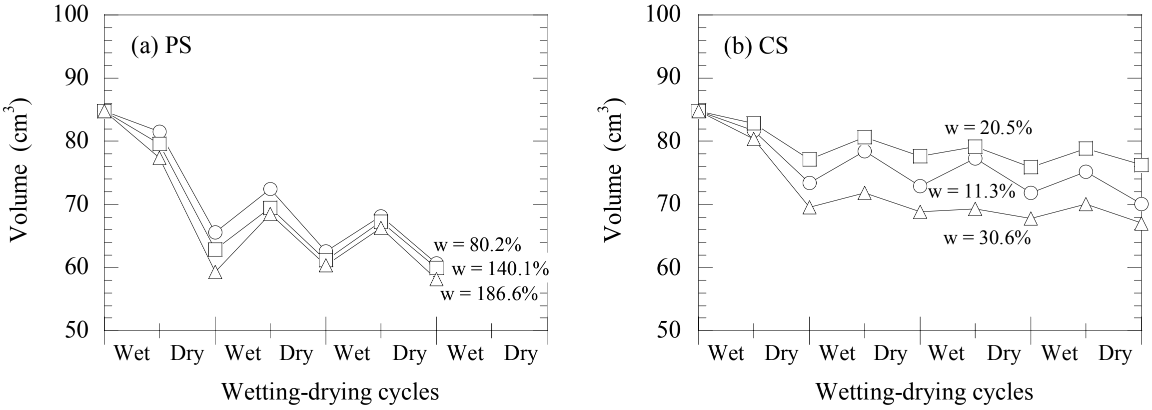

Fig. 6.5 Volume change behavior of the (a) PS and (b) CS subjected to the wetting-drying cycles

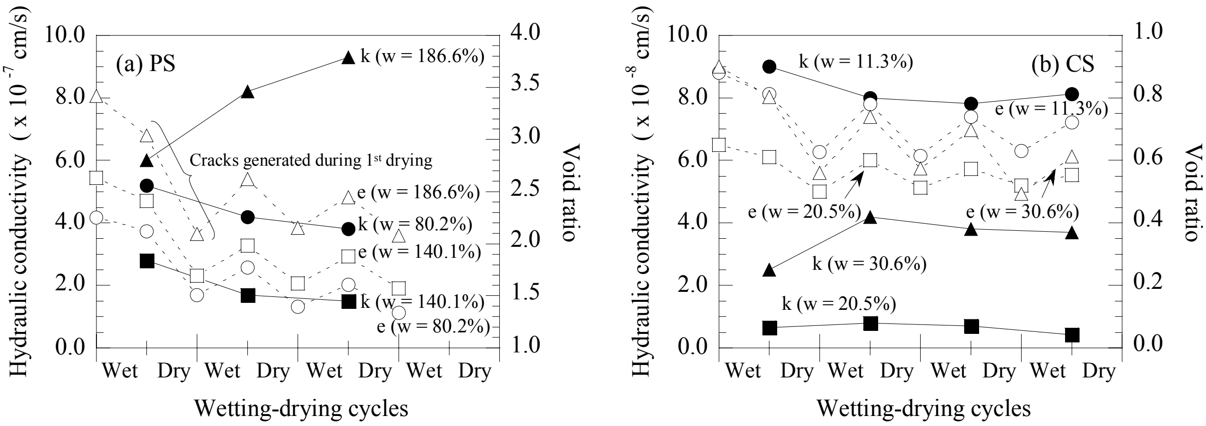

Fig. 6.6 Hydraulic conductivity and void ratio of (a) PS and (b) CS subjected to wetting-drying cycles

the PS with molding water content of 186.6%, of which the cracks are generated during the first drying stage, increased with increasing in the number of wetting-drying cycles. Cracks occurred during the drying stage introduce a macro pore structure, and these pores controll the hydraulic conductivity during subsequent hydration. Unlike the PS (molding water content of 186.6%), the shrinkage without any cracks in the compacted PS and CS seems that consolidation happened, even if shrinkage do generated by the drying. The shrinkage without any cracks contributes to maintaining or decreasing the original hydraulic conductivity. In the evaluation of the hydraulic conductivity on shrinkage, the development of cracks must be a key issue.

6.4.3. Cracks Induced by Desiccation Shrinkage

For most of the shrinkage and hydraulic conductivity tests on the compacted PS and CS, the development of cracks induced by the desiccation shrinkage were not observed. This is because the cracks due to the shrinkage may occur, if the normal shrinkage during the drying is restrained. Capillary force is increased with decreasing in the meniscus radius of the inter-particle due to the dehydration; consequently normal shrinkage is developed. When the shrinkage is restrained, large tensile stress is generated inside of the specimen. As a result, the cracks will occur. Brian and Benson (2001) have observed the shrinkage behavior of compacted clay specimen with a 10.2 cm in diameter under drying procedure similar to that in this study. They found that the generation of the cracks was remarkable for the specimen with higher molding water content. Since in the present experiments, the size of the specimen used is smaller, and the drying of the specimen is more uniformly carried out than that conducted by Brian and Benson (2001), the shrinkage of the specimen may not be restrained. Because of this reason, it is thought that no large cracks induced following the shrinkage in the present experiments. Abu-Hejleh and Znidarčić (1995), and Konrad and Ayad (1997) theoretically discussed the desiccation shrinkage of soft cohesive soil and the generation mechanism of the cracks with drying. Similar results were reported by them.

6.4.4. Durability Evaluation by Centrifuge Test

Figure 6.7 indicates shrinkage settlement and swelling displacement on the surface of compacted PS layer with molding water content of 138.2 % under wetting-drying cycles during the geotechnical centrifuge tests. The maximum crack depth is also plotted in Fig 6.7. The significant settlement in the first wetting stage is due to the consolidation effect. The shrinkage settlement is remarkable in the first drying stage. In the subsequent drying-wetting processes, the shrinkage settlement and the swelling displacement are relatively smaller than that in the first drying stage. Crack depth, however, increases with the increasing in the number of drying from the second drying step. Although shrinkage is remarkable in the first drying, no cracking occurs. However, during the second drying, cracks develop although the shrinkage is small. These mechanisms will be explained as following. The compacted sludge layer is receiving the side restraint due to the shape and the size of layer. Therefore, a completely free condition exists only on the surface of the compacted sludge. Under such a condition, vertical shrinkage is firstly generated on the compacted sludge in drying stage, subsequently, the horizontal shrinkage is generated and cracks seem to be developed in this stage.

Figure 6.8 shows the hydraulic conductivity of compacted PS under the wetting-drying cycles. From the initial value of 3.0×10^{-7} cm/s, the hydraulic conductivity increases semi-log proportional to the crack depth, after cracks are formed during the second drying stage. Five cycles of wetting-drying result in the maximum crack depth of about 30%

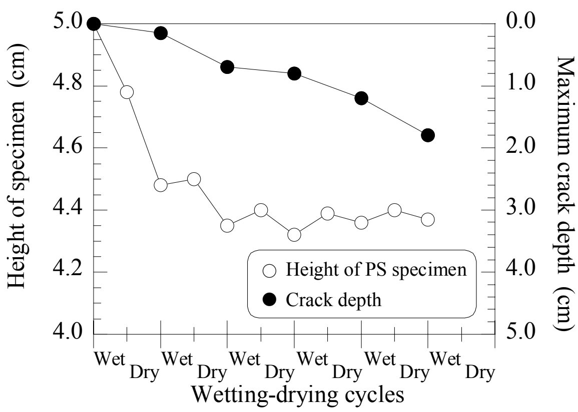

Fig. 6.7 Relation between crack depth and specimen height for PS under wetting-drying cycles

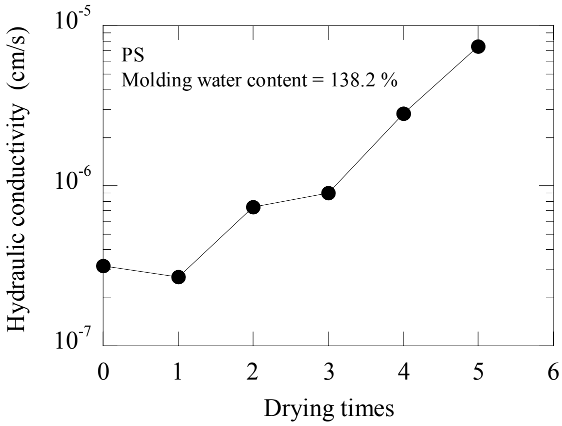

Fig. 6.8 Variation of the hydraulic conductivity of PS with drying times

of the layer thickness, and more than one order of magnitude increase in the hydraulic conductivity is observed.

6.5. Simulation of Sludge Barrier Durability under Wetting-Drying Cycles

6.5.1. Simulation Methods

By analyzing the soil-water and suction distributions in the compacted sludge materials exposed to the drying process, the vertical depth of a crack generated may be predicted. Furthermore, the durability of the compacted sludge materials is evaluated by calculating the changes in the suction distribution after being exposed to the measured field weather conditions. For analyzing the distributions of soil-water and suction in the compacted sludge materials under drying process, an evaporative flux from the surface of compacted sludge materials needs to be calculated. The UNSAT-H model was used for the estimation of the evaporative flux from the surface of compacted sludge materials and the changes in the suction distribution with the evaporative flux. In the UNSAT-H model, the evaporation is calculated using an integrated form of Fick's law of diffusion, which considers the flow of heat to and from the soil surface, the flow of water from subsurface to the soil surface, and the transfer of water vapor from the soil surface to the atmosphere. The governing equation used in the UNSAT-H model is shown in Eq. (5.7) of Chapter 5. A term of q_{vt} in the Eq. (5.7) represents thermal vapor diffusion. The thermal vapor diffusion, q_{vt}, is derived from the Fick's law and is defined by Eq. (6.1):

$$q_{vt} = -\frac{D}{\rho_w} H_R \frac{\partial \rho_{vs}}{\partial T} \frac{\partial T}{\partial z} \tag{6.1}$$

where q_{vt} is the flux density of thermal vapor diffusion (cm/hr), D is the vapor diffusivity in soil (cm^2/hr), ρ_w is the density of liquid water (g/cm^3), ρ_{vs} the saturated vapor density (g/cm^3), T is the temperature (K), z is the in-depth distance (cm), and H_R is the relative humidity. The vapor diffusivity in soil, D, and relative humidity, H_R, can be determined using Eqs. (6.2) and (6.3) (Campbell, 1987), respectively:

$$D = \alpha(\theta_s - \theta)D_a \tag{6.2}$$

where α is the tortuosity factor (the most common formulation is to set α equal to 0.66 (Penman, 1940; van Bavel, 1952)), D_a is the diffusivity of water vapor in air, which typical value is 0.24 (cm^2/s), and the quantity (θ_s - θ) represents the air-filled porosity.

$$H_R = -\exp\left[-\frac{\phi M g}{RT}\right] \tag{6.3}$$

where M is the molecular weight of water, which typical value is 18 (g/mol), g is the gravitational acceleration (cm/s^2), R is the gas constant, which typical value is 8.317 (J/(mol·K)), and ϕ is the matric potential head due to capillary suction forces.

