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Lining of Waste Impoundment and Disposal Facilities

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16. Abstract (Limit: 200 words)

"Lining of Waste Impoundment and Disposal Facilities" provides information on performance, selection, and installation of specific liners and cover materials for specific disposal situations, based upon the current state of the art of liner technology and other pertinent technology. It contains descriptions of wastes and their effects on linings; a full description of various natural and artificial liners; liner service life and failure mechanisms; installation problems and requirements of liner types; costs of liners and installation; and tests that are necessary for preinstallation and monitoring surveys.

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LINING OF WASTE IMPOUNDMENT AND DISPOSAL FACILITIES

by

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CHAPTER 3. LINING MATERIALS AND LINING TECHNOLOGY

3.1 Introduction

The purpose of lining a waste disposal site is to prevent the potentially polluting constituents of the waste from leaving the site and entering the groundwater or surface water system in the proximity of the site. The pollutants, as discussed in Chapter 2, include organic and inorganic materials, solids, liquids, gases, and bacteriological species. In their performance liners function by two mechanisms:

a. To impede the flow (flux) of the pollutant and pollutant carrier, usually water, into the subsoil and thence into the groundwater. This requires a construction material having low permeability.

b. To absorb or attenuate suspended or dissolved pollutants, whether organic or inorganic, in order to reduce the concentrations so that they fall within the ranges set by the EPA for groundwater. This absorptive or attenuative capability is dependent largely upon the chemical composition of the liner material and its mass.

Most liner materials function by both mechanisms but to different degrees depending on the type of liner material and the waste fluid and constituent. Membrane liners are the most impermeable of the liner materials, but have little capacity to absorb materials from the waste. They can absorb a relatively small amount of organic material but, due to their small mass, their total absorption is small. Soils can have a large capacity to absorb materials of different types, but they are considerably more permeable than a polymeric membrane. However, the greater thickness of the soil can result in low flux through the liner. The choice of a particular liner material for a given site will depend upon many factors which are discussed throughout this Manual. In this chapter, the major candidates for use as liner materials are discussed.

For the purpose of this manual, we consider a liner to be a material constructed or fabricated by man. Such a definition includes soils and clays having low permeability which are (1) either brought to a site or available on the site and (2) compacted or remolded to reduce permeability and increase strength.

Liners can be classified in a variety of ways, such as construction method, physical properties, permeabiliity, composition, and type of service. These classifications are presented in more detail in Table 3-1. In this chapter, the various types of liners are discussed in the following classes:

- Soil and clays
- Admixes
- Polymeric membranes
- Sprayed-on liners
- Soil sealants
- Chemisorptive liners.

For each type of lining material, characteristics and general features of the liner and the advantages and disadvantages are discussed.

TABLE 3-1. CLASSIFICATION OF LINERS FOR WASTE DISPOSAL FACILITIES

A. BY CONSTRUCTION:

- Onsite construction:
  - Raw materials brought to site and liner constructed on site.
  - Compacted soil.
  - Mixed on site or brought to site mixed.
  - Sprayed-on liner.
- Prefabricated:
  - Drop-in polymeric membrane liner.
- Partially prefabricated:
  - Panels brought to site and assembled on prepared site.

B. BY STRUCTURE:

- Rigid (some with structural strength)
  - soil
  - soil cement
- Semirigid
  - Asphalt concrete
- Flexible (no structural strength)
  - Polymeric membranes
  - Sprayed-on membranes

C. BY MATERIALS AND METHOD OF APPLICATION:

- Compacted soils and clays.
- Admixes, e.g. asphalt concrete, soil cement.
- Polymeric membranes, e.g. rubber and plastic sheetings.
- Sprayed-on linings.
- Soil sealants.
- Chemisorptive liners.

3.2 Soils and Clay

3.2.1 Introduction

Due to their general availability, clayey soils should be considered the first alternative for a waste confinement liner. Based on engineering, environmental, and economic criteria, an initial analysis should assess whether the native soil present at a site can be used to produce an effective liner. If the result of such an analysis is negative, other alternatives must then be explored.

A soil liner is the soil material which is native at or near the waste disposal site and which has been properly treated, remolded and/or compacted so that a flow-impeding layer of low permeabliity to wastes has been obtained.

The compacted soil liner has to be designed following a proper environmental analysis. Criteria have to be set forth and the functionality of the soil liner has to be assessed. If the environmental conditions have been analyzed and understood, and the design complies with all environmental requirements, failure should not occur.

3.2.2 Fundamental Properties of Soils

3.2.2.1 Chemistry and Mineralogy of Soils and Clays

Soils to be used as liners at waste disposal facilities must contain a relatively large proportion of fines, i.e. particle size less than 2 μm. For a broad range of soils a close correlation between soil permeability and clay (less than 2 μm) content has been observed; soils exhibiting low permeability generally contain a large proportion of fines. The minimum amount of clay size particles required in soil to yield a good soil liner is 25-28% by weight.

The distinction between clay and nonclay made at particle size equal to 2 μm is based on the observation that clay minerals are heavily concentrated below this size, while nonclay minerals constitute the bulk of soil solid phases above this particle size.

This general rule deserves a more careful consideration. When examining particular clay minerals, one finds that typically each clay mineral has a preferred particle size distribution in the range less than 2 μm; since the colloidal character of a particle increases with the decrease in size, different clay minerals will have different degrees of colloidality. Also of significance is the real colloidal range, considered to be from 1 nm to 1 μm (Daniels and Alberty, 1963, p.573). Due to the relatively large surface area of the clay size fraction in a clayey soil, the mechanical behavior of such a soil is dependent on the surface physico-chemical characteristics; and since these are determined by the clay mineral chemistry, the bulk behavior of a clayey soil can be understood only if the chemistry and the mineralogy of the clay fraction are carefully considered.

Of importance in the clayey soil functioning as a liner are the changes in interlayer spacing and the consequent volume changes exhibited by different clay minerals when exposed to organic or inorganic chemicals that are likely to appear in a waste impoundment fluid. Table 3-2 presents some of the essential features of kaolinite, illite, and montmorillonite, the three clay mineral species most likely to constitute the bulk of the clay fraction of many soils. These three species are discussed briefly below.

TABLE 3-2. TYPICAL VALUES FOR PROPERTIES OF KAOLINITE, ILLITE, AND MONTOMORILLONITE[a]

Clay mineral	Particle thickness			Charge[c] per formula weight	Surface area, m^2/g	Exchange capacity meq/100g	
	Contracted (nm)	Hydrated[b] (nm)	Volume change, %			Cation	Anion
Kaolinite (nonexpansive 1:1 lattice)	200.0	202.0	1	0	8	10	pH dependent
Illite (nonexpansive 2:1 lattice)	20.0	22.0	10	1.0	80	15	pH dependent
Montmorillonite (expansive 2:1 lattice)	2.0	6.0	200	0.5	800	100	pH dependent <5

[a] Brown and Anderson, 1980, p 124.
[b] Four water layers absorbed for each available basal surface.
[c] Units are multiples of electrostatic units (esu). One charge = 4.8029×10^{-7} esu.

Kaolinite particles have a small negative surface charge. Adjacent layers of this mineral are strongly held together by hydrogen bonding. For this reason kaolinite exhibits little interlayer expansion; adsorption can occur only on external particle surfaces. Edges of the crystal exhibit broken bonds which give rise to a small number of highly pH-dependent positive or negative charges.

Illite is characterized by the presence of "fixed" potassium ions in a twelve-fold coordination between two planes of oxygen atoms. The high charge of the crystal unit combined with the perfect fit of the potassium ion in its cavity, promotes rigidity and impedes water penetration between crystal layers; consequently a limited swelling occurs. A second very important aspect of the presence of "fixed" or "nonexchangeable" potassium between successive layers is that, despite its high negative charge, i.e. its potentially high exchange capacity, the real cation exchange capacity (CEC) is only a fraction of the potential one. The real CEC of the illite is consequently intermediate between the CEC of kaolinite and that of montmorillonite.

Montmorillonite has characteristically the smallest particle size of the three clay minerals. Typically the site of the negative charge is in the inner octahedral layer in which incomplete isomorphous substitution occurs; this generates a minimal cohesion between successive 2:1 layers, resulting in a very receptive and chemically active structure. Montmorillonite readily absorbs polar organics or positively charged organic groups or inorganic ions. It exhibits the greatest surface area, CEC, and shrink-swell potential. In water, it can absorb on its interlayer surfaces 300% of its solid phase weight and consequently has the capacity for large shrinkage if the water is displaced by other fluids that yield a lower interlayer spacing.

Permeability of the three clays as a function of exchangeable calcium and sodium contents is shown in Figure 3-1. Yong and Warkentin (1975) observed that clays with divalent cations are, in general, more permeable than those with monovalent cations adsorbed onto interlayer surfaces. This effect is the same in all clays but is most pronounced in montmorillonite.

Calcium montmorillonite adsorbs interlayer water to yield a stepwise increase in basal spacing from 1 nm (oven dry state) to about 2 nm (Theng, 1979). This increase corresponds to a 1 nm thickness of water per surface when Ca-montmorillonite is fully expanded. With divalent cations such as calcium or magnesium adsorbed to its surfaces, montmorillonite resists dispersion and remains flocculated. While the flocculated state usually yields a higher permeability, its structure is more stable than the easily dispersed sodium saturated montmorillonite.

Sodium montmorillonite adsorbs interlayer water to yield a basal spacing from 1 nm (oven dry state) to over 5 nm (Theng, 1979). This represents a thickness of 4 nm for water on each surface. While this much interlayer expansion would at first appear advantageous for its ability to reduce clay liner permeability, the expansion is reversible and hence sodium montmorillonite is susceptible to shrinkage if it dries. Another problem with sodium-montmorillonite is that when it is fully expanded, it is susceptible to dispersion and internal erosion (see 4.3.3., "Piping"). A dispersed clay that lacks structural strength could flow as a viscous fluid if it is free to expand. This condition may cause problems such as loss of strength in waste impoundment side walls.

Quirk (1965) found the permeability of sodium saturated montmorillonite decreased when the concentration of NaCl in the percolating solution was decreased. A similar clay saturated with calcium showed no appreciable decrease in permeability with decreases in the calcium concentration in the percolating solution. Quirk and Schofield (1955) showed larger permeability decreases with decreasing electrolyte concentration in the percolating solution for clays with higher percentages of sodium on the exchange sites.