For analyzing the suction or soil-water distribution generated in the compacted sludge materials under drying process by using Eq. (6.1), the unsaturated infiltration parameters for the compacted sludge materials have to be applied. Thus, the data of soil-water retention curve for the compacted sludge materials, which were discussed in Chapter 5, are applied for calculating the distribution of soil-water and suction during drying process. The relationship between volumetric water content, θ, and matric suction, ϕ, of the compacted sludge materials is quantified applying the van Genuchten model in this simulation. Furthermore, the van Genuchten-Mualem function is applied to estimate unsaturated hydraulic conductivity of the compacted sludge materials. The details for the unsaturated infiltration characteristics of the compacted PS and CS as barrier materials have been mentioned in the Chapter 5. The simulation outline for the UNSAT-H model has been shown in Fig. 5.9. In the simulation, the shrinkage due to drying is not considered.

6.5.2. Prediction of Crack Depth in Compacted Sludge under Drying Process

The depth of the crack generated in the compacted PS during drying process observed from the geotechnical centrifuge test is compared with the simulated suction distribution in the compacted PS. Figure 6.9 shows the distribution of suction in the compacted PS layer (water content of 138.2%) subjected to the drying of 50°C. Regarding the drying conditions, since the evaporation was one-dimensionally carried out in the both cases of centrifuge test and simulation on the sludge surface under the constant temperature condition of 50°C, the drying conditions in the case of the simulation and the centrifuge tests are considered to be almost the same. Observed crack depth in the centrifuge test was also plotted in the Fig. 6.9 by converting drying time of 24 hours in each drying stage to the cumulative drying time. The suction gradient is produced in the compacted PS due to the drying evaporation. Comparing the observed crack depth with the estimated suction distribution, the observed crack depth in each drying stage is achieved up to the close vicinity of the inflection point (drying front) of the suction distribution. The drying front is defined as a boundary region where separates the already-dry region from that which is undergoing or waiting drying. The speed with which the drying front moves depends on the characteristics of the drying conditions (e.g., temperature, relative humidity) and of the soil to be dried (e.g., type, water content, thickness of the layer). It is predicted that the vertical crack depth due to the drying of the compacted PS is developed with the drying front progressing. Furthermore, the hydraulic conductivity of the compacted PS is increased due to the cracks development (*see* Fig. 6.8). Therefore, landfill cover system in which the compacted sludge is applied as the barrier layer, should have a protective layer to prevent the development of cracks that induced by the drying evaporation in the sludge barrier. Evaluating the effect of protective layer on the drying cracks in the barrier layer will be another important issue in the design of cover system.

6.5.3. Simulation Results

Suction distributions in the compacted PS barrier layer, with a water content of 138.2% and a 30 cm in thickness, are estimated using the weather data observed in Kyoto area, 1998, as input data, for the purpose of evaluating the durability under wetting-drying cycles as a sludge barrier in cover system. The simulation results indicate (a) variations in suction value at the specific depth of compacted PS with elapsed time, and (b) the suction distribution in the PS

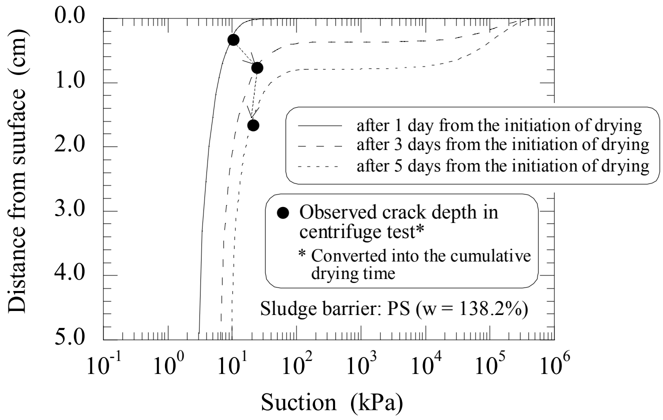

Fig. 6.9 Variation in suction distribution of compacted PS subjected to the drying evaporation

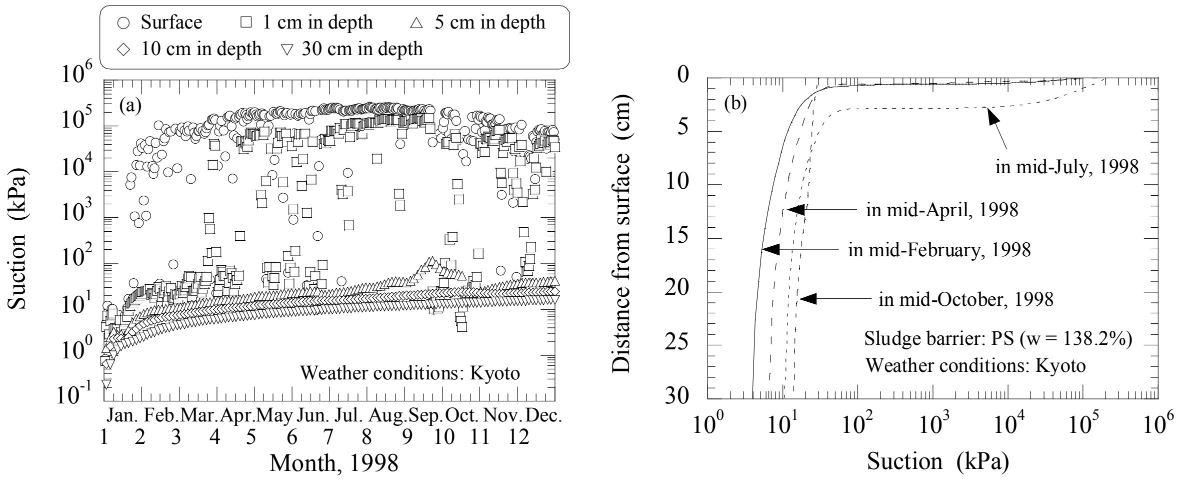

Fig. 6.10 Variation in suction distribution of compacted PS subjected to measured field weather conditions

barrier in specific days, are shown in Fig. 6.10. The variation of suction in the vicinity of compacted PS surface is relatively larger than that in deeper depth of compacted PS (*see* Fig. 6.10(a)); thus the vicinity of the compacted PS surface is affected by the wetting-drying cycles through a year. The drying front approximately achieves to the depth of 5 cm from the surface in the most dried season in Kyoto (*see* Fig. 6.10(b)). If a vertical crack is assumed to develop up to the drying front, a crack generated in the compacted PS may be developed up to the depth of 5 cm from the surface, under the weather conditions of Kyoto. Consequently, maintaining a sufficient barrier thickness larger than the crack depth, and putting a protective soil layer on sludge barrier are considered to be important countermeasures for protecting the durability of sludge barrier under the wetting-drying conditions.

6.6. Summary and Conclusions

In this Chapter, the hydraulic performance and desiccation shrinkage (cracking) behavior of paper sludge (PS) and construction sludge (CS) subjected to the wetting-drying cycles as the landfill barrier materials (i.e., sludge barrier) in cover system are discussed by conducting the flexible-wall hydraulic conductivity tests and geotechnical centrifuge tests. The geotechnical centrifuge tests are performed to simulate the durability of the hydraulic conductivity under the wetting-drying cycles of the sludge barrier at the field scale for a shorter period of time. Furthermore, the generated vertical depth of a crack was predicted by analyzing the soil-water and suction distributions in the compacted sludge materials which exposed to the drying process. The durability of the compacted sludge materials as a barrier layer in cover system is evaluated by calculating the changes in the suction distribution under exposing to the measured field weather conditions. The following conclusions can be drawn:

(1) The molding water contents of PS and CS have large effect on desiccation shrinkage. Both CS and PS compacted around their optimum water contents show the smallest volumetric shrinkage strain.

(2) The degree of the shrinkage and the swelling of the compacted sludge under wetting-drying cycle are different from the previous cycle, because the drying stage in the past cycle affects subsequent swelling behavior. The compacted sludge subjected to the drying-stage can resist the subsequent intrusion of the water from the outside.

(3) Shrinkage without cracking contributes to maintaining the originally low hydraulic conductivity of compacted sludge. Therefore, the development of cracks is mainly responsible to the increase the hydraulic conductivity.

(4) When the crack generated by desiccation shrinkage under restraint condition, the crack depth in PS became about 30% of the layer thickness after 5 wetting-drying cycles. And on increase in hydraulic conductivity is observed more than 1 order of magnitude.

(5) Maintaining a sufficient barrier thickness laying a protective layer on sludge barrier so that a drying front may not develop in a sludge barrier, are considered to be an important countermeasures for maintaining the durability of sludge barrier under the wetting-drying conditions.

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CHAPTER 7

Landfill Gas Migration under Installation of Cover Systems with Sludge Barrier

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7.1. General Remarks

The installation of cover system on reclaimed waste layer in landfills is required to reduce the negative impacts of leachate generated by the waste layer, on the areas surrounding the landfills. The most important function of the cover system is to control the rainwater infiltration into the underlying waste layer. For this purpose, the compacted clay materials, which can maintain low hydraulic conductivity, have been generally used as the barrier material (i.e., clay barrier) in cover system. Because paper sludge (PS) and construction sludge (CS) have low hydraulic conductivity, the feasibility of these two types of sludge as the barrier material (i.e., sludge barrier) in cover system has experimentally evaluated, in the preceding Chapters. In addition, the performance of rainwater interception of the cover system with PS and CS barriers has been quantitatively evaluated, using the water balance analysis with considering the saturated or unsaturated conditions of the PS and CS barriers.

In the landfill where the cover system is installed, the anaerobic condition of waste is promoted due to the scarcity of oxygen intrusion into the waste layer. The gas, called "landfill gas", is generated by the biological decomposition of organic components in reclaimed waste. The landfill gas is composed of methane, carbon dioxides, carbon monoxide, ammonia, and other ingredients. The carbon dioxides and the methane are mainly generated under an aerobic and an anaerobic condition, respectively. Thus, the landfill gas whose major component is methane is mainly generated in the landfill where the cover system with low permeability of water and gas is installed. The landfill gas cannot be released to the atmosphere due to the installation of cover system with low gas permeability. As a result, the generated landfill gas is accumulated inside the landfill, and the gas pressure is increased within the waste layer. The cover systems, as one of the critical containment facilities in landfill, are designed considering their rainwater interception performance. Therefore, at the landfill with the cover systems, the design of landfill gas control systems, for intensively releasing or controlling the generated landfill gas without accumulating the gas pressure within the waste layer, is another critical issue. As one of the countermeasures considered, when the cover system with low gas permeability is installed on reclaimed waste, the gas permeability of the cover material should be clarified, and the adequate gas vent well, which can reduce the gas pressure increased in the waste layer, should be equipped.

In this Chapter, the coefficient of gas (air) permeability for compacted PS and CS is measured experimentally in order to estimate the gas permeability of these two types of sludge. The relationship between the compaction conditions (depended on a molding water content, and a dry density), the water saturation, and a coefficient of air permeability is evaluated on the PS and CS. After clarifying the gas permeability of PS and CS as sludge barrier in cover system, the performance of the passive gas vent well in the waste layer with cover system is discussed. The influenced region by single passive gas vent well on reducing the pressure of ambient landfill gas is calculated parametrically. The contents in this Chapter have been reported by Kamon et al. (2002a).

7.2. Generation and Migration of Landfill Gas

7.2.1. Compositions of Landfill Gas

Landfill gas is initially generated by an aerobic decomposition process during waste reclamation, and its generation continues until the provision of free oxygen to sustain the process will become insufficient (Maciel and Jucá, 2000). After installing the cover system completely with materials having low hydraulic conductivity, an anaerobic decomposition of the organic components in waste takes place. During the first phase of anaerobic decomposition, the presence of organic acids reduce the leachate pH to 4 or 5, making it toxic for the methane production bacteria. A small amount of methane is generated during this period. As the process continues, methane productions bacteria will be more predominant (second anaerobic phase) and will be faster the transformation of volatile acids to methane and carbon dioxide. As a result, the pH increases to neutral range values of 7 to 8. After a long period, the production of methane will be reduced and more aerobic conditions will be established by means of oxygened water percolation through cover layers into the mass of waste (Maciel and Jucá, 2000). Maciel and Jucá (2000) reported some important factors affecting the composition and amount of gases generated in landfills. Table 7.1 presents some of these factors related to landfill gas production.

Landfill gas is a mixture of different gases and its properties may vary according to the composition. Its composition depends on the waste characteristics, types of cover, age, and the position correcting the sample. The typical percentage distributions of gases found in MSW landfill, as well as data on molecular weight and density are shown in Table 7.2. The main constituents of landfill gas, as shown in Table 7.2, are the methane (CH_4) and carbon dioxide (CO_2), called “Greenhouse Gases” and considered to be responsible for Earth-heating. In addition, methane is a flammable gas when in open atmosphere, explosive in a confined space, and asphyxiant either alone or mixed with air. Some of minor landfill gas constituents

Table 7.1 Factors controlling the landfill gas generation (adapted from Maciel and Jucá, 2000)

Physical dimension of the site	Anaerobic processes normally dominate in depth greater than 5 m.
Waste type	The composition of waste affects the rate, quality, and quantity of gas generated.
Site operations	Reduction of waste volume by compaction and rapid infilling of a small area of a site will shorten the aerobic degradation.
Waste density	At greater waste density, higher the production of gas per unit volume of void space.
Moisture content	Increase in moisture content by recirculation of lachate in the cells will accelerate gas generation.
pH within the landfill	Methane production will proceed optimally between a pH range of 6.5 to 8.5.
Waste temperature	The optimum temperature range for maximizing the methane production is between 35°C and 45°C.
Ingress of oxygen	Undesirable presence of oxygen during the anaerobic phase will delay gas generation.

Table 7.2 Typical components found in landfill gas (summarized from Tchobanoglous et al., 1993)

Component	Percent (v/v) (%)	Molecular weight (g/mol)	Volumetric weight (g/cm^3)
Methane	45 - 60	16.03	7.18×10^{-4}
Carbon dioxide	40 - 60	44.00	1.98×10^{-3}
Nitrogen	2.0 - 5.0	28.02	1.25×10^{-3}
Oxygen	0.1 - 1.0	32.00	1.43×10^{-3}
Sulfides,etc.	0 - 1.0		1.54×10^{-3}
Ammonia	0.1 - 1.0	17.03	7.72×10^{-4}
Hydrogen	0 - 0.2	2.016	8.97×10^{-4}
Carbon monoxide	0 - 0.2	28.00	1.25×10^{-3}
Trace constituents	0.01 - 0.6		

could have toxic effects, especially in industrial waste landfills (Maciel and Jucá, 2000). Therefore, these dangerous properties need to be minimized and one of the ways to prevent such problems is to transform and utilize landfill gas as a combustible gas.

7.2.2. Landfill Gas Migration Through Soil Barrier

Many investigations of landfill gas migration have demonstrated that both diffusive and advective transport can be important processes (Ghabaee and Rodwell, 1989):

Diffusion

Diffusive fluxes are caused by variation in gas concentrations within the porous medium due to the Brownian movement of the gas molecules. The diffusive flux can be described by

Fick's law shown in Eq. (7.1):

$$J_d = -\varepsilon_a D \frac{\partial C}{\partial x} \tag{7.1}$$

where, J_d is the diffusive flux (g/m^2·s), ε_a is the gas-filled porosity, D is the diffusion coefficient in the soil (m^2/s), and $\partial C/\partial x$ is the concentration gradient (g/m^4). Millington and Quirk (1961) gave further details for the calculation of the diffusion coefficient in the soil, D, as shown in Eq. (7.2):

$$D = D_0 \tau \tag{7.2}$$

where, D_0 is the diffusion coefficient of air at the given temperature (m^2/s) and τ is the tortuosity which can be estimated using following Eq. (7.3):

$$\tau = \frac{\varepsilon_a^{7/3}}{\varepsilon^2} \tag{7.3}$$

where, ε is the total porosity of the soil. Sallam et al. (1984) found from laboratory experiments that this expression gives a reasonable description of the changes in diffusion as a result of changing air-filled porosity (obtained by changing the water content).

For diffusion through unsaturated soil, Fick's law only holds if the mean free path of a gas molecule is smaller than the pore size (i.e., the gas molecules collide with each other within the pore space) (Williams and Aitkenhead, 1991). Most pores even in clay are much bigger than 500 Å (small pore will be water-filled), it means that the Knudsen diffusion, which is usually encountered when the mean free path of gas molecules is bigger than the pore diameter, has no importance in most case.

Advection

The advective flow of gas in porous media is similar in some ways to the flow of water, and different in other ways. The similarity lies in the fact that the flow of both fluids is usually impelled by a pressure gradient. The dissimilarity results from the relative incompressibility of water in comparison with gas, which is highly compressible so that its density and viscosity are strongly dependent on pressure as well as temperature. Furthermore, Dullien (1979) pointed out that the flow of gases due to pressure gradient differs from the flow of liquids in that the velocity at the pore walls cannot generally be assumed to be zero for gas transport. Darcy's law, which governs the flow of liquids in porous media, is based on viscous flow in which the velocity is zero along the pore walls. Non zero velocities at the pore wall will results in greater flow than predicted by Darcy's law. This additional flow is termed "slip flow" or "drift flow". However, flow measurement performed by Alzaydi and Moore (1978) showed that Darcy's law can provide a fair approximation of gas flow in a low gas permeability material. This indicates that the magnitude of slip flow is very small relative to viscous flow. Brusseau (1991) also indicated that the slip flows are not observed when the pressure difference is low and can on this basis be excluded from the modeling process for gas advective transport conditions. He also stressed the fact that for low pressure difference gas compressibility can be neglected and therefore the imcompressibility assumption is valid. It has also been shown that the equation for motion of gas should of the same form as the equation for motion of groundwater (Darcy's law), if the maximum pressure difference is less than 80 kPa (Bouazza and Vangpaisal, 2000). In any case, it is highly unlikely that the differential pressure across a typical landfill cover be higher than 10 kPa (Bouazza and Vangpaisal, 2000; Manassero et al., 2000).

Based on the above analysis, if the mass flow of landfill gas in porous media is assumed as following the Darcy's law, the advective flux of a gas constituent through a porous media is described as following Eq. (7.4):

$$J_a = -C \frac{K}{\mu} \frac{\partial P}{\partial x} \tag{7.4}$$

where, J_a is the advective flux (g/m^2·s), C is the gas concentration of the constituent (g/m^3), K is the intrinsic gas permeability of the porous material (m^2), μ is the dynamic viscosity of the gas (Pa·s), and $\partial P/\partial x$ is the pressure gradient (Pa/m).

The relationship between intrinsic gas permeability and coefficient of gas permeability of the porous material is described as Eq. (7.5):

$$k = \frac{K\rho g}{\mu} \tag{7.5}$$

where, k is the coefficient of gas permeability of the porous material (m/s), ρ is the density of the gas, and g is the gravitational acceleration (m/s^2). Thus, it is assumed that the intrinsic gas permeability is a function only of the properties of the porous material, not the permeating gas.

7.2.3. Coefficient of Gas (Air) Permeability for Soil

The landfill gas passage from the waste layer to the atmosphere is done through a barrier layer in cover system. The compacted clay or sludge used for the barrier layer is usually under unsaturated condition. In this case, coefficient of gas (air) permeability, k_a, and hydraulic conductivity (coefficient of water permeability), k_w, depend on a degree of saturation, S, (Maciel and Jucá, 2000). The coefficient of air permeability, k_a, increases when the degree of saturation, S, decreases, while the hydraulic conductivity is increased with increase in the degree of saturation, S. The air phase for an unsaturated soil generally becomes continuous as the degree of saturation, S, is reduced to approximately 85% or lower (Fredlund and Rahardjo, 1993). The flow of air through an unsaturated soil commences at this point. For higher degree of saturation, S, the level of coefficient of air permeability, k_a, is approximately zero, with air-flow reduced to a diffusion process through the pore-water (Fredlund and Rahardjo, 1993).

A number of relationships between the coefficient of air permeability, k_a, and the degree of saturation, S, have been developed. Published coefficients of air permeability relations are classified into two categories: those functional forms are the power function of air saturation and the function based on soil-water characteristic curves. Power relations include equations by Corey (1954), Wyllie (1962), and Falta et al. (1989). Relations based on the soil-water characteristic curve data constitute application of the theories by Burdine (1953) or Mualem (1976) describing the effect of the soil-water characteristic curve. Brooks and Corey (1966) applied Burdine's theory to their function of pressure head and water content, describing the soil-water characteristic curve in order to predict the coefficient of air permeability, k_a, based on the soil-water characteristic curve (referred to as Brooks-Corey model) . The following Eqs. (7.6) and (7.7) have been proposed by Brooks and Corey (1966) to describe the $k_a(S_e)$ function (i.e., Brooks-Corey expression):

$$k_a = k_d\left(1 - S_e\right)^2\left(1 - S_e^{(2+\lambda)/\lambda}\right) \tag{7.6}$$

$$S_e = \frac{S - S_r}{1 - S_r} \tag{7.7}$$

where, k_d is the coefficient of permeability with respect to the air phase for a soil at a effective degree of saturation, S_e, of zero, λ is the pore-size distribution parameter, S_e is the effective degree of saturation, and S_r is the residual degree of saturation. van Genuchten (1980) also described the $k_a(S_e)$ function as the following Eq. (7.8) (i.e., van Gunuchten-Mualem expression):

$$k_a = k_d\left(1 - S_e\right)^{1/2}\left\{\left(1 - S_e^{1/m}\right)^{2m}\right\} \tag{7.8}$$

where, m is the parameter depending on the pore-size distribution. The values of k_a at different degrees of saturation can be computed using Eq. (7.6) or (7.8) and expressed in terms of the relative coefficient of air permeability, k_{ra}, as shown in Eq. (7.9):

$$k_{ra} = \frac{k_a}{k_d} \tag{7.9}$$

The soil-water characteristic curves for two types of sludge used for the present experiment have been measured by a soil-water retention test, which is discussed in Chapter 5. Thus, the functional relationship between the air permeability and the degree of saturation, *S*, which is obtained by the present experiment, can be evaluated using the Eq. (7.6) of Brooks-Corey expression, and the Eq. (7.8) of van Genuchten-Mualem expression.