Blackmore and Marshall (1965) found that increasing sodium chloride concentrations in a fluid passing through a film of sodium saturated montmorillonite suppressed the double layer. The diminished double layer effect inhibited swelling while it decreased interlayer spacing and permeability.

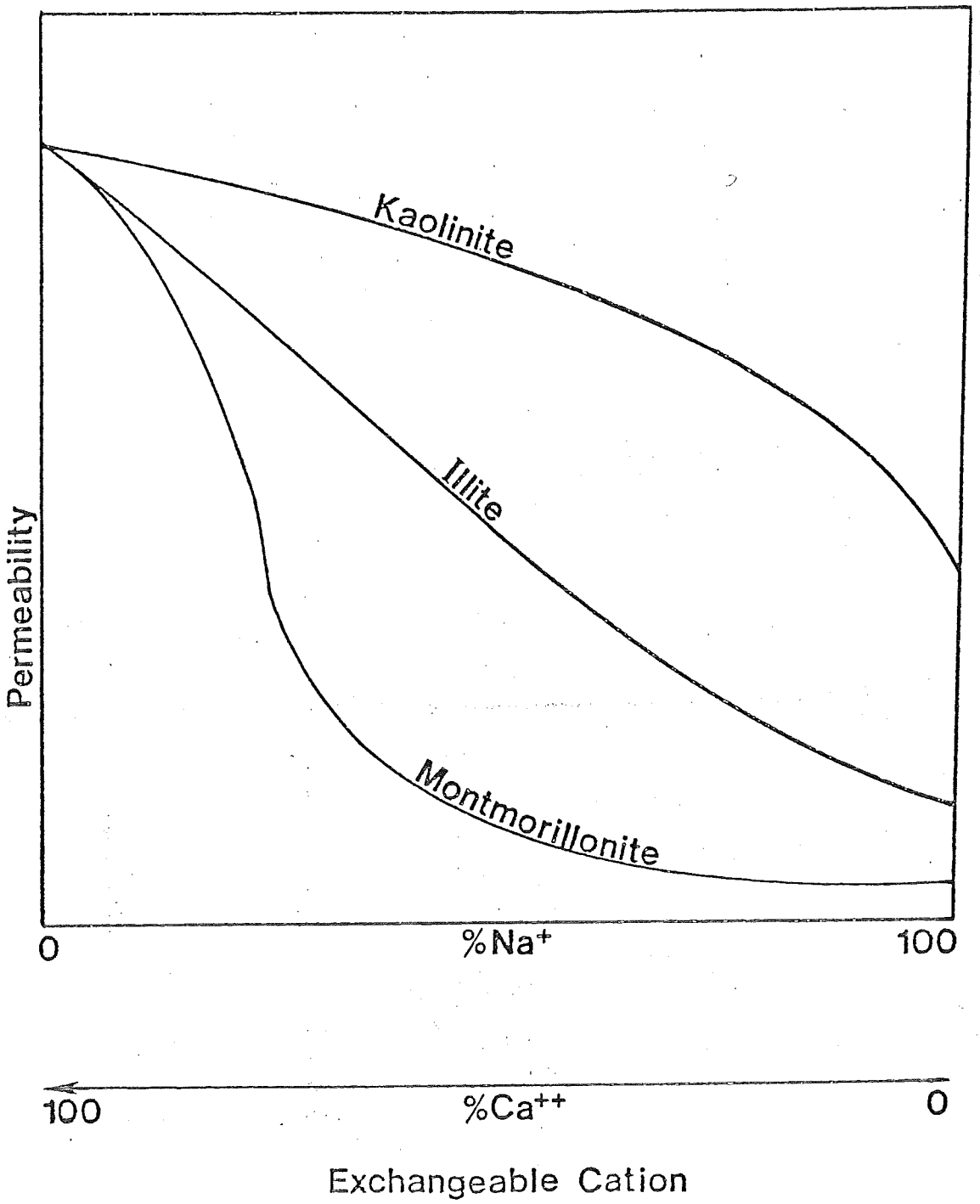

Figure 3-1 Relative permeability values for three clays with variable percentages of calcium and sodium on exchange sites (Yong and Warkentin, 1975).

3.2.2.2 Attenuation Properties of Soil Liners

A clayey soil liner below a potentially polluting waste can attentuate a pollutant by reducing the level of contamination (if present), spreading the contamination over a much longer period, and lowering considerably the maximum value of contamination, i.e. highest discharge of contaminant per unit area and time during the period of operation. The term "attenuation" at the same time implies that a soil liner cannot be an "absolute" liner. This point is illustrated schemetically by Figures 3-2 and 3-3 for which the following assumptions were made:

- The rate of contamination is defined as the amount of polluting species passing through a unit area per unit time, e.g. grams per square meter per day.

- The rate of contamination is evaluated in all cases at a depth L from the waste interface. The depth L is also taken as the thickness of the liners in cases B through E, inclusive.

- The permeability of the liners in Cases B through E, inclusive, is assumed to be the same.

- The permeability of the native soil is assumed to be greater than that of the clayey soil liners.

- The absorptive capacity for the polluting species in Case C is less than the corresponding capacity in Case D.

- The absorptive capacity of the liner in Case E is greater than the total mass loading of polluting species of the single load situation.

- The single unit load situation is defined as a set, finite amount of polluting species at a given hydraulic gradient which, over time, will approach zero as the polluting species passes through the soil and/or liner and is not replenished.

- The constant, continuous waste loading is defined as the maintenance of a given constant concentration of polluting species at a given hydraulic gradient over time. This implies replenishment of the polluting species in the waste.

- The analysis assumes saturated flow.

- The analysis assumes that the waste does not damage the liner or its structural integrity.

Shown in Figure 3-3 are rate curves for the two loading conditions defined above. Analysis of Figure 3-3 reveals that Case A, as expected, has the highest flux of pollutant contamination, reached within the shortest relative time period. In the situation of single unit loading a plateau is reached, continues for a period of time and then drops off as the amount of pollutant

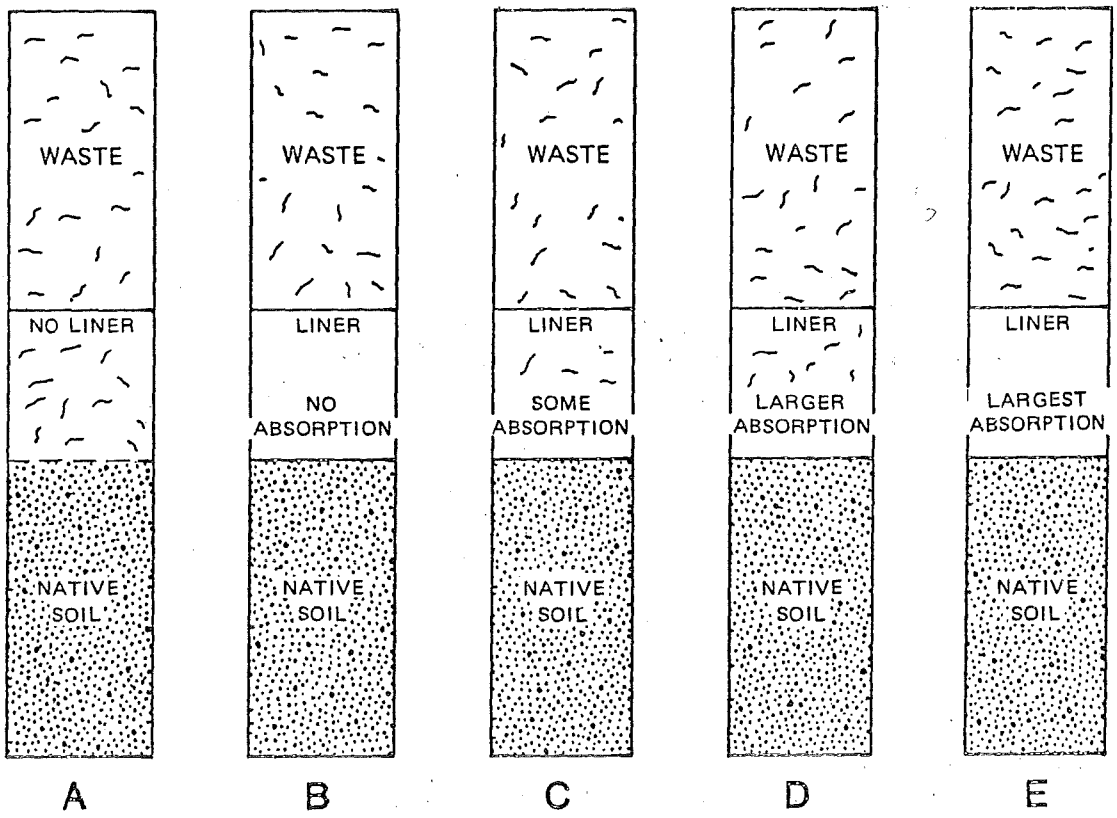

Figure 3-2. Attenuation of polluting species by soil liners of different absorptive capacities. Schematic representation of each liner situation under consideration.

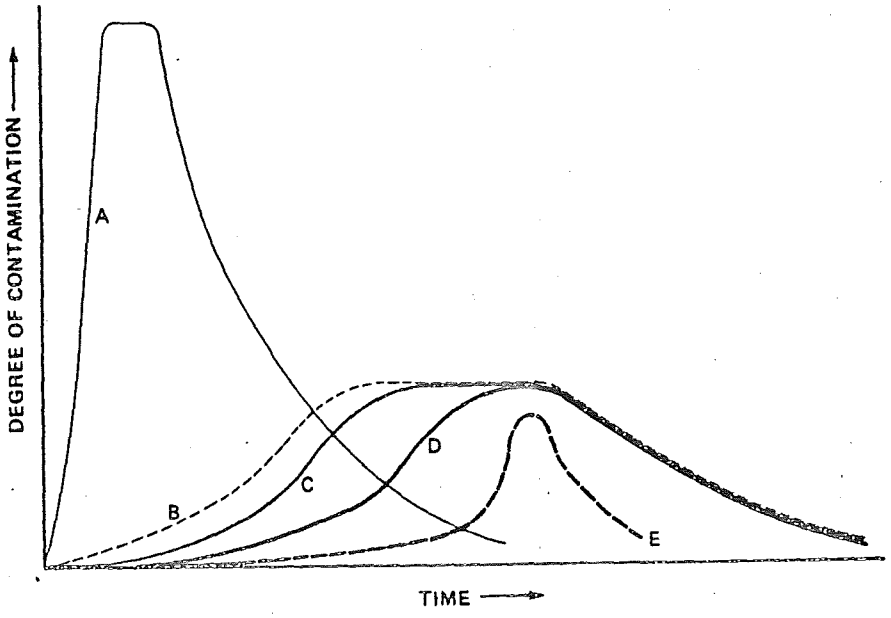

Figure 3-3. The rates of contamination of soil liners from Figure 3.2 at the depths equal to the thickness of the liners.

remaining decreases. In the situation of constant, continuous loading, the plateau is maintained due to the steady loading condition. In Case B, a liner is present which functions solely to impede flow by reduced permeability.