7.3. Experimental Approaches to Air Permeability of Sludge Barrier

7.3.1. Materials and Specimen Preparation

The sludge materials used in this Chapter are the same as those reported in the preceding Chapters. The basic properties of these two types of sludge are shown at Table 3.3.

The compaction procedures of PS and CS samples used for the gas permeability test are categorized into two series described as follows. In the first series, PS and CS with various molding water contents were compacted in a mould (5 cm in diameter and 3 cm in height) to achieve the same levels of dry density as that previously obtained by standard compaction curves at each molding water content. This series is to evaluate the effect of the compaction conditions of PS and CS on air permeability. In the second series, PS and CS with various molding water contents were compacted in the mould (5 cm in diameter and 3 cm in height) with specific dry density. The second series can evaluate the relationship between the degree of saturation, *S*, and air permeability under the conditions that dry density is the same.

The compacted specimen was coated with a sealing glue (silicone grease) on the lateral surface, followed by a enclosure of rubber membrane. The fine silicone grease was used to ensure the absence of sidewall flow (leakage) between the specimen and the rubber membranes. The specimen was placed on between dry porous stones and filter-papers in a modified triaxial cell for gas permeability test

7.3.2. Measurements of Air Permeability

A modified apparatus from an original triaxial compressive cell was used for measuring air permeability of compacted sludge in this experimental study. The apparatus enables application of the confining, inflow, and outflow pressures. The inflow and outflow pressures can be applied using any types of gas. The compressed air (μ = 1.82 x 10^{-5} Pa·s, ρ = 1.184 kg/m^3 at 25°C) was used as inflow gas fluid. The simplified diagram of the apparatus is shown in Fig. 7.1.

A low value of confining pressure was applied in order to obtain a perfect adhesion of the rubber membranes on the specimen. The sidewall leakage, occurring between the outer surface of the specimen and membranes, can be eliminated. The driving pressure does not exceed the confining pressure (Stylianou and DeVantier, 1995). In this test, the confining pressure was maintained at 5 kPa, exceeding the inflow pressure for each test.

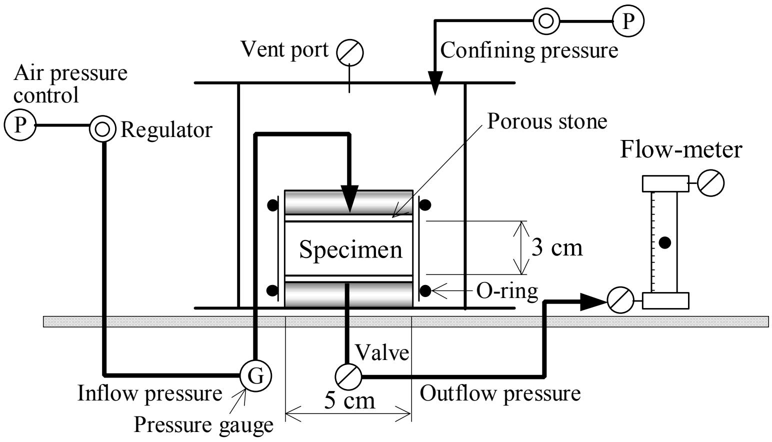

Fig. 7.1 Schematic diagram of gas permeability experimental setup

Air-flow was directed vertically downward through the specimen. The inflow pressure applied on the top of the specimen varied from 5 to 30 kPa depending on the degree of saturation, *S*, of the specimen. Low pressure of 5 kPa was applied to comparatively dried specimen, while a pressure of 10 kPa was used to the wetter specimens to reduce the testing durations. The outflow valve was connected to a variable area type flow meter at the atmospheric pressure. The variable area type flow meter is precalibrated by the manufacture to record air-flow ranging from 10 to 1000 standard cm^3/min.

7.3.3. Calculations of the Coefficient of Air Permeability

The gas transport through porous media can be described by the Hagen-Poiseuille law. Empirically the laminar, advective gas flow may be described for small pressure gradients by Darcy's law. Considering the possible changes of flow medium (gas) and temperature, the intrinsic gas permeability, K_a, describes the material-specific property of porous media to cause gas flow dependent upon pressure differences (Didier et al., 2000):

$$K_a = \frac{2q\mu\Delta x P_2}{(P_1^2 - P_2^2)} \tag{7.10}$$

where, K_a is the intrinsic gas permeability considering a compressible fluid (m^2), q is the apparent flow velocity (m/s), Δx is the height of specimen, μ is the dynamic viscosity of air at room temperature (Pa·s), P_1 is the absolute inflow pressure (Pa), and P_2 is the absolute outflow pressure (Pa). The coefficient of air permeability, k_a, is calculated using Eq. (7.11):

$$k_a = \frac{K_a \rho g}{\mu} \tag{7.11}$$

where, ρ is the density of air (kg/m^3) at room temperature and g is the gravitational acceleration (m/s^2).

7.4. Results and Discussions

7.4.1. Differential Pressure and Flow Rate

The application of Darcy's equation to the case of compressible gases shows that the gas-flow rate is not proportional to the differential pressure, ΔP, between the top and bottom of the specimen but rather to (P_1^2-P_2^2) (Bouazza and Vangpaisal, 2000). In the present experiment, the air-flow through the PS and CS specimens is measured at the outflow port which is at the atmospheric pressure, therefore $P_2 = P_{atm}$. It is important to specify at what range of differential pressure in the test should be performed, so that Darcy's equation can be used. McBean et al. (1995), and Bouazza and Vangpaisal (2000) pointed out that advective flow can be an important process, even for a differential pressure (between landfill and adjacent regions) as low as 3 kPa. They also reported that the highest gas pressure build up within a landfill can be around 8 kPa if the landfill is deep, wet, lined, and covered using the barriers. In any case, it is unlikely that the differential pressure used in the typical landfill cover would be higher than 10 kPa (Bouazza and Vangpaisal, 2000; Manassero et al., 2000). The differential pressure used in the present experiment ranges from 5 to 30 kPa. The highest pressures (more than 10 kPa) are used to verify, further, the validity of Darcy's equation.

The relationship between an increase in differential pressure and an air-flow velocity, for PS and CS specimen with molding water contents of 139.9% and 17.3%, respectively, is evaluated, in order to check the validity of applying the Darcy's equation to estimate the coefficient of air permeability, k_a. Figure 7.2 shows the result of the air-flow velocity as a function of the differential pressure applied, as well as a linear regression of the data, for PS and CS specimens with molding water contents of 139.9% and 17.3%, respectively. The proportional relations between the air-flow velocity and the differential pressure are observed for the range of differential pressures used in the present test. Consequently, Darcy's equation is applicable for the range of differential pressure used in this test.

7.4.2. Compaction Condition and Coefficient of Air Permeability

The evaluation of a coefficient of air permeability, k_a, for compacted PS and CS on their parameters for the compaction is an important issue, because of a few the research which evaluates the relationship between the coefficient of air permeability, k_a, and compaction conditions for compacted soil. On the other hand, a saturated hydraulic conductivity (i.e., coefficient of water permeability) for compacted PS and CS is strongly affected by their water content during the compaction, and the minimum hydraulic conductivity for compacted PS and CS is demonstrated at approximately 60% and 8% wetter than their optimum water

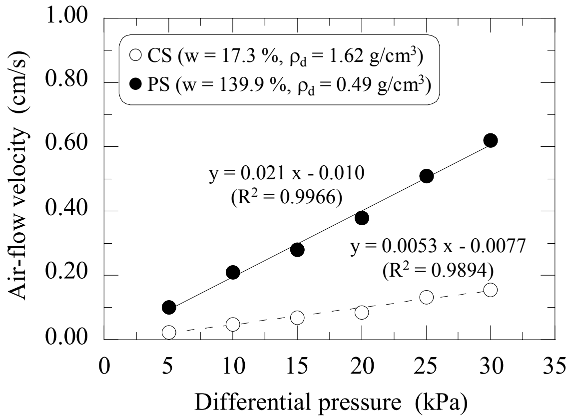

Fig. 7.2 Air-flow velocity depending on the differential pressure for (a) PS and (b) CS

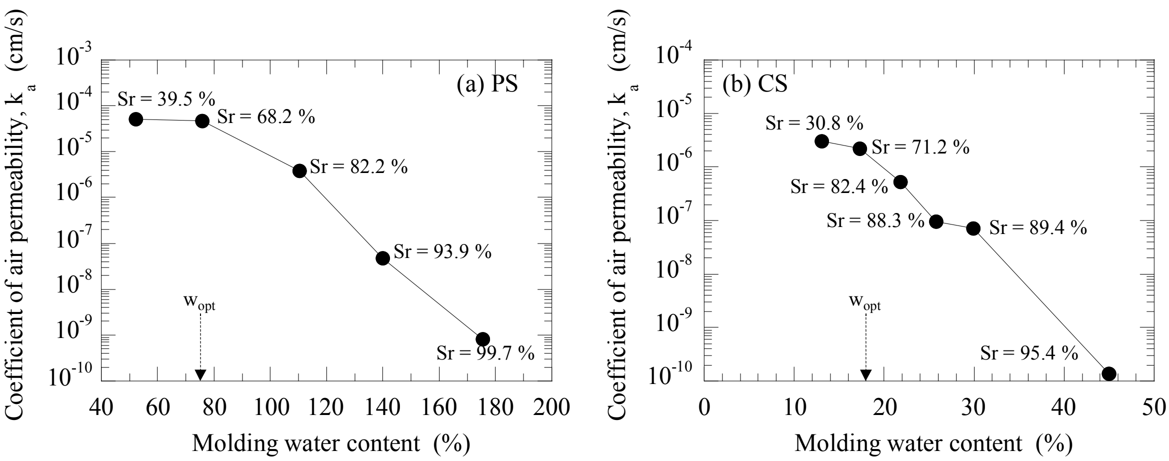

Fig. 7.3 Relationship between coefficient of air permeability and molding water content for (a) PS and (b) CS

contents, respectively. Those properties are discussed in detail in the Chapter 3.

Figure 7.3 shows the variations of the coefficient of air permeability, k_a, on the molding water contents of compacted PS and CS specimens. The compaction was carried out to achieve the same levels of dry density as the compaction curves under various molding water contents (those are corresponded to the compaction at the first series). The specimens of both PS and CS show the tendency for a coefficient of air permeability, k_a, to decrease, with increase in the molding water content. The coefficient of air permeability, k_a, tends to decrease rapidly as the optimum water content is approached, in the case of both PS and CS specimens. However, with respect to the compaction method at the first series, the dry density at the time of compaction of the PS and CS specimens decreases with the increase in the molding water content at the wet of the optimum water contents. Figure 7.4 shows the relationship between the coefficient of air permeability, k_a, and the dry density of the compacted PS and CS (the compaction method are following the first series). It can be evaluated that the coefficient of air permeability, k_a, of PS and CS specimens does not depend on an increase or a decrease in dry density.

Both Figs. 7.3 and 7.4 indicate the degree of saturation, S, in the compaction of PS and CS specimens. The coefficient of air permeability, k_a, is decreased with the increase in the degree of saturation, S, in the compaction, in both cases of PS and CS specimen. Consequently, the coefficient of air permeability, k_a, for PS and CS may not be affected by the compaction condition determined by the molding water content or dry density, but depends on the degree of saturation, S, in compaction which is determined by both water content and dry density. A later section describes the influences that the degree of saturation, S, contributes to a coefficient of air permeability, k_a.

7.4.3. Coefficient of Air Permeability and Hydraulic Conductivity

The relationships between a coefficient of air permeability, k_a, and a hydraulic conductivity (i.e., coefficient of water permeability), k_w, of PS and CS are evaluated. Generally, the coefficient of air permeability, k_a, decreases and hydraulic conductivity, k_w, increases with increasing a degree of saturation, S (Fredlund and Rahardjo, 1993).

An comparison can be made between the saturated hydraulic conductivity, k_w, which is measured in degree of saturation of approximately 100% (Kamon et al., 2001, 2002b), with the coefficient of air permeability, k_a, which is measured in degree of saturation of approximately zero. For example, the saturated hydraulic conductivity of compacted PS with the dry density of 0.55 g/cm^3 is 7.8 x 10^{-7} cm/s, while the coefficient of air permeability measured at the degree of saturation of approximately zero is 6.2 x 10^{-5} cm/s. The saturated hydraulic conductivity and coefficient of air permeability of compacted CS with the dry density of 1.48 g/cm^3 are 6.5 x 10^{-8} cm/s and 4.2 x 10^{-6} cm/s, respectively. The coefficient of

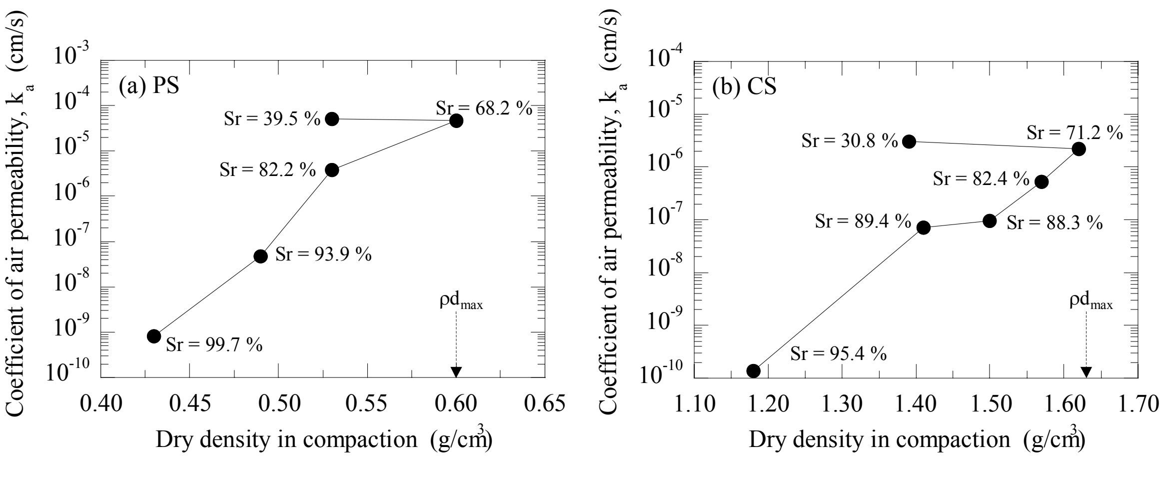

Fig. 7.4 Relationship between coefficient of air permeability and dry density in compaction for (a) PS and (b) CS

air permeability, k_a, for both types of sludge is significantly greater than the coefficient of water permeability. The difference in air and water viscosities is one of the reasons that the coefficient of air permeability is greater than the coefficient of water permeability. The coefficient of permeability is inversely proportional to the dynamic viscosity of the fluid, μ, as shown in Eq. (7.5). The dynamic viscosity of water, μ_w, is approximately 56 times the dynamic viscosity of air, μ_a, at an absolute atmospheric pressure of 101.3 kPa and a temperature of 20 °C. Assuming that the volume-mass properties of a soil do not differ for completely saturated and completely dry conditions, the saturated coefficient of water permeability would be expected to be 56 times smaller than the coefficient of air permeability at the dry condition (Koorevar et al., 1983; Fredlund and Rahardjo, 1993).

7.4.4. Degree of Saturation and Coefficient of Air Permeability

Figure 7.5 shows the coefficient of air permeability, k_a, as a function of degree of saturation, S. The plots in the Fig. 7.5 are the coefficient of air permeability, k_a, in the case where the PS and CS specimens are compacted according to their compaction curves (those are corresponded to the compaction at the first series), and those under the conditions that the dry density in compaction is constant, which are corresponded to the compaction at the second series (i.e., the dry density of PS and CS in compaction was fixed at 0.54 and 1.48 g/cm^3, respectively). There is no large difference with both compaction methods on the tendency of the coefficient of air permeability, k_a, for the degree of saturation, S, in both PS and CS. The coefficient of air permeability, k_a, is significantly reduced for both types of sludge at the degree of saturation, S, more than approximately 70%. The air-flow through the PS and CS is extremely low when the soil is reaching the saturation condition (the level of degree of saturation is more than 70%). These tendencies do not depend on the compaction method determined by the first or second series. However, at the degree of saturation, S, with same

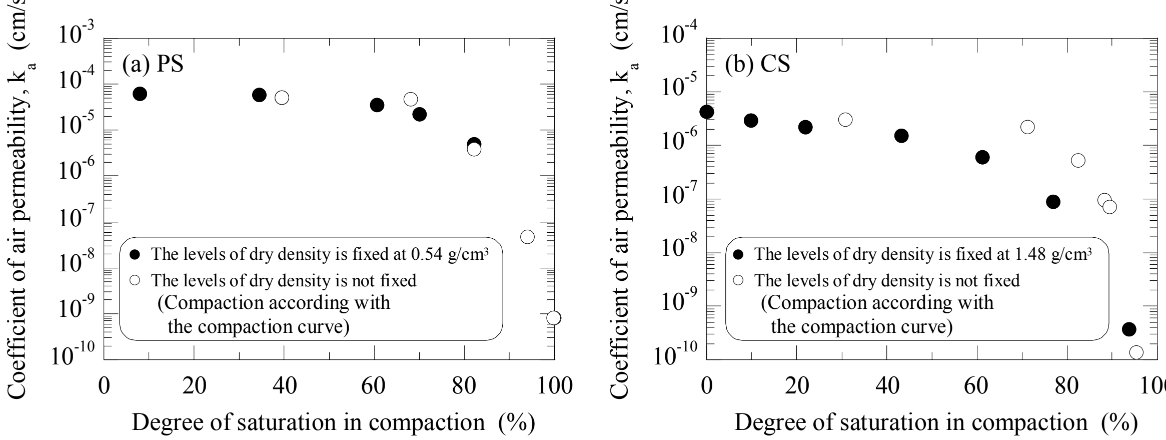

Fig. 7.5 Air coefficient of permeability as a function of degree of saturation for (a) PS and (b) CS

levels, the coefficient of air permeability, k_a, is dependent on the dry density; the specimens with larger level of dry density result in smaller level of the coefficient of air permeability, k_a. These behaviors are probably due to the presence of a discontinuous air phase in the PS and CS. In this phase, the air flows only through the pore water (as entrapped air) and small-interconnected air channels (Langfelder et al., 1968; Fleureau and Taibi, 1995).

The coefficient of air permeability, k_a, tends to decrease rapidly as the optimum water content is approached in the case of both PS and CS specimens, which are mentioned in the preceding section. Because, the compaction at the optimum water content provides the degree of saturation, S, over 70% on both PS and CS. At this point, the air phase in the specimens becomes occluded, and the flow of air takes place as a diffusion of air through water. The occluded stage for soils with a high clay content usually occurs at water contents higher than the optimum water content (Freflund and Rahardjo, 1993).

7.4.5. Predictions of Coefficient of Air Permeability on Degree of Saturation

The compatibilities of Brooks-Corey expression shown in Eq. (7.6), and that of van Genuchten-Mualem expression shown in Eq. (7.8), with the relationship between the degree of saturation, S, and coefficient of air permeability, k_a, obtained for the PS and CS, are evaluated. The PS and CS barriers, in cover system, can be predicted that the degree of saturation, S, of sludge barrier is changed from initial compaction state due to change in the weather. In evaluation of the air permeability of the sludge barriers under above conditions, if the Brooks-Corey expression or van Genuchten-Mualem expression is applicable, it will be useful.