The distinction between A and B is one of intensity of contamination; the capacity of contamination is the same since the integrals below the two curves for times between 0 and t are assumed to be equal. The factors controlling the degree of contamination are mainly physical (flow properties) inasmuch as absorption is assumed not to affect them drastically. In drawing these two curves, one assumes that the contaminant concentration of any polluting species follows the same pattern; the flat plateau represents the situation of the maximal concentration controlled by the characteristic dissolution/precipitation reactions of the effluent. All in all, the permeability differential has the effect of diminishing the intensity factor but leaving the capacity of contamination unaltered, only a retardation effect is accomplished.

In examining Cases C and D, it can be seen that reaching the maximum rate of contamination is delayed due to the absorptive capacity of the liners. At any time prior to attaining the maximum rate plateau the difference in the rate curves reflects the magnitude of absorptive capacity of each liner. This absorptive capacity can be defined as the altering of the chemical composition of the waste fluid with respect to the polluting species such that the species is irreversibly retained in the liner. Characteristic curves like C and D will be generated depending on the degree of retention of the polluting species by the liner. If the relationship between a particular contaminant and the liner is so large that the pollutant is almost totally retained, then no plateau will exist for such a curve for single loading and a situation similar to that described for Case E will result. The characteristic curve shown by Case E reflects "leakage" in the system. In the situation where constant, continuous loading is applied, the characteristic curve for Case E after a relatively long period will approach the plateau level of Cases B, C, and D.

Further analysis of Figure 3-3 shows the rate differential of the characteristic curve plateaus for the constant, continuous loading situation. This differential, given the assumptions stated previously, is mostly a function of permeability of the soil versus the liner.

Attenuation and cation exchange capacity -

The term "attenuation" is sometimes erroneously linked to metallic trace elements through the well-known cation exchange properties of soils; this connection can be grossly misleading on at least two bases:

a. Soil cation exchange properties are revealed by <u>true exchange</u> reactions. A cation can be adsorbed and temporarily stored into the liquid-solid interface; upon the decrease of concentration in the liquid phase, it will be again released. Consequently, if an exchange would have been present in Soil B, then the degree of contamination versus time relationship should have been a mirror

image of Curve B, but slightly displaced to the right on the time scale. The picture described by Soil C does not correspond to such a situation. Curve C describes the situation in which the retention of the cation was permanent. Although these reactions are known in soil chemistry (for metallic species like K, Li, etc.), the engineer designing a waste disposal site should not rely on them.

b. For the same ionic composition and strength of waste effluent and the same activity of a particular cation, the larger the cation exchange capacity of a soil, the larger the amount of the element temporarily stored. However, this amount is so dependent on the chemistry of the effluent (its purity with regard to the particular element considered, ionic composition, strength, secondary mineral formation (precipitation/dissolution), the presence of chelating agents, complex formation with different than assumed ionic forms, etc.) that the effect of the magnitude of the exchange capacity is almost totally offset.

Modeling the functionality of complex systems has been done (Lowell, 1975). Such an analysis will require an accurate assessment of the interaction between the particular waste and soil considered. This analysis can be only obtained by setting up and performing a research program on the matter.

3.2.3 Engineering Characteristics of Soils and Clays

3.2.3.1 Atterberg Limits

Although empirical, the Atterberg limits are a powerful tool in assessing, as a first approximation, the mechanical behavior of a clayey soil. This is reflected in the emphasis on plasticity parameters in the classification of fine soils using the Unified Soil Classification System (Appendix I).

The consistency of a given soil differs with moisture content. Clayey soils, which are the types of soil required for liners, represent the most vivid example in this respect. The effect the degree of hydration of clayey soil has on its compressive strength is well known. The plastic limit and the liquid limit are tremendously valuable in engineering work for characterizing and assesing soils. The plastic limit is the soil moisture content just below which the soil is friable and just above which the soil is plastic (it can be molded as a paste which exhibits a permanent set). The liquid limit is the soil moisture content just below which the soil is barely plastic (although creep deformation can very easily occur) and just above which the soil flows (it behaves like a suspension). At the liquid limit, soil behavior is a blend of plastic deformation (tends to cease deformation upon stress removal) and liquid flow (deforms freely after stress removal).

Though the plasticity test generates results in terms of mechanical behavior, it can furnish considerable information because it is simultaneously a chemical and a physical test. It can furnish information regarding the reaction of clay and water, a reaction which is determined largely by the chemical and mineralogical properties of the clays in the soil.

The "Atterberg Limits" is the test which will help classify the soil and, consequently, will heavily weigh in the decision-making process: it will answer the question whether the waste disposal site is covered by a soil which can be improved in a soil liner. Soils which belong to the groups CL or CH should be considered the most suitable. Probably the most favorable soils are those with a liquid limit between 35 and 60, placed above the A-line in the PI versus LL chart of the Unified Soil Classification (Appendix I).

The informative power of this test can be increased by coupling it with knowledge of the clay content, i.e. the percentage of particles less than 2 μm in diameter. The concept of "activity", introduced by Skempton (1953), normalizes the plastic effect per unit weight of clay. Since this has been found to be characteristic of any particular clay (Skempton, 1953) depending on the type of clay (Mitchell, 1976), quick information regarding the type of clay in the soil can be obtained without resorting to x-ray analysis or other more quantitative mineralogical analyses.

The Atterberg limits are the result of a reaction between the clay phase of a soil and water. Consequently, the composition of both soil and the water has an affect on the plastic behavior of the soil. To bypass the difficulty of using fluid with unknown and always different composition, the ASTM methods recommend the use of distilled water which standardizes the procedure, but by so doing the methods alter the magnitude of the plastic effect. In most engineering work this procedure may not lead to significant errors. However, a soil liner may be subjected during service to very aggressive liquids, in many instances concentrated electrolyte solutions. Consequently, the use of a moderately high electrolyte solution, such as 0.01 N $CaSO_4$ rather than distilled water is recommended.

If the soil is suspected of being chemically sensitive (for instance a soil containing more than 20%, by weight, montmorillonite clay), we recommend performing Atterberg Limits by using both distilled water and 0.01 N $CaSO_4$ on duplicate samples. Eventually, an actual waste liquor can be simulated and used as a molding fluid.

The power of this test lies in the fact that it is a simple, routine, and relatively inexpensive test and, when using different fluids with different compositions, valuable information can be gained regarding to the buffer capacity of the soil, i.e. its ability to resist physico-chemical alterations and perhaps mechanical changes.

3.2.3.2 Compactibility

The properties of a soil, particularly those of importance for a soil liner, can be significantly altered by mechanical compaction. From the standpoint of liners, the most important effect of compaction is upon the permeability of the soil.

The fundamental aspects of soil compaction were established almost a half a century ago by R. R. Proctor (Burmister, 1964). It is well known that any soil has a unique laboratory density/moisture relation, i.e. Proctor curve, if compaction conditions are held constant. The relationship defines

a unique density value (maximum density) and a corresponding moisture content (optimum moisture content) which are essential parameters for design consideration.

Because construction equipment and methodology can be variable and complex, standardizing uniquely the testing conditions will not necessarily generate similar information for the field work. Thus, the modified AASHTO compaction test procedure was introduced (McDowell, 1946). Based on this test a large amount of information was collected on the effect of the field compactive effort on the density-moisture content relationship.

In respect to this relationship, it is found that increasing the compactive effort increases the maximum density and decreases the optimum moisture content. Also quite well established is the effect of soil mechanical composition on the shape and position of the density-moisture content function:

a. The more granular the soil (the more sandy it is), the higher the maximum density and the lower the optimum moisture

b. The finer the soil (the more clayey it is), the less defined the maximum density and the flatter the curve of the density as a function of moisture content.

Since the poorly defined peak on the density curve is a characteristic of soils with more than 25-30% clay and since soils containing less than this proportion of clay are tentatively excluded from construction of soil liners, it becomes quite evident that:

a. The quality control for the density achieved in the field requires considerably more attention and skill in the case of clayey soils.

b. The density obtained in the field should be only the first stage of the field testing program; other tests must be conducted.

By superimposing the conclusions of the test results on general information, the design should optimize the field compaction procedures to be used by the construction contractor.

It is well known that any increase of compactive energy, any increase in persistence of the applied energy, or any decrease of the layer thickness results in a soil with a higher density. This constitutes basic information. The test results have to quantify the interaction among these three factors.

The uniformity of moisture content of the undisturbed soil which is to be compacted, is of major significance when light rather than heavy rollers are being used. Heavy rollers seem to be more effective in compacting soil in a field of variable moisture content (Burmister, 1964). On the other hand, working the soil in the high moisture range with heavy rollers can result in the development of positive pore-water pressures, an effect which seems to be particularly true for highly plastic clayey soils (Porter, 1946). Under such compaction the pore pressure which develops in the clay cannot dissipate because of the low permeability of the clay. This aspect of the problem is

important since, for reasons which will be discussed in Section 3.2.3.4, the compaction operation in the field has to be conducted in the range of high moisture content.

Compaction which results in generation of positive pressures and consequently prevents densification is called "over compaction" (Turnbull, 1964; Holtz, 1964). To prevent its occurrence, or at least to minimize its dimension, a compaction procedure called "stage compaction" is recommended (Burmister, 1964). According to Burmister, stage compaction is the "principle of matching the supporting value and compactibility of soil layers by appropriate compactive effort, weight, and efficiency of roller for two or more stages of compaction in order to insure efficiency and effectiveness of compaction at all stages". The principal reason for recommending stage compaction is that stage compaction uses most of the compactive effort in densification rather than shear deformation and thereby will increase the efficiency of that effort. Indeed, when the strength of the compacted soil is the primary concern, shear deformation is a wasted effort; but, in our case, lateral displacement of the soil below the compacting implement is useful and is discussed further in Section 3.2.3.4. Despite its limited use (since our primary concern is flow impedance rather than strength), the designer should consider stage compaction as a strategy in compacting the soil, because any densification should yield a soil blanket with a lower permeability. In compacting highly plastic clayey soils, the stage compaction may very well be the only way to deal with this material. It has been also found that, in the case of plastic soils, the thickness of the rolled layer should be decreased (Porter, 1946).

In general, when the compaction is performed in the field, at a moisture content over the optimum, operation limitations restrict using heavy rollers; in this case the desired density has to be achieved by increasing the number of passes over the compacting layer. Some experimental results have also shown that increasing the foot-pressures used and decreasing the layer thickness in the same proportion result in equally dense soils (Sowers and Gulliver, 1955).

All this information is based on empirical considerations. Behavioral exceptions may be found and it is the role of the designer to identify and to use them advantageously.

The design specifications must define the desired compaction in terms of relative density or percent compaction and should be stated as "percentage of the laboratory maximum density". This is particularly true in the case in which the investigated area is covered by different soil types.

3.2.3.3 Volume Changes

Upon remolding and compaction a soil with a particular density can be obtained for which a set of relevant flow properties can be ascertained. Since a certain fraction of the soil bulk volume will be filled with air at the end of the compaction operation, the pore water pressure will be on the negative side, i.e. water will be held under tension in soil. The presence of a soil matrix will reduce the value of the water potential below the reference level

(pore-water potential equal to 0). As indicated by Seed et al. (1962), the basic factors controlling swelling in a soil that is not fully saturated are those responsible for generating negative pore-water pressures. As a general rule, all other factors being equal, the lower the pore pressure (the larger the absolute value in the negative range), the more prominently manifested is the swell of the soil.

The affinity of soils for water is expressed by the moisture characteristic (moisture-retention water characteristic) curve in which the equilibrium moisture retained at different suctions (negative pore-water pressures) is measured. The measurement of negative pore-water pressures in samples freshly compacted in the laboratory should provide the designer with valuable qualitative information concerning the swell/shrink properties of the soil.

Since causes of negative pore-water pressures in unsaturated soils are explained in terms of soil-matrix intrinsic properties (if the pore fluid is at a standard reference temperature and osmotic potential), the affinity of the soil for water is measured and expressed as "matric potential".

Most often soil strength is defined as the sum of a cohesional component and a frictional one, with the first component being viewed as relating to the soil and the second one being ascertained only when normal loads are operative. Similarly, one can look at positive pore-water pressures as those whose manifestation depends on environmental conditions, while the matric potential, similar to the cohesional strength, is a characteristic of the soil. Finally, one should accept the point of view that there is a relationship between the swell-behavior of soils and the resistance opposed by soils when compacted (Meade, 1964).

For design considerations, volume changes are important because they occur over time, after the placement of the soil liner. The design parameters have to be established as a result of a careful consideration of swell behavior and its influence on soil flow properties. Consequently, a prediction of the swell pressure and swell potential of the soil has to be made. A detailed development and analysis of swell behavior is presented in Appendix VI.

The effect of waste chemistry on volume changes and, consequently, on soil flow properties is discussed in Section 4.2.2.3.

3.2.3.4 Permeability

Soil permeability, K, is a numerical measure of the ability of the soil to transmit a fluid. When water is the permeating fluid through the soil, it is often called hydraulic conductivity. K can be determined using Darcy's relationship:

$$J = K\Delta H$$

where J = volume of fluid passing through unit cross sectional area of soil per unit time,

ΔH = hydraulic gradient, i.e. the rate of change of hydraulic head in the direction of flow.

When H is expressed as "weight hydraulic potential" (units of length, L) and J as volume per area times time (units of LT^{-1}), the permeability is expressed in the c.g.s. system as cm sec^{-1}.

Darcy's law is a useful relationship which has been widely used to determine K. Its validity has been questioned for very dense clayey soils because the fluid permeating the soil does not behave as a Newtonian liquid and the soil does not preserve a rigid structure (due to osmotic and seepage forces). Although the reputation of this relationship has been eroded, it still is the main relation describing flow in a clayey soil; in qualitative terms it will always be accepted, since, the larger the hydraulic gradient, the larger the resulting flux.

Because K depends on properties of both components, soil and fluid, the term "soil permeability" is not justified; to remove the effect of fluid properties, the notion of "intrinsic permeability coefficient" (IPC) is introduced; thus:

$$K' = K \frac{n'}{G}$$

where K' = IPC expressed in the c.g.s. system as cm^2

K = permeability (cm sec^{-1})

n' = kinematic viscosity (cm^2 s^{-1})

with $$n' = \frac{n}{\rho}$$

n' = viscosity (g cm^{-1} s^{-1})

ρ = density (g cm^{-3})

G = gravitational constant, 981 cm s^{-2} at 45° latitude.

It can be observed that the fluid viscosity normalizes the resistance to flow due to the fluid cohesiveness, while the fluid density normalizes the effect of gravity on fluid flow. In principle the use of IPC values permits the comparison of K values of several soil samples of the same soil, when permeated by different fluids.

At G = 981 cm s^{-2}, the relation between K and K' for water at 25°C is expressed by the equation:

$$K' = 0.91 \times 10^{-5} K$$

where K' is expressed in cm^2 and K in cm s^{-1}.

In the case of lined disposal sites, the soil liner may be contacted by fluids with very exotic chemical compositions. In this case the kinematic viscosity correction which is operated on K to produce K' is a small improvement.

The primary goal of constructing a blanket with low permeability is carried out in the field by the compaction of soil. Through this operation a dense soil material is obtained with a lower void ratio, since, according to both Taylor-Poisseuille and Kozeny-Carman relationships (Lambe and Whitman, 1979), K is proportional to $e^3 (1+e)^{-1}$ where e is the void ratio.

All other factors being constant, a reduction in the void ratio should result in a lower permeability, K. The reduction of the void ratio upon compaction promotes two changes in the soil: a decrease of the effective area available for flow (effective pore area being measured perpendicular to the flow direction), and decrease of the median pore size value. According to the capillary model, the flux (LT^{-1}) is proportional to the second power of the radius.

A very important consequence of the K-e function is that soils being particulate media and K being always measured on specimens for which the sample size is much greater than average particle size, K will always be positive since the condition e=0 contradicts the definition of a soil as a porous medium. Consequently, truly impermeable soils (K=0) do not exist.

Most undisturbed soils have permeabilities in the range of 10^{-7} cm s^{-1} to 10^{-3} cm s^{-1}. Intrinsic soil characteristics as well as naturally occuring environmental conditions play a tremendous role in the resulting broad range of permeabilities encountered in nature. If one looks over the whole range of permeability values of undisturbed soil, the particle size characteristics seem to be the most relevant; soils with more than 25-30% clay size particles are concentrated in the lower range of permeabilities, i.e. 10^{-7} cm s^{-1} to 10^{-5} cm s^{-1}.

However, if one tries to correlate K with the percentage of clay size particles over that narrow range of permeabilities, the relation between particle size and permeability becomes less significant; i.e. other factors become relatively more significant in their effect upon flow properties. For one thing, the types of clay minerals present in the clay fraction and, the particle size distribution in the less than 2 μm fraction, play a very important role. The interlayer chemistry of the crystal-unit, the susceptibility of the particles to disperse or flocculate upon hydration and/or mechanical remolding and the average size of a typical tactoid (agglomerate of clay particles), all are factors which have a profound effect upon soil-water flow characteristics; they can alter the K value by as much as two orders of magnitude for otherwise apparently very similar soils. Since this can make the difference between using or not using a particular soil as a liner, the site-designing group has to obtain pertinent information regarding these factors.

The physico-chemical behavior of a clayey soil has such an overwhelming effect upon soil permeability because of the dependence of soil clay structure on physico-chemical properties and the effect of the structure on permeability.

If a soil clay fraction would have a fixed structure, totally insensitive to changes in hydration conditions and stresses applied, then the K versus w (water content) function would be a mirror change of the γ density versus w function with the lowest permeability corresponding to the maximum density, on the γ vs. w graph. All of the physico-chemical factors mentioned have a drastic effect on clays, particularly in the case of montmorillonite clays; they have a very sensitive structure and the permeability of a montmorillonite soil is far from being a simple function of density.

Lambe (1958) showed that the permeability of compacted clays is much greater on the dry compared to the wet side of the optimum. He also concluded that the higher the compactive effort, the smaller the difference between the ranges of permeabilities achieved on both sides of the optimum. Clay compacted dry-of-optimum was found to have an "open", flocculated structure, while the wet-of-optimum tends to have a dispersed structure; this effect determines the flow properties to such an extent that almost nothing is left from the generally accepted belief that soil permeability and density are inversely related. Lambe (1958) also noted that on the dry-of-optimum side, there seems to be a threshold pressure beyond which the clay structure tends to reproduce the structure of the "wet"-clay, i.e. to orient its particles parallel to a preferred plane.

Some of these conclusions were carefully investigated in the sixties by Mitchell et al. (1965). In carrying out the investigation on a silty-clay (the more general term "clayey-soil" is often used for silty-clay) for which some of its mechanical properties were very well established, Seed and Chan (1959) confirmed the above conclusions and new effects were simultaneously revealed. Thus, combining different compactive efforts with different moisture contents to produce a unique, quite high density soil (108 lb/ft^3 or 1.732 g cm^{-3}), the permeability showed a slight increase with the water content, on the dry side of optimum.

When samples were compacted again by kneading compaction using the same compactive effort at different combinations of w and γ, the slight effect of K increase with density and moisture content in the dry-of-optimum range was still present; but when the experiment was repeated with a different soil (over a more narrow range of moisture content) the effect was not present, i.e. samples prepared dry-of-optimum indicated a "normal" drop of permeability with an increase in density.

The research conducted by Mitchell et al. (1965) indicated that when samples are compacted at a moisture content below the optimum moisture, the K versus w function depends on so many factors that, if precise information is needed, testing of the soil is the only answer; it is essential to reproduce the field conditions.

Any further increase in moisture content beyond the optimum moisture, results in a tremendous drop in K-value. Furthermore, the results presented by Mitchell et al. (1965) indicate that the choice and use of a particular compacting effort is significant to the K-value, particularly on the wet side of optimum. Using a very high moisture content and compacting their silty-clay soil using a kneading compactor at the same void ratio, the authors

obtained a large range of K-values (over two orders of magnitude) using different compactive efforts. The conclusion is that by compacting the soil at a very high moisture content the permeability can be significantly decreased by structural arrangements of soil (clay) particles rather than diminishing the total void space of the soil.