Figure 7.6 shows the relative air permeability as a function of the effective degree of saturation, S_e, for PS and CS compacted at the same level of dry density, respectively, as well as prediction curves using the Brooks-Corey and the van Genuchten-Mualem expressions. The relative air permeability k_{ra} can be estimated using Eq. (7.9). In applying the effective degree of saturation, S_e (Eq. (7.7)) to the Brooks-Corey expression (Eq. (7.6)), and the van Genuchten-Mualem expression (Eq. (7.8)), a residual degree of saturation, S_r (*used in* Eq. (7.7)), a pore-size distribution parameter, λ (*used in* Eq. (7.6)), and a parameter depending on the pore-size distribution, m (*used in* Eq. (7.8)), are required. Normally, a soil-water characteristic curve should be used for obtaining the S_r, λ, and m. Therefore, those data are applied from the fitting results for the soil-water characteristic curves obtained by a soil-water retention test as discussed in Chapter 5. In Fig. 7.6, the predicted curve using the van Genuchten-Mualem expression over-estimates the relative air permeability in both PS and CS. The Brooks-Corey expression can estimate the relative air permeability of both PS and CS at higher accuracy than that using the van Genuchten-Mualem expression. In the meantime, as discussed in Chapter 5, both the Brooks-Corey and the van Genuchten models can estimate the unsaturated water infiltration characteristic for the PS and CS at the high accuracy. If the compatibility of these two models is evaluated on the aspects of unsaturated water infiltration

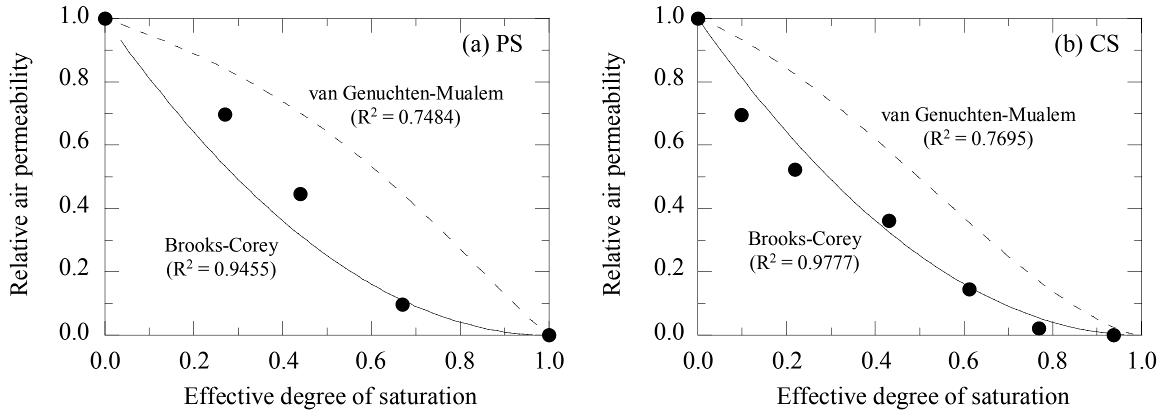

Fig. 7.6 Comparison of experimental data and fitted lines based on the Brooks-Corey and the van Genuchten-Mualem expressions for (a) PS and (b) CS

characteristic and unsaturated gas characteristics, the Brooks-Corey model is more useful especially for the estimation of gas migration properties of compacted PS and CS under unsaturated state than that using the van Genuchten expression.

7.5. Design and Performance of Landfill Gas Vent Well

7.5.1. Fundamental Design Philosophy

The landfill gas generated in waste layer is difficult to release into the atmosphere if the low gas permeability of the sludge barrier, for example, PS and CS in cover system. Accordingly, the installation of the cover system induces the increase of landfill gas pressure in the waste layer. Therefore, gas vent wells need to be installed for the purpose of reducing the increased landfill gas pressure. The gas vent wells are desirable to be arranged at an effective interval for reduction of the landfill gas pressure. The desirable for reduction of landfill gas pressure by the gas vent wells is to decrease the pressure of landfill gas to less than the loading pressure by the installation of cover system. The design of gas vent wells can be defined as presuming the magnitude of ambient landfill gas pressure, which is the primary driving force causing flow of gas, and predicting the sphere of influenced region by single passive gas vent well on reducing the pressure of generated ambient landfill gas.

7.5.2. General of Passive Gas Vent Well

The gas pressure generated inside of landfill serves as the driving force for the movement of the gas, for the passive gas vent wells. One of the most common passive methods for the control of landfill gas is based on the fact that relieving gas pressure within the landfill interior can reduce the lateral migration of landfill gas. For this purpose, the passive gas vent wells are installed through the landfill cover system extending down into the waste layer (Rowe, 2000).

The pressure of landfill gas confined within the waste layer by the cover system with low gas permeability is forced to the more porous gravel-filled passive gas vent wells, which is close to equilibrium with the atmosphere. The landfill gas, under the diffusive and advective pressure gradient, moves to this low-pressure, low-concentration area created by the vent well and from there dissipates harmlessly into the atmosphere. This process is therefore seemingly influenced by the following factors: (1) magnitude of ambient landfill gas pressure, which is the primary driving force causing flow of gas, (2) magnitude and rate of fluctuations in atmospheric pressure, which amplify or attenuate the internal landfill gas pressure gradient to the surface or to the well, (3) gas permeability of the reclaimed waste, which determines the rate at which gas can vent, (4) variations in degree of saturation of cover material and waste, which alters the gas permeability, (5) degree of gas impermeability of cover system, which influences the distance and direction of gas movement, and (6) existence of any frost and/or snow cover, which alters the gas permeability and/or hermetic integrity of the cover system (Lofy, 1996).

7.5.3. Calculation Model

The passive gas vent well is installed for reducing the pressure of landfill gas generated in waste layer. In order to evaluate the reduction effect of gas pressure by a passive gas vent well, the Dupuit's equation, which is commonly applied from an artesian well to pumping, is modified to suit to the flow of gas. This modified Dupuit's equation used in the calculation is expressed in Eq. (7.12):

$$Q = \frac{2\pi DK\left(Pe_{\max} - Pw\right)}{\mu \log_e\left(Re_{\max}/Rw\right)} \tag{7.12a}$$

$$Pe = Pw + \frac{Q\mu}{2\pi DK}\log_e\left(\frac{Re}{Rw}\right) \tag{7.12b}$$

where, Q is the gas flux which flows toward the center of a gas vent well. (m^3/s), D is the thickness of reclaimed waste layer (m), K is the intrinsic gas permeability of the waste layer (m^2), μ is the dynamic viscosity of landfill gas generated in the waste layer (Pa·s), Pe_{max} is the assumed maximum pressure (Pa) at maximum distance from the well, Re_{max} is the distance (m) out to the assumed maximum pressure point, Pe_{max}, Re is the distance from the passive gas vent well (m), Pe is the gas pressure (Pa) in the distance, Re, Pw is the pressure at the well casing (Pa), and Rw is the radius of a passive gas vent well (m).

Figure 7.7 shows an illustrative pressure of ambient landfill gas, *Pe*, at some distance, *Re*, from a passive gas vent well with a concomitant pressure decrease in the immediate vicinity of the vent well to an assumed pressure of zero gauge. For the purposes of defining the situation with the modified Dupuit's equation, it is assumed that beyond a certain distance, *Re*, from the vent well, the landfill gas pressure, *Pe*, is constant in all directions, is at its maximum, and is no longer measurably influenced by the vent well. Those boundary conditions are defined as $Pe_{\max}$ and $Re_{\max}$ in Eq. (7.12). It is further assumed that the pressure at the discharge point in the vent well, *Pw*, is equivalent to zero gauge pressure, which is equal to the atmospheric pressure. With these assumptions, the magnitude of the driving force for advective gas flow towards the vent well is simply the positive ambient landfill pressure value, *Pe*. The application of modified Dupuit's equation to the prediction of passive venting gas flow rates by a passive gas vent well is performed ignoring the coinciding diffusive transport of gases in response to a concentration gradient, as it is customarily assumed that flow resulting from a pressure gradient is usually far greater in magnitude than diffusive flow. It is rationalized that the predicted results, using a advective pressure model, would establish

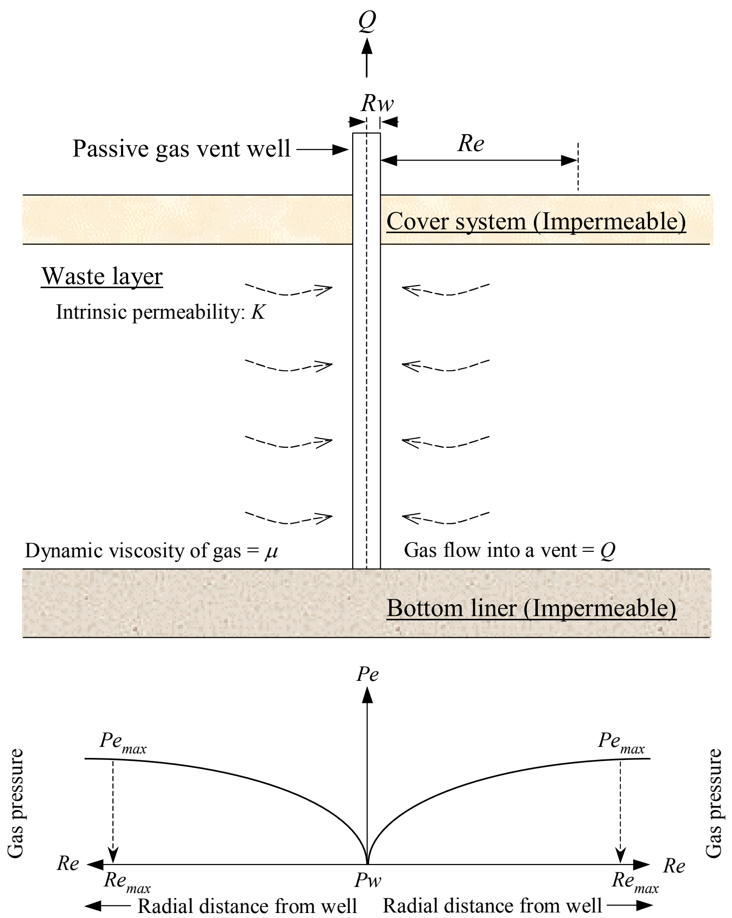

Fig. 7.7 Situation of passive gas vent well and corresponding pressure versus distance profile

the upper boundary for predicted flow, as diffusive flow is an order of magnitude slower and less important phenomenon influencing transport of landfill gas (Lofy, 1996).

Because the landfill surface without cover system is recognized as a "porous" boundary, it does not mathematically satisfy the necessity of confined condition in the modified Dupuit's equation which is developed for pumping artesian ground water. However, in the evaluation of gas pressure under the condition of installing gas-impermeable cover system, the same boundary conditions as that of modified Dupuit's equation can be achieved. The use of modified Dupuit's equation can approximate or provide some indication of how the passive gas vent well performs under varying conditions of gas pressure and the gas permeability of waste.

7.5.4. Estimation of Landfill Gas Pressure Level and Parameter Selection

The validity of the calculation results is directly related to the assumed degree of gas permeability of the cover system. For the purpose of considerable insights provided by the equation, it is assumed that the cover system is contiguous, extended beyond the zone of calculated influence of the well, and is essentially impervious.

The levels of gas pressure generated inside a waste layer provide an important parameter in the design of a passive gas vent well under installation of an impermeable cover system. The maximum magnitude of gas pressure generated in waste layer, $Pe_{\max}$, may be approximately predicted using the degree of intrinsic gas permeability of cover system which is placed on waste layer and the production rate of landfill gas in the waste layer. When the transport of the gas is assumed to be controlled in advection due to the pressure gradient, the gas pressure generated within the waste layer can be calculated as a function of the intrinsic gas permeability of the cover system. The relationship between the average gas pressure generated in waste layer under installation of cover system and the gas production rate in the waste layer is represented as Eq. (7.13):

$$\Delta P_{gas} = \frac{qDL_c}{K_c / \mu} \tag{7.13}$$

where, ΔP_{gas} is the average gas pressure generated in waste layer (Pa), q is the gas production rate in waste layer ($m^3/(s \cdot m^3)$), D is the thickness of waste layer (m), L_c is the thickness of cover system (m), K_c is the intrinsic gas permeability of cover system (m^2), and μ is the dynamic viscosity of landfill gas generated in waste layer ($Pa \cdot s$).