The picture presented above refers to laboratory soil specimens compacted with a kneading compactor, which has a better resemblance to the compaction of soils in the field compared to static compaction (Lambe and Whitman, 1979). It has been conclusively shown that particle preferential orientation and consequently permeability reduction upon compaction is more prominent in samples which were compacted using a kneading compactor rather than impact, vibratory, or static compaction (Seed and Chan, 1959; Mitchell et al., 1965). But it has been pointed out by Mitchell (1956) that clays, and presumably clayey soils, are quite different in their behavior upon remolding. In general, clay deposits which are believed to have been formed in marine or brackish environments, are quite efficiently remolded, i.e. a preferred orientation upon remolding can be achieved relatively easily; this conclusion seems to be particularly true for clays which have been precompressed in nature at very low stresses, e.g. the Scandinavian sensitive clays. The clay deposits, which have been sedimented in fresh waters and have been highly precompressed, e.g. the Texas and New Orleans clays investigated by Mitchell, cannot be efficiently remolded since they already have an oriented structure to start with, i.e. in their undisturbed state. Consequently, in establishing the technology to be used for producing a soil liner, the designer should have information on:

a. Natural preconsolidation pressure of the soil cover.

b. Conditions at formation or deposition of the soil considered.

This kind of information, associated with proper testing to reveal the sensitivity upon remolding of the soil, should provide the designer with the knowledge required to produce a soil-liner with a lower permeability.

Since soil compaction has been used intensively as a procedure for improving the strength characteristics of soils, the sensitivity of a soil is the ratio of the strength of the soil undisturbed to that of the soil after remolding. Seed and Chan (1959) have indicated that about the simplest test for revealing a sensitive structure is to compare the undrained stress-strain characteristics of an undisturbed soil sample with those of a sample remolded wet-of-optimum (Mitchell, 1964). Because, in the case of a soil liner, the permeability is the property to be considered and since soil flow properties have to be at least as much structure sensitive as strength properties are, the permeability rather than the strength has to be determined on undisturbed and remolded samples. Static and kneading compaction procedures should be used in parallel to evaluate the effect of void ratio and to concentrate the eventual effect that a particular structure might have on permeabilty.

The following conclusions should be observed when clayey soils are compacted to produce the lowest possible permeability:

a. The lowest permeabilities correspond, as a general rule, to the condition when the soil is compacted wet-of-optimum moisture.

b. The sensitivity of a soil-clay-tactoid structure is ascertained by increasing the available water and an available compactive effort and measuring the decrease in permeability. The testing program should reveal the relative significance of these two factors and lead to the optimization of field compaction; the investigation should identify the relative effects upon permeability of structural versus density changes when the soil is being compacted.

c. Compactive implements which promote shear deformation of soil will generate a better oriented structure and consequently help obtain a soil blanket with a low permeability (K).

d. The higher the moisture content during compaction, the more critically important is the density obtained, e.g. a small decrease in density (1%) may result in an increase in permeability by one order of magnitude.

3.3 Admixed Liners

3.3.1 Introduction

A variety of admixed or formed-in-place liners have been used in the impoundment of water. The linings include asphalt, concrete, soil cement, and soil asphalt, all of which are hard-surface materials. There has been a limited amount of experience in the use of some of the admixes in the lining of sanitary landifills and the lining of impoundments of brine. They are also undergoing exposure testing in two EPA research projects (Haxo and White, 1976; Haxo et al., 1977). In this section the following types of admixes are discussed: hydraulic asphalt concrete, soil cement, and soil asphalt. Also included is a discussion on the use of prefabricated asphalt panels for lining water reservoirs. Bentonite clay, which can be considered as an admixed material, was discussed in Section 3.2.

3.3.2 Hydraulic Asphalt Concrete (HAC) and Asphalt Panels

Hydraulic asphalt concretes, used as liners for hydraulic structures and waste disposal sites, are controlled hot mixtures of asphalt cement and high quality mineral aggregate, compacted into a uniform dense mass. They are similar to highway paving asphalt concrete but have a higher percentage of mineral fillers and a higher percentage (usually 6.5 - 9.5) of asphalt cement. The asphalt used in hydraulic asphalt cement is usually a hard grade, such as 40-50 or 60-70 penetration grade. These harder asphalts are better suited as liners than softer paving asphalt.

Hydraulic asphalt concrete can be compacted to have a permeability coefficient less than 1×10^{-7} cm s^{-1}. It is resistant to the destructive wave action of water, growth of plants, and effects of weather extremes (temperature). Such aspnalt concrete is stable on side slopes, resisting slip and creep, and

CHAPTER 4. LINING MATERIALS IN SERVICE ENVIRONMENTS

4.1 Introduction

In Chapter 2, the various wastes which are destined for land disposal are discussed with particular reference to their potential effects upon the integrity of liners. In Chapter 3, various materials which might be used for lining waste storage and disposal impoundments are described and their properties discussed. In this chapter, we discuss the environmental effects upon lining materials during service and the interaction of liners and wastes that might exist in service environments.

Considerable information exists regarding water resistance of materials of which linings are made, regardless of whether they are soils, asphalts, or polymeric membranes. However, the wastes, although most contain water, also contain many other ingredients which have varying effects upon lining materials. The pollutants, which liners are designed to prevent from entering the groundwater, are generally not the aggressive agents in the waste liquids; usually they are of relatively low concentrations. It is necessary to bear in mind the totality of all constituents in a waste in assessing a liner material for a given application; the chemical composition of both the waste and the lining material must be considered.

Although there is much information on the effects of water on lining materials, no similar amount of information exists on the effects on lining materials of chemicals and other fluids which might be found in the waste streams produced by varuous industries. Considerable information is available on the effects of chemicals and relatively simple mixtures of chemicals upon many polymeric materials that are used as containers, tank linings, pipe linings, and gaskets in direct contact with chemicals, solvents, and oils, but these polymers are selected and compounded for the specific application. Consequently, the EPA undertook several research programs to study the effects of waste liquids and chemicals on lining materials:

Sanitary Landfill Leachate (Haxo, 1976-1980)

Hazardous Wastes (Haxo, 1976-1980)

Flue Gas Desulfurization Sludges (Styron and Fry, 1979)

State-of-the-art Study of Liners (Stewart, 1978)

Field Verification of Liners (Pacey and Haxo, on-going)

Effect of Organic Chemicals on Soil (Brown, on-going)

In this chapter, we discuss the known effects of various wastes upon individual lining materials. Also discussed are the permeability of lining materials to wastes and gases.

4.2 Effects of Waste Fluids on Soils

Because soil permeability is the essential property which has to be considered in the case of a soil liner, any alterations of a soil due to the presence of a waste-leachate has to be identified. The most relevant characteristics which can alter soil flow properties are:

a. the dispersing/flocculating tendencies of the soil when contacted by the waste-leachate;

b. the alterations in the shrink/swell properties of the soil;

c. the change of pore-size distribution characteristics;

d. the dissolution/precipitation of chemical species, thus inducing an alteration of the proportion of soil volume available for flow, and

e. the modification of the adsorption properties of the soils.

Some of the possible changes will be indicated in the next paragraphs. Also included is a discussion of the effect of some physical properties of the waste-leachate on the performance of a soil-liner.

4.2.1 Discussion of Waste Fluids in Contact with Soils.

The composition and the relative quantities of different constituents in the soil solution are in general "equilibrium" values. The soil solution is simultaneously the cause and the effect of a particular set of properties of the soil matrix. As soon as the soil solution is changed to a foreign solution (waste-leachate) the soil itself will tend to develop new properties. This change is important since it allows us to assess the performance of the soil liner in time.

4.2.2 Waste Fluids

To determine the effect of a specific waste on the permeability of a specific clay soil liner, two unique fluids must be investigated. These are the flowable liquid constituents of the waste and the flowables generated from percolating water leaching through the waste.

A waste's flowable constituents, hereafter referred to as the primary leachate, includes both fluids in the waste (the solvent) and all components dissolved in these fluids (solutes). The primary leachate of a waste depends on the composition of the waste and may be aqueous-organic, aqueous-inorganic, organic, or sludges as discussed in Chapter 2.

Flowables generated from percolating water are composed of water (the solvent) and all the components dissolved in this water (solutes). This flowable mixture, hereafter referred to as the secondary leachate, may be aqueous-organic or aqueous-inorganic, depending on the waste's composition.

The dominant liquid or solvent phase of a waste fluid may be water or any organic fluid. The solute phase in a leachate are those chemicals that dissolve in the solvent phase. For example, in organic acids and bases, the acid or base may or may not be the solvent phase while in inorganic acids and bases, water is usually the solvent phase. Both primary and secondary waste fluids can be divided into a solvent phase and a solute phase. While the solvent phase will usually be the dominating factor controlling the leachate effect on the permeability of a clay soil liner, the solute phase also has the capacity to greatly affect clay permeability.

4.2.3 Solvent Phase of the Waste Fluids

4.2.3.1 Organic Fluid

Essentially all available literature on the behavior of organic fluids in soils relates to systems where water is the solvent (Goring and Hamaker et al., 1972). Water is viewed as the carrier fluid and the organic chemical is in trace quantities. In the case of clay liners in direct contact with concentrated organic fluids, the various equations for organic adsorption by clay minerals have limited usefulness. More relevant to organic fluids behavior in soils are basic chemical and physical properties such as dipole moment and viscosity. Table 4-1 is a list of solvent phases studied and their properties most likely to relate to their behavior in clay liners. Organic fluids placed in waste disposal facilities cover the spectrum of chemical species. The list is an attempt to select the most prevalent and representative of the fluids. The groupings of the organic fluids which will be considered are acids, bases, neutral-polar and neutral-nonpolar.

The term "organic-acid" is used to represent any organic fluid that has acid functional groups. This group of organic fluids is further divided to represent the two functional group types: phenolic and aliphatic acids. The group has the potential to be very reactive with, and mobile in, clay liners.

"Organic-base" is the group to represent any organic cation. Since these fluids are cationic, they will adsorb strongly to clay surfaces. By adsorbing to the clays, these fluids have the potential for causing volume changes in clays by changing interlayer spacings; and also of dissolving certain constituents out of the clay mineral. The two forms of organic bases represented are aromatic amines and alkyl amines.

"Neutral-nonpolar" organics are those organic fluids that have no charge and a small, if any, dipole moment. This group of fluids is further divided into aliphatic and aromatic hydrocarbons. With no charge and little dipole moment, these chemicals have the potential for moving through clay liners rapidly, and eroding the pores through which they pass, thus increasing permeability. These chemicals may also increase a clay liner's permeability by displacing water from the clay liner which may in turn cause shrinkage.

TABLE 4-1. POTENTIAL ORGANIC CHEMICALS IN WASTE FLUIDS
Relevant Physical and Chemical Properties

Organic chemicals			Temp. range of liquid state, °C		Density @ 20°C (gm/cm^3)	Viscosity @ 20°C (centipose)	Dielectric constant @ 20°C	Water solubility @ 20°C (gm/l)	Vapor pressure @ 20°C (mm Hg)	Molecular weight
Type		Name	Melting point	Boiling point						
Acid	Aliphatic	Acetic acid	17	118	1.05	...	6.2	∞	11.4	60.05
Acid	Phenolic	Phenol	43	182	1.07	...		82.0	0.2	94.11
Base	Aromatic amine	Aniline	-6	184	1.02	4.4	6.9	34.0	0.3	93.13
Base	Alkyl amide	Formamide	2.6	193	...	...	110	...	...	...
Neutral-polar	Alcohol	Methanol	-98	65	0.79	...	32.6	...	92	32.02
Neutral-polar	Aldehyde	Butyraldehyde	-99	75	0.82	...	...	...	71	72.10
Neutral-polar	Alkyl halide	Chloroform	-63	61	1.49	4.7	8.0	...	160	119.38
Neutral-polar	Ketone	Acetone	-95	56	...	20.7	...	...	0.05	...
Neutral-polar	Glycol	Ethylene glycol	-13	198	1.11	19.9	...	...	...	62.07
Neutral-nonpolar	Alkane	Heptane	-91	98	0.68	0.409	...	0.003	53.0	100.21
Neutral-nonpolar	Aromatic	Benzene	5	80	0.88	0.652	2.3	1.8	76.0	78.11
Neutral-nonpolar	Alkyl benzene	Xylene	-47	139	0.87	0.810	2.6	0.20	6.5	106.16
Neutral-nonpolar	Mixed alkane	Paraffin oil	-8	~321	0.87	~28	30	...	...	~375
Water			0	100	1.0	1	78.5	∞	17.5	18.02

"Neutral-polar" organics have no charge but exhibit stronger dipole moments than the "neutral nonpolar" organics. This group of fluids is further divided to represent its functional group types: alcohols, aldehydes, glycols, alkyl halides and ketones. Their effect on permeability may be related to their effect on the clay's interlayer spacing and on changes they cause in the pore water's surface tension.

4.2.3.2 Water

Water is a unique solvent in several ways. It has a very strong dipole moment, 1.87×10^{-18} esu, which means a high tendency of water molecules for orientation; this property makes the water a good solvent. The proportion of dissociated water molecules on the average is very low, only one in every ten million; yet this minute ionization is essential for many reactions to occur. The dielectric constant of water (the tendency of water molecules to orient to an electric or magnetic field) is quite high (approximately 80); this is the basis for adsorption of water on clay surfaces.

At 77°F water has a viscosity of almost one centipose; this is a highly temperature-sensitive property. The significance of viscosity for flow properties was shown previously, (Section 3.2.3.4).

The surface tension of the water (71.9 dyne cm^{-1} at 25°C) is responsible for the capillary effect, water retention properties, and consequent flow properties. The tension of the soil-water interface should extend only one to two molecular layers (1.6×10^{-7}cm) for homogeneous surfaces. Since clay surfaces do not comply with this condition, the effect is present at much larger distances away from the surface.

Due to the sensitivity of water properties to the presence of solutes (see next paragraphs), a clayey soil liner may shrink, swell or heave, crack or pipe. Water may also increase the hydraulic gradient that moves fluids in soil. While water might not be the solvent in the waste's primary leachate, it is always the solvent in the secondary leachate.

4.2.4 Dissolved Components in Waste Fluids

Organic chemicals are dissolved in a waste's leachate, regardless of what fluid represents the waste's solvent phase. However, the relative abundance of a given organic component of the solute will depend on what the solvent is. If, for example, the solvent is a nonpolar organic fluid, it would have a large carrying capacity for solutes such as other nonpolar organics. If the solvent is water, its carrying capacity for nonpolar organics in its solute phase would be relatively small. It should be noted, however, that many organic solutes (acids, alcohols, esters, ethers, amines, ketones) reduce the surface tension of the pore water with important repercussions on shrink-swell and flow properties.

In general, inorganic compounds, particularly electrolytes, do not change the surface tension of the pore water. The presence of inorganic solutes in water is, however, important since they alter the water-clay reaction via other mechanisms with important consequences.

Probably the most relevant effect induced by the presence of electrolytes in a soil water phase is the flocculation of the clay particles. The Schultze-Hardy rule, which applies in the case of "non-specific" electrolytes, states that the flocculation value (the minimal concentration needed to flocculate a dispersed clay soil) depends on the valence of the ions of opposite charge to that of the clay particle; the larger the charge (valence), the greater the flocculating power, and the flocculating value. Van Olphen (1963) indicates an observed flocculation value of 25-150 mmol/L for trivalent ions. For negatively charged clay colloidal particles, the flocculating power of cations decreases in the order:

$$Cs>Rb>NH_4>K>Na>Li$$

For positively charged aluminum and iron sesquioxides and broken edges of a clay structure at very low pH values, the flocculating power of anions decreases in the order:

$$F>Cl>Br>NO_3>I>CNS$$

Another important feature of the clay-tactoid structure is that upon addition of a low electrolyte concentration, lower than the flocculating value, the clay becomes quite susceptible to flocculation. This can be achieved upon addition of a polar or a low dielectric constant compound.

The flocculating capacity of electrolytes operates by depressing the thickness of the "double-layer". Different types of flocculation are recognized: face-to-face (FF), edge-to-face (EF), and edge-to-edge (EE). They result in quite dfferent technological performances; thus, in dilute clay suspensions viscosity increases when EF and EE flocculation occurs and decreases when the clay flocculates FF.

Sometimes, the addition of a second compound affects the flocculation power of the electrolyte; thus, when Na-polymetaphosphate is present in the system, the flocculating value of the first compound - the electrolyte - is drastically increased, which means a tendency of the clay to be in a dispersed state. On the contrary, situations exist in which the presence of the second electrolyte lowers the flocculating value more than would have been expected if a simple additive effect of the two electrolytes would have been operative.

From the information presented in the previous section, it should be clear that apart from soil densification by compaction, soil structural features (flocculated/dispersed structures) are important for assessing the suitability of a soil liner. In this context then, a dispersed structure should be associated with a desirable condition. At present, it is believed that one of the main reactions which promotes a dispersion of the clay is the neutralization of the positively charged sites of the clay, i.e. its "broken" edges. The aluminum present in the octahedral layer forms an insoluble salt with the anion which explains the anion over-saturation of the broken-edge and, consequently, the generation of a negatively charged edge. Therefore, the presence in the water phase of any anions which have a large valency and which will tend to react with broken-edge octahedral aluminum in the manner described above, should promote at least a partial dispersion, by deflocculating the edge-to-face (EF) structure.

Another mechanism by which a clay is dispersed is cation exchange. This occurs anytime a low hydrated cation, e.g.Ca^{2+}, is replaced by a cation, e.g. Na^{+}, for which a thick "double-layer" is characteristic.

Highly alkaline solutions and the corresponding solutes also have a dispersion effect by the same charge-reversal mechanism. This type of peptization seems to be particularly efficient in the case of kaolinite clays (Van Olphen, 1963, p. 117).

From this succinct description, it should be clear that the presence of different inorganic compounds in the secondary leachates, their concentration, and the characteristic pH should have a profound effect on the structure of the clay and, consequently, on the performance of a soil liner.

4.3 Effects of Waste Fluids on Soils - Failure Mechanisms

4.3.1 Dissolution of Clay

Dissolution of a clay liner can be brought about by an infiltrating chemical that dissolves the exposed surfaces of a pore or channel. Either organic or inorganic acids or bases may solubilize portions of the clay structure. Acids have been reported to solubilize aluminum, iron, alkali metals and alkaline earths while bases will dissolve silica (Grim, 1953). Since clay minerals contain both silica and aluminum in large quantities, they are susceptible to partial dissolution by either acids or bases.

Pask et al. (1945) boiled several clay minerals in acid and found the percent solubilization of alumina was 3% from kaolinite, 11% from illite, and greater than 33% from montmorillonite. Grim (1968) found the solubility of clays in acid "varies with the nature of the acid, the acid concentration, the acid-to-clay ratio; the temperature and the duration of treatment." He also found that the action of an acid on clay was enhanced when the acid had an anion about the same size and geometry as a clay component. This would permit even weak acids, e.g. organic acids, to dissolve clays under some conditions.

Hurst (1970) found that the permeability of geologic formations could be increased by pumping in acetic or formic acid. Johansen et al. (1951) reported flow increases for water wells following their treatment with a solution containing citric acid. Grubbs et al. (1972) found acid waste as the probable casual agent in the permeability increase of carbonate-containing minerals. X-ray diffraction studies of the four clay minerals injected with acid waste showed them to be dissolved or completely altered. Diffraction peaks showed the most variability with montmorillonite clays.

Acidization is the name used for the process of permeability increase by acid mineral dissolution. This process is widely used to increase the permeability and hence the productivity of oil wells (Sinex, 1970).

An ever present source of organic acids in waste impoundments is anaerobic decomposition by-products. These include acetic, propionic, butyric, isobutyric and lactic acid. Anaerobic decomposition will yield the carboxylic acid derivatives of whatever organic fluids are placed in the impoundment.

Material that encrusts at the base of wells used to inject waste usually consists of calcium, magnesium and iron carbonates, along with imbedded sand and clay particles. In order to remove the carbonate compounds, they must be dissolved and then held in solution against precipitation forces. Dissolution is usually accomplished with a strong acid. At this point, calcium will reprecipitate (as calcium sulfate in the presence of sulfuric acid) unless it is chelated and removed by a flowing fluid. Chelating agents effective at preventing reprecipitation of various carbonates are citric acid, tartaric acid, and glycolic acid (Anonymous, 1977).