The gas production rate per unit volume of waste layer, q, is calculated applying the triangular gas production model proposed by Tchobanoglous et al. (1993), under assumption in which the whole quantity of biodegradable organic content in waste is converted into the gas. Although various models have been proposed for prediction of gas production rate in

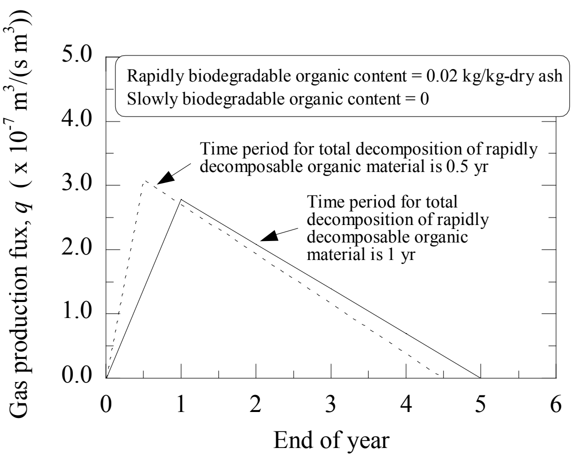

Fig. 7.8 Estimated gas production over a five-year period from the rapidly and slowly decomposable organic materials in incinerator ashes reclaimed in landfill

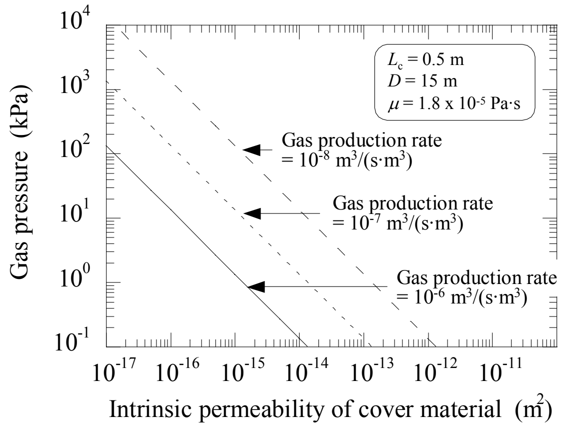

Fig. 7.9 Estimated landfill gas pressure in waste layer using the intrinsic gas permeability of waste and maximum gas production rate

waste layer, the triangular gas production model applied in this calculation has been generally used. As a calculation result using the triangular gas production model, if the kind of waste is assumed as homogeneous incinerator ashes, Fig. 7.8 shows the gas production rate from the incinerator ashes per unit volume. Furthermore, from the various trial calculations assuming the waste which various kinds mixed and the different calculation conditions, the maximum gas production rate in the various kinds of waste can be estimated as range of 10^{-7} to 10^{-5} $m^3/(s \cdot m^3)$. Using these values for Eq. (7.13), the prediction of gas pressure generated within the waste under the cover system of the layer thickness of 50cm with various levels of intrinsic gas permeability is shown in Fig. 7.9. It is found that the installation of cover system with lower gas permeability induces increase in gas pressure in waste layer.

The parameter settings used in the calculation are shown in Table 7.3. These

Table 7.3 Assumed values for each parameter and initial conditions

Waste		
Layer thickness	D	15 m
Intrinsic gas permeability	K	1 x 10^{-10} m^2
Pressure		
Atmospheric pressure		
Maximum pressure of ambient landfill gas	$Pe_{\max}$	From 0 to 20 kPa above atmospheric pressure
Gas pressure at the discharge point of a passive gas vent well	Pw	Equivalent in the atmospheric pressure
The ambient landfill gas pressure initially assumed to occur at a radial distance	Re_{max}	From 0 to 1000 m
Passive gas vent well		
Diameter of passive gas vent well	Rw	From 0.1 to 1.0 m
Landfill gas		
Dynamic viscosity of landfill gas	μ	1.8 x 10^{-5} Ps･s
The landfill surface considered to be gas impermeable to beyond the calculated zone of influence defined by radial distance.		
The waste considered to be completely homogeneous and of constant uniform permeability in the area defined by radial distance.		

parameters are referred from a number of scenario conditions looking at different landfills reported by Kjeldsen (1995), Bouazza and Vangpaisal (2000), and Manassero et al. (2000). The situation modeled in the calculation is as described in the cross-section profile shown in Fig. 7.7. Essentially, a shallow vent well is placed on the center of an assumed homogeneous section of reclaimed waste with uniform gas permeability. The pressure outside the hypothetical gas line constitutes the steady-state ambient landfill gas pressure, which is at its maximum for that are of the landfill and constitutes the driving force to propel gas towards the lower pressure vent well outlet.

7.5.5. Gas Pressure Reduction by Passive Gas Vent Well

Figure 7.10 shows the calculated pressure profiles versus distance from the center of a passive gas vent well with 30 cm in diameter, as changing the condition of assumed ambient maximum gas pressure levels, $Pe_{\max}$. When the desirable for reduction of landfill gas pressure by the gas vent well is assumed to decrease to less than the loading pressure by the installation of cover system, the level of loading pressure by the installation of cover system with PS or CS barrier can be assumed as approximately 10 kPa. In this case, the pressure caused by the generated landfill gas needs to be decreased up to less than 10 kPa using the passive gas vent wells. As noted, the distance from the center of a passive gas vent well, which can reduce pressure up to less than the loading pressure by cover system, is defined as an effective sphere of influence. As an example in Fig. 7.10, even if generated ambient maximum landfill gas pressure, $Pe_{\max}$, is assumed as 15 kPa which is more excessive pressure

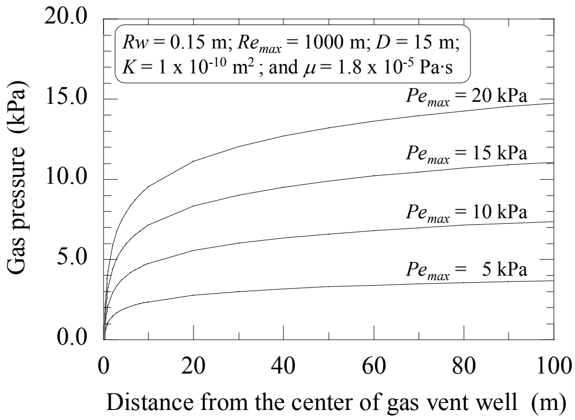

Fig. 7.10 Pressure profile beside a passive gas vent well with 30 cm in diameter

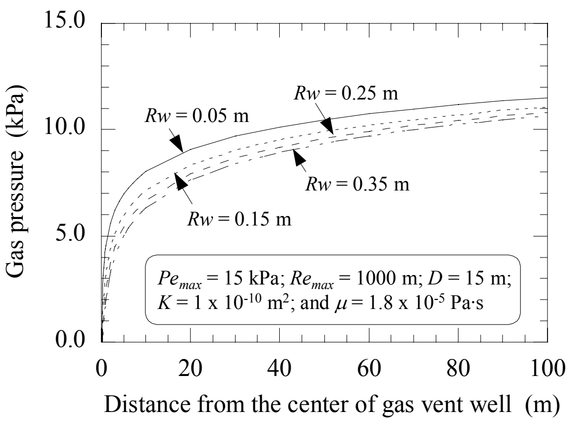

Fig. 7.11 Pressure profile beside a passive gas vent well with respective diameters

than that investigated in the field by Manassero et al. (2000), the landfill gas pressure can be decreased up to the target pressure level (i.e., 10 kPa) at the circumference of 55 m from the center of a passive gas vent with a diameter of 30 cm. In this case, the effective sphere of influence is found as 55 m. Consequently, the arrangement interval of respective passive gas vent wells should be maintained at less than 110 m, and such arrangement of the passive gas vent well can reduce the generated landfill gas pressure effectively.

The pressure profiles versus distance and the venting gas flow rates are dependent with the diameters of a passive gas vent well and the maximum pressure levels of ambient landfill gas, $Pe_{\max}$. However, the profile of pressure versus distance is unchanged by the changes the gas permeability of waste, k, and relatively unchanged by the changes in distance out to the assumed maximum pressure point with holding all other parameters constant. It suggests that the effective sphere of influence and the venting gas flow rate are largely determined by the magnitude of the diameter of a passive gas vent well, $2Rw$, and the absolute ambient maximum pressure levels, $Pe_{\max}$. Figure 7.11 shows calculated pressure profiles for a passive gas vent well with 10, 30, 50, and 70 cm in diameter, respectively. The effective sphere of influence by a passive gas vent well with 10 cm in diameter, for decreasing landfill gas pressure to less than 10 kPa, is only within the distance of 30 m while that for a well with 70 cm in diameter is 2.3 times greater. Consequently, a larger diameter of passive gas vent is effective for reduction of landfill gas pressure.

7.6. Summary and Conclusions

The coefficient of air permeability for compacted PS and CS was measured experimentally in order to estimate the air permeability of these two types of sludge. After clarifying the air permeability of PS and CS as sludge barrier in cover system, the performance of the passive gas vent well under the condition where the cover system is placed on waste layer is discussed. The influenced region by single passive gas vent well on reducing the pressure of ambient landfill gas is calculated parametrically. The conclusive results are as follows:

(1) Because of the proportional relations between the air-flow velocity and the differential pressure is observed for the range of differential pressures ranging from 5 to 30 kPa, the Darcy's equation is applicable for the range of differential pressure used in this test.

(2) The coefficient of air permeability, k_a, for both PS and CS tends to decrease, with increase in the molding water content. The coefficient of air permeability, k_a, is significantly reduced at the degree of saturation, S, more than approximately 70% for both types of sludge. The air-flow through the PS and CS is extremely low when the soil is reaching the saturation condition. These tendencies are not dependent on the compaction method.

(3) The values of the coefficient of air permeability, k_a, are significantly greater than the values of the hydraulic conductivity (coefficient of water permeability) for both types of sludge. The difference in air and water viscosities is one of the reasons for the coefficient of air permeability being greater than the coefficient of water permeability.

(4) The Brooks-Corey expression is applicable for the gas migration properties of compacted PS and CS under unsaturated state.

(5) The PS and CS barriers contribute to minimize the landfill gas migration to atmosphere, as a gas barrier.

(6) The levels of landfill gas pressure generated in the waste layer under installing the cover system can be predicted using the air permeability of cover system and modeling the gas production rate from a waste layer.

(7) The reduction objective level of gas pressure by a passive gas vent should be considered as the same level as a surcharge load by installing a cover system. When the installation of cover system with CS or PS as a barrier is assumed, the generated gas pressure can be reduced up to a objective level, within the distance of 55 m from a center of passive gas vent with 30 cm in diameter.

(8) The increase of the diameter of a passive gas vent well is more effective for reducing the gas pressure.

(9) The design concept and performance of passive gas vent well under installing the relatively impermeable cover system are clarified.