It is well known that strongly alkaline solutions can partially solubilize silica-containing soil constituents. Nutting (1943) showed even extremely dilute solutions of alkali to be effective at removing silica from smectites by dissolution from the crystal lattice.

4.3.2 Volume Changes in the Soil

Volume changes of soils have to be understood since their occurrence can alter the assumed density and flow properties of the soil liner.

Volume changes in clay or soil liners occur when there is a change in the water content of the clay and when the soil can overcome the overburden loading constraint. Adsorption of water on external surfaces occurs with all three clay minerals if they are below saturation. For a given change in water content, the magnitude of volume change is dependent on the clay mineral type, the arrangement of the clay particles, the size of the clay particles, the surface area per unit weight of the clay, the existing moisture conditions, and on the kind and proportion of cations adsorbed to the clay. From Table 4-2, it can be seen that montmorillonite (and, to a lesser degree, illite) may cause problems associated with changes in the volume of clay liners. Two contracted lattice sheets of montmorillonite have a 2 nm thickness. The adsorption of water on montmorillonite interlayer surfaces gives this type of clay soil the potential for greater than a 200% difference in volume between the dehydrated and hydrated state. This expandable lattice property of montmorillonite is what makes its large volume changes unique. The structure of the soil liner in operation will not be the same as the one immediately following compaction. There are at least two reasons to suspect changes of the soil structure following compaction:

a. Upon removal of compacting implements the stresses applied will relax with the result that the soil will tend to rebound to a higher void ratio. Even where this process takes place without loss of water into the atmosphere, the magnitude of negative pore water pressure will, in general, increase, and the expansion process should tend toward a larger void ratio, the value of which depends on soil elastic properties, the magnitude and persistence of stresses applied during compaction,the overburden of the point-soil element considered, the void ratio achieved at the end of compaction, etc. If superimposed on this, there is also a net loss of water in evaporation, then there will be a simultaneous reduction of the original volume of the soil. The reduction in volume can result in the

TABLE 4-2. PHYSICAL AND CHEMICAL PROPERTIES OF THE CLAY SOILS

Clay mineral	Fine clay <0.2 μm		Coarse clay 2.0-0.2 μm		Clay, %	Silt, %	Sand, %	Organic matter, %	Horizon depth, in.	pH, units	Fe_2O_3, %	Cation exchange meq/100g			Base sat. %
	%	Mineral[a]	%	Mineral[a]								Total cap.	Ca^{++}	Na^{+}	
Montmorillonite (Lufkin)[b]	84	MT-1	16	QZ-1 KK-2 MT-2	48.3	29.7	22.0	0.9	7-21	4.6	.42	33.6	16.1	3.3	75
Calcium saturated Montmorillonite (Houston Black)[c]	~75	MT-1 KK-3	~25	MT-1 KK-2 QZ-3	55.5	40.1	4.4	3.0	0-16	7.7	.20	60.4	...	...	...
Kaolinite (Nacogdoches)[b]	67	KH-1 MI-3	33	KK-1 QZ-2	43.2	28.0	28.8	...	7-29	5.4	...	6.2	4.5	Trace	45
Illite (Hoytville)[d]	39	I-1 MT-2	61	I-1 QZ-2	46.7	38.0	15.2	...	9-27	7.1	...	26.6	19.0	...	86

[a]MT = Montmorillonite, QZ = Quartz, KK = Kaolinite, MI = Mica or Illite, KH = Tabular Halloysite, MV = Hydrous Mica-like, I = Illite, 1 = 40%, 2 = 10-40%, 3 = <10%.
[b]Soil Conservation Service, 1976.
[c]Davidson and Page, 1956.
[d]Blevins and Wilding, 1968.

formation of microfissures which, in turn, can greatly alter the flow properties of the soil liner while in service.

b. The compaction of a soil material into a soil liner generates, at a given set of conditions, a certain characteristic structure with a corresponding void ratio and, if the internal structure of the soil is rigid, a certain set of flow properties. During compaction the equilibrium soil structure and void ratios achieved depend on the forces which oppose soil deformation, strongly correlated with the characteristic swelling pressure of the system, i.e. soil-matrix and soil-fluid compositions. Since the fluid composition during compaction is different from the composition during service exposure, changes in swelling characteristics have to be expected, and a corresponding volume change of the soil liner can result if permitted by constraining stresses.

Based on waste-effluent chemistry data, the soil engineer has to conceive a hypothetical reaction between the soil liner and the waste effluent. The testing of the hypothesis has to be performed in the laboratory using the oedometer on remolded soil specimens. The testing should generate upon compression, void ratio values similar to that encountered in the field and/or to pressure values slightly over the expected overburden in the field. The rebound curves have to be obtained in parallel with water and waste effluent. The compression and swelling indexes have to be compared and the conclusion should produce at least a semiquantitative set of information with regard to the volume changes which may be expected during the operation of the system.

An alternative, indirect, and less expensive way of predicting the swell/shrink behavior of a soil liner while in operation is to use Atterberg Limits (Sections 3.2.3.1 and 5.2.2.1) together with percentage clay content of the soil in the empirical relationship established by Seed et al. (1964).

$$S = 3.6 \times 10^{-5} A^{2.44} C^{3.44}$$

or

$$S = 2.16 \times 10^{-3} (PI)^{2.44}$$

where: S = percent swell under a surcharge of one psi

A = activity

C = percentage clay

PI = plasticity index.

Due to the character of these relationships, they will provide only approximate information with regard to swell. Also, to serve the present purpose, the PI in waste effluent can be determined.

If the general chemistry of the waste and the characteristics of the soil indicate that no essential volume change of the soil liner will occur, then

the soils engineer should review the most preeminent elements which determine the shrink/swell characteristics of a soil system. Acccording to Mielenz and King (1955) these elements are: the kind and amount of clay minerals present in the soil, exchangeable composition, electrolyte concentration of the soil solution, particle size distribution, void size characteristics, soil structure, and opposing normal load.

In this discussion on volume-change characteristics of a soil, it was tacitly assumed that the greater the tendency of a soil to change its volume, the more drastic the changes in flow-properties can be expected. While this seems to be a logical conclusion, the quantification of such a statement is lacking at the present time, but it is certainly true for our case in which the original soil solution is changed in the long run for the waste effluent. In the general swelling of a soil, an increase in void ratio should increase soil permeability since a larger proportion of the total soils volume is available for flow. In our particular situation, this may very well not be the case, because simultaneously with the volume increase, a change in pore size distribution will occur and with it a decrease in the median pore size of the soil. This latter effect can easily offset the volume increase tendency of the soil and thus result in a lower permeability of the soil.

Ideally, a soil specimen should be treated with the waste-effluent in a realistic way, its mechanical behavior observed, and any alterations in flow properties recorded. The final argument for using a particular soil as a liner (apart from its attenuating capacity) is its resistance to the flow of waste effluent into underlying, undisturbed soil.

4.3.2.1 Shrinkage and Cracking

Extraction of interlayer water causes the shrinking and associated cracking exhibited by montmorillonitic soils (Baver et al., 1972; Grim, 1968). Cracking is a result of the clay undergoing three dimensional shrinkage. Where the rate of water extraction is not uniform, cracks will form in wetter soil (Yong and Warkentin, 1975). The water content of a clay liner may change if an organic leachate displaces the water from the clay liner. To understand more fully how much water will be displaced by what organic fluids, it is necessary to first understand the nature and forms water takes in a clay liner.

In clay liners, the clay particles are initially surrounded by multiple layers of water. The thickness of the water between adjacent montmorillonite lattice sheets will effect the plasticity, interparticle bonding, compactibility, and water movement within a clay liner. These properties change as the thickness of the interlayer water changes (Yong and Warkentin, 1975).

Examination of the forces holding water to the interlayer surfaces in montmorillonite will assist in predicting the effects of an intruding organic leachate on the interlayer spacing.

It is widely believed that water layers immediately adjacent to montmorillonite interlayer surfaces are nonliquid, hexagonally structured, and held much more strongly than water layers further out from the surface (Grim, 1968).

The thickness of this structured water varies depending on the adsorbed cation. Montmorillonite has a structured water thickness of 1 nm or four water layers per clay surface where calcium is the adsorbed cation. Sodium montmorillonite gives a structured water thickness of 0.75 nm or 3 water layers on its surfaces (Grim et al. 1945). Glaeser (1949) found the distance between dehydrated montmorillonite layers exposed to acetone was 1.25 nm and 1.51 nm where sodium and calcium, respectively, were the adsorbed cations.

These surface-bound layers of water are held strongly to the clay. They represent however only a part of the interlayer water. The water layers further out from the clay surfaces are held in place by the large polarity of water reinforced with its ability to hydrogen bond. This water chain extends back to the structured water layers anchored to the clay surface. The outer layers of water would easily be displaced by an intruding fluid. Where the replacing fluid has a low dipole moment, a decrease in interlayer spacing would probably result. If the intruding fluid had a higher affinity for the clay surface than the structured surface layers of water, an even greater decrease in interlayer spacing should be possible.

It may be possible for fluids with a high dielectric constant to displace interlayer water. High dielectric constants in a fluid tend to ionize the molecular species in the fluid. Ionization in turn increases the amount of adsorption onto clays for the ions. Asphaltenes and resins from the heavy end of a crude oil were found to adsorb more readily to montmorillonite when they were suspended in solvents of high dielectric constant (Clementz, 1976). A major conclusion of the Clementz study was that ionized asphaltenes in an aromatic solvent with a high dielectric constant may displace water from the clay surface irreversibly. Consequently the clay was hydrophobic and had its cation exchange capacity (CEC) reduced 52%. This CEC decrease would increase the permeability of a clay to charged or polar organic chemicals.

When a fluid adsorbs to a surface, that surface is said to be wetted by the fluid. Clay surfaces in a clay liner are water-wet initially. If a permeating organic fluid has a higher affinity for the clay surface, the clay may become organic-wet. Petroleum engineers commonly refer to surfaces with oil adsorbed as being oil-wet (Raza et al., 1968). Water and oil are said to compete with each other for solid surfaces in oil reservoirs. A quantitative measure of the preferential wettability of a clay surface for water and an organic fluid can be represented as the difference between the water-clay and organic-clay interfacial energies as represented by the Yong-Dupre equation (Adams, 1941).

$$(E_{org:c}) - (E_{w:c}) = (E_{org:w}) (\text{Cos } \theta)$$

where:

$E_{org:c}$ = interfacial energy per unit area between the organic and clay (dyn cm^{-1})

$E_{w:c}$ = interfacial energy per unit area between the water and clay (dyn cm^{-1})

$E_{org:w}$ =interfacial energy between the organic and water (dyn cm^{-1})

θ = angle at the organic:clay:water interface measured through water, (degrees).

There is no direct method for measuring either the organic:clay or the water: clay interfacial energy (Raza et al., 1968). Their difference, however, is equivalent to the product of the water:organic interfacial energy and the cosine of the water:clay:organic contact angle. This contact angle is easily calculated from capillary rise measurements of the fluids.

If an intruding organic leachate displaces the anchored water layers, there will be no forces holding the balance of the interlayer water against gravitational drainage forces. In this case montmorillonite interlayer spacing would decrease from 2 nm to whatever the interlayer spacing value would be for the newly adsorbed organic fluid. Table 4-3 is a list of the interlayer spacings for clay minerals after exposure to organic fluids. There is an abundance of x-ray diffraction data for clay minerals with correlations to the interlayer cation, dehydration temperature and the immersion fluids pH, dielectric constant, and concentration (Grim, 1968; Theng, 1974; Barshad, 1952). The usefulness of this data is limited because as a matter of standard procedure, the clay minerals are initially dehydrated. In order to simulate more closely the situation in a clay liner, there needs to be a series of x-ray diffraction studies on hydrated clay minerals. First, x-ray diffraction data is needed for the clay liner saturated with water as it would be prior to the impoundment of a waste. Secondly, x-ray diffraction data is needed for the same clay liner after organic leachates have permeated the clay. If a substantial interlayer spacing decrease is observed for the second set of x-ray diffraction data, shrinkage cracks may be anticipated for that particular liner-waste combination.

It is interesting to compare the interlayer spacings for an organic fluid at different dehydration temperatures (Table 4-3). Dehydration at higher temperatures removes more of the strongly adsorbed interlayer water. Subsequent treatment with the organic fluid yields lower interlayer spacings for the clay dehydrated at the highest temeprature. This is true to a lesser degree when sodium rather than calcium is the interlayer cation. When the clay is dehydrated at room temperture (20°C) there is still water coating its surfaces. However, both a neutral polar compound (N-butanol) and a neutral nonpolar mixture (paraffin oil) reduced the interlayer spacing of the clay dehydrated at 20°C over the interlayer spacing of the clay treated with water only after dehydration. This indicates that even compounds less attracted to a clay surface than water can displace some of the interlayer water and decrease interlayer spacing. If this scenario were to take place in a clay liner, even such common neutral nonpolar organic solvents as xylene, benzene, heptane or paraffin oil may decrease interlayer spacings and cause shrinkage cracks in clay liners.

Another factor that will influence the interlayer spacing in montmorillonite is the orientation of an organic interlayer cation. Greene-Kelly (1955) found the interlayer spacing of the clay with aniline adsorbed depended on the

TABLE 4-3 INTERLAYER SPACINGS OF CLAY ORGANIC COMPLEXES ON MONTMORILLONITE[a]

Compound	Interlayer spacing Ca sat.	Interlayer spacing Na sat.	Dielectric constant 20°C	Dipole moment	Dehydration temp., °F
Benzene	.99	.99	2.3	0	250
Benzene	1.52	.99	2.3	0	150
Methanol	1.71	...	32.35	1.66	250
Ethanol	1.70	1.34	25.00	1.70	170
Ethanol	1.70	1.35	25.00	1.70	250
n-Butanol	1.52	1.32	17.70	1.66	20
n-Butanol	1.45	1.32	17.70	1.66	170
n-Butanol	1.45	1.32	17.70	1.66	250
n-Decanol	3.68	...	5.0	1.7	250
Paraffin oil	1.45	...	2.10	0	20
Paraffin oil	9.9	...	2.10	0	250
Water	1.92	...	78.5	...	250
Methyl ethyl ketone	1.73	...	18.85	2.74	250
β-Alanine	2.5	...	150.0	...	250

[a] Barshad (1952).

concentration of aniline. Table 4-4 gives the interlayer orientation and spacing with several aniline concentrations on a montmorillonite clay.

In a waste impoundment, the affinity an organic leachate has for the hydrated clay surfaces will determine if it will displace the in-place water. Since the clay surface is negatively charged, organic leachate components that are polar or positively charged will have an affinity for clay surfaces. Since the water on the clay surface is several layers thick, water solubility will also improve an encroaching fluid's access to the clay surface. The strength with which water is held to a clay surface will vary with the cations adsorbed to the clay and the charge density of the clay. These factors may change from one place to another in a clay liner. In order to predict interlayer spacing and permeability of inplace clay liners, a series of x-ray diffraction studies

TABLE 4-4 INTERLAYER ORIENTATION AND INTERLAYER SPACING IN MONTMORILLONITIC CLAY [a]

Sorbed fluid	Orientation	Interlayer cation (meq/g)	Interlayer spacing (nm)
Aniline	Upright	Na (SAT)	1.50
H_2O	Flat	Aniline (0.62)	1.28
H_2O	Upright	Aniline (0.91)	1.42
H_2O	Upright	Aniline (SAT)	1.50
H_2O	Upright	Aniline (SAT)	1.52

[a] From Greene-Kelly, 1955.

should be undertaken on the range of liner-waste combinations likely at a given waste disposal site. If the leachate contains organic cations, the charge density of the clay will affect the resulting interlayer spacing. Weiss (1963) found the interlayer spacing after exposure to alkylammonium ions to be 1.3 nm, 1.9 nm and 2.76 nm for montmorillonites with low, medium, and high charge density respectively.

A recent review of the interactions between montmorillonite and its interlayer water is given by Low (1979).

4.3.2.2 Swelling

Swelling behavior in clays is dependent on what clay minerals are present. Both kaolinite and illite are relatively low swelling while montmorillonite is high swelling (See Table 4-2). Kaolinite and illite show the greatest volume decrease on initial dehydration and very little swelling on subsequent rewetting (Yong and Warkentin, 1975). This characteristic would limit the capability for these two clays to "self heal" or to close shrinkage cracks once they form. Montmorillonite clays exhibit both large and reversible volume changes under normal conditions. However Gieseking (1939) found that montmorillonite lost its ability to "self heal" or swell when exposed to organic cations.

Hughes (1975) found three main mechanisms responsible for the reduction or reversal of swelling in sodium bentonite (a clay in the montmorillonite family):

a. Adsorption of organics by the clay surfaces interferes with clay-water interactions and blocks further swelling.

b. The exchange of monovalent sodium for divalent calcium on the clay surface causes a reduction and reversal in swelling.

c. Excess sodium will compete for available water with clay surfaces, causing a decrease in the thickness of the water shell surrounding the clay.

Clay structure refers to the arrangement of the plate-like clay particles. The two extremes in this structure are the flocculated or edge-to-face arrangement and the dispersed or face-to-face arrangement. Different sets of forces are acting to attract and repulse clays from each other in these structural forms. The forces acting on the clays also change with ion concentration, ion valence, dielectric constant of the fluid around the clay, temperature, size of the ions in both the hydrated and dehydrated states, pH, and the clay's anion exchange capacity (Lambe, 1958).

It should be understood that organic leachates may cause either shrinking or swelling of clay liners. Barrier (1978) reported swelling of montmorillonite clays as a result of exposure to several forms of acetonitrile, xylene, cyclopentane, alcohols, glycols, and ketones. A liner that swells and heaves may loose its integrity during heaving, or it may shrink later when water replaces imbibed organic fluids.

A clay's potential for volume change can be significantly related to its Atterberg limit values. Table 4-5 shows these relationships for clays of three swell potentials (Holtz and Gibbs, 1956). However, an actual quantification of a clay's swell potential would be much more complex. Some of the factors listed by Bowles (1979) as effecting swell behavior in soils were clay type, surcharge load, void ratio, method of saturation, and general environmental conditions.

Table 4-5. SWELL POTENTIAL VERSUS ATTERBERG LIMIT VALUES IN THREE CLAY SOILS

	Swell potential		
	Low	Medium	High
Plastic range[a]	0-30	30-50	>50
Shrinkage limit[b]	> 12	10-12	<10

[a] The range of water contents below which the clay will behave as a fluid and above which it loses its cohesiveness.

[b] The water content of a clay below which no further shrinkage occurs.

It has been suggested that potential swelling is mineralogically determined, and that exchangeable cations "perturb" rather than dominate the swell-shrink characteristics (Ravina and Low, 1972).

Soil volume-change represents a very important potential mechanism of failure of a soil liner.

4.3.3 Piping

Underseepage as the result of soil piping is an ever present danger in earthen dams. Mansur and Kauffman (1956) describe piping as "the active erosion... pressure and the concentration of seepage in localized channels." Jones (1978) found the early stages of piping development to be associated with vertical contrasts of the structure and permeability in soils. Soil piping was also associated with shifts in a soil pore size distribution toward macropores with no corresponding change in total porosity. A reactive fluid may enlarge the surface area of a pore by dissolution of the pore wall and by the dissolution of the soil matrix between two pores. While a fluid's reactivity is reduced by its action on the pore wall, the size increase of the pore will increase the turbulent character of the flow inside the pore and consequently the erosion power of the moving fluid. In this manner, any variability in the pore size distribution of a clay liner may be magnified with time. Schechter and Gridley (1969) found that wormhole formation was the result of a reactive fluid's preferential flow in larger pores. He went on to say that a quasi-equlibrium is reached where further growth in a pore is limited by the rate of diffusion of the reactive fluid.

Seepage by reservoir water into dams has been reported to have caused dispersive piping and eventual tunneling all the way through earth dams. Tunneling was reported to occur in soils with a local permeability of 1×10^{-5} cm s^{-1}.

Differential solution and subsequent leaching, especially with calcareous sediments, was reported to result in the formation of channels, sink holes and cavities (Mitchell, 1976). In this respect dissolution (the first mentioned failure mechanism) seems to be in some circumstances a precondition for piping.

Cedergren (1967) reported that differential leaching of limestone, gypsum and other water soluble mineral components can lead to development of solution channels that get larger with time and substantially increase permeability. He warned not to underestimate the importance of minor soil and geologic details on the permeability of soil formations as they cause the majority of failures in dams, reservoirs and other hydraulic structures.

Cedergren (1967) concluded that most failures caused by seepage can be placed in two categories