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CHAPTER 8

Conclusions and Further Research

8.1. Major Conclusions of this Dissertation

This dissertation presents an environmental geotechnical study on the waste containment technology which consists of a cover system, to minimize the infiltration of rainwater into waste, and the negative impacts of the waste leachate on the surrounding environments of landfill, is highlighted by the environmental geotechnical approaches. In particular, the significance of cover system with waste sludge barrier layer is emphasized on reducing the rainwater penetrations into landfills. Geotechnical tests are conducted systematically to evaluate the feasibility of waste sludge such as paper sludge (PS) and construction sludge (CS) as barrier material in cover system. The main results are summarized as follows.

In **Chapter 1**, while clarifying the overall backgrounds and the objectives of this dissertation, the research flow and content are presented.

In **Chapter 2**, the present status of the waste managements and landfill technologies are reviewed and a discussion is made to compare the difference in some regulations between Japan and the other countries. This provides an environmental geotechnical background for the reusing of waste sludge, such as paper sludge (PS) and construction sludge (CS), for cover material, and for emphasizing the engineering significance of a cover system in the landfill containment facilities in Japan.

In **Chapter 3**, the feasibility of PS and CS as landfill cover materials, especially the barrier layer in cover system, is explored by geotechnical and chemical evaluations. Various geotechnical efforts including hydraulic conductivity, shear strength, consolidation, and x-ray diffraction (XRD), as well as a chemical analysis of the effluents, are performed to evaluate the suitability of both PS and CS as landfill cover materials. The long-term behavior of PS and CS with respect to settlement and hydraulic conductivity is investigated by geotechnical centrifuge tests. The basic properties of PS and CS indicate their suitability for use as cover materials. The leaching properties of heavy metals from both types of sludge satisfy the environmental quality standards for soil in Japan for reusing them as cover materials. The compacted CS exhibited a hydraulic conductivity on the range of 1.2×10^{-8} to 6.5×10^{-8} cm/s, which satisfy the requirements for hydraulic conductivity of landfill covers distributed by

USEPA. In the case of compacted PS, there is no remarkable difference in the hydraulic conductivity (in the range of 3.5 x 10^{-7} to 9.0 x 10^{-7} cm/s) for a wide variation of molding water contents ranging from 50 to 150%. Compacted PS and CS maintain a hydraulic conductivity of 10^{-7} and 10^{-8} cm/s order of magnitude, respectively, under a centrifugal loading field of 60 G over 24 hours. This implies that the PS and CS can maintain such a low hydraulic conductivity as long as 9.86 years if used as barrier materials.

In **Chapter 4**, the cover systems with sludge barriers, such as PS and CS, at several sites in Japan is assumed as study models and their performance is discussed. Using water balance analysis, the effects of final and daily cover systems on reducing the quantity of rainwater percolating to the waste layers, under the humid climatic conditions in Japan, and the rainwater interception of final cover systems on the changes in the hydraulic conductivity and thickness of the sludge barrier layers are evaluated. An evaluation of the effects of the installation of daily and final cover systems on leachate reduction is also performed during the post-closure period of a landfill from the beginning of the waste reclamation. About the water balance analysis, an analytical method of the HELP model is applied, which can evaluate simply the rainwater interception ratio of daily and final cover systems. The results of the water balance analysis indicate that the sludge barrier layer with a hydraulic conductivity of 1 x 10^{-7} cm/s and a thickness more than 10 cm would intercept the rainwater infiltration into waste layer as high as 99%. For the purpose of landfill containment facilities design, the amount of the waste leachate generated with and without cover systems for an assumed landfill site is calculated. The results indicate that the absolute quantity of leachate generated in the waste layer when a daily cover system is used during the waste reclamation stage, can be decreased to 10% of that if only a surface layer is used. Furthermore, the quantity of leachate generated in the waste layer after the installation of final cover system decreases to 1.7% of cases when only a surface layer is installed. Consequently, the installation of an assumed cover system can contribute to a significant reduction in the generation of leachate in the waste layer.

In **Chapter 5**, soil-water characteristic curves for PS and CS are examined to estimate their unsaturated infiltration parameters if applied as barrier layer in cover system. The rainwater interception of each type of cover systems constituted with compacted sludge under unsaturated states is predicted using the UNSAT-H model. The obtained data from the soil-water retention test are identified by representative fitting models, such as the van Genuchten model and the Brooks-Corey model. From the soil-water characteristic curves and the fitting parameters, the effects of the molding water content and the dry density of the compacted PS and CS on the soil-water characteristic curves are clarified. The water retentiveness of PS and CS compacted at a higher molding water content is more significant, because α and n (fitting parameters for the van Genuchten model) of the PS and CS decrease with an increase in the molding water content of the sample. An evaluation of the water interception performance of the cover systems with the PS and CS barriers, under unsaturated states, shows that the cumulative quantity of percolation water through the cover systems is less than 15 cm. This equals approximately 1% of the rainfall throughout the three-year analyzing period. Although the occurrence of rainwater percolation from cover systems depends on the compaction water content and the tendency of the varying levels of precipitation at each site, over 99% of the rainfall can be intercepted by the installation of a cover system, even under the weather conditions which exist in Owase, Kyoto, and Tokyo, Japan.

In **Chapter 6**, the hydraulic performance and desiccation shrinkage (cracking) behavior of compacted PS and CS subjected to the wetting-drying cycles as sludge barrier are evaluated from modified flexible-wall hydraulic conductivity tests. Also, the long-term durability of sludge barrier under cyclic wetting and drying is evaluated by geotechnical centrifuge modeling, in which hydraulic conductivity of compacted sludge under wetting-drying cycles are measured in a centrifugal loading field with short time scale. The generating vertical depth of a crack is also predicted by analyzing the soil-water and suction distributions in the compacted PS and CS which exposed to the drying process. The durability of the compacted PS and CS as barrier layer in cover system is evaluated by calculating the changes in the suction distribution under exposing to the measured field weather conditions. The test results indicate that the molding water contents of PS and CS have a large effect on the desiccation shrinkage. Both CS and PS compacted around their optimum water contents show the smallest volumetric shrinkage strain. Shrinkage without any crack contributes to maintain the original hydraulic conductivity of compacted PS and CS. If the crack occur by desiccation shrinkage under restraint condition, however, the crack depth become about 30% of the layer thickness after 5 wetting-drying cycles in case of the PS, and the hydraulic conductivity increases to one order magnitude larger than its initial one. Consequently, it is considered that maintaining a sufficient barrier thickness, and laying a protective layer on sludge barrier so that a drying front may not develop in a sludge barrier, are important countermeasures for maintaining the durability of sludge barrier under the wetting-drying conditions. Landfill cover system that adopted the compacted PS and CS as the barrier layer should have a protective layer so that dry cracking can be prevented.

In **Chapter 7**, the coefficient of air permeability for compacted PS and CS is measured experimentally in order to estimate the landfill gas migration in these two types of sludge. After clarifying the air permeability of PS and CS as sludge barrier in cover system, the performance of the passive gas vent well under the condition where the cover system is placed on waste layer is discussed. The influenced region of a single passive gas vent well on reducing the pressure of ambient landfill gas is calculated. The coefficient of air permeability for both PS and CS tends to decrease, with increase in the molding water content. The coefficient of air permeability is significantly reduced when the degree of water saturation of both types of sludge increases more than 70%. The air-flow through the PS and CS is extremely low when the soil is reaching the saturation condition. The coefficient of air permeability is significantly greater than the hydraulic conductivity (coefficient of water permeability) for both types of sludge. The difference between air and water viscosities is responsible for the coefficient of air permeability being greater than the coefficient of water permeability. Through the air permeability tests for compacted PS and CS, it is found that the PS and CS barriers contribute to minimize the landfill gas migration to atmosphere, saving as an effective gas barrier. When the installation of cover system with CS and PS as barrier layer is assumed, a passive gas vent well with 30 cm in diameter can reduce the generated gas pressure up to a objective level within 55 m influence area. The objective levels should be considered as the same level as a surcharge load by installation of a cover system. The increase of the diameter of a passive gas vent well is more effective for reducing the gas pressure. The design concept and performance of passive gas vent well under installing the relatively impermeable cover system are clarified.

8.2. Engineering Significance of this Research

The present study discusses quantitatively the rainwater interception performance of cover system with sludge barrier, taking into consideration the geographical conditions of landfill in Japan, as well as feasibility of waste sludge materials such as PS and CS as sludge barrier. This research that mentioned clearly the functional capability of the cover system in containment facilities, will provide a geotechnical basis for installing cover system in landfill in Japan. The research on influence of the cover system on the generation and migration of landfill gas known as another environmental problems surrounding the landfill is further attempted. Several studies have been done on designing the cover system and gas collection system to control the landfill gas. Therefore, the evaluation techniques with creative experimental and analytical strategies in this research will provide good reference to the similar research followed. The engineering significance of the present study to the waste management and landfill technology can be summarized as following:

1) The research results obtained from the evaluation of the feasibility of waste sludge such as PS and CS to the cover materials, contribute to the development of effective management of wastes, so as to reuse the waste sludge in a practical way.
2) The application of the PS and CS as cover material in landfill provides another attractive selection. The utilization of PS and CS materials for the landfill cover systems will not only save landfill space, but also lower landfill costs significantly.
3) The research results prove that the infiltration of rainwater into waste layer can be considerably minimized by the installation of cover system even in Japan, where there is a monsoon climate and it is considered that the impact of rainfall on the cover systems will be much larger than that in the United States and the most European countries. The control of rainwater infiltration by the cover system is effective for reducing the generation of waste leachate; thus the significance of the cover system to the waste containment facilities for not giving negative effect to surrounding geo-environments is suggested.

8.3. Further Research Needed

The results of this research are mainly obtained from the laboratory experiments including the geotechnical centrifuge modeling, and the water balance analysis using the parameters, which are obtained from the laboratory tests. For a practical installation of the cover system with sludge barrier in landfill in Japan, validity of these results obtained from the present study should be evaluated in the field. Some subjects left for a further study to complete this research are listed as following:

1) The potential utilization of PS and CS for barrier materials in landfill cover systems has been examined. Although the long-term behavior of these two types of sludge has been evaluated based on the centrifugal simulation for the consolidation and the hydraulic conductivity, the degradation the organic materials in the sludge and its effect on the durability at least during the service life should be investigated in the future.
2) Since the cover systems are installed on the slope in many cases in order to elevate their drainage performance, the slope stability inside the compacted sludge barrier was performed. However, the slope stability along the interface between the sludge barrier and the underlying waste layer should be analyzed.
3) This research focuses on the performance of the barrier layer, which is one of the constituting layers of cover system. The whole performance of entire constituting layers of cover system should be evaluated, when a practical cover system is designed. In particular, the mechanical properties of surface layer under unsaturated conditions have to be evaluated.
4) In the United States and the most European countries, industrial products, such as GCL and GM, has applied already as cover materials. When those industrial products as alternative cover material are used in Japan, it is necessary to evaluate their validity again.
5) The compatibility between the water balance analysis and the field data has to be evaluated. It was reported that HELP model and UNSAT-H model used in this study are well compatible with the data observed in the dry region. However, those evaluations have not yet performed in the region with humid climate such as that in Japan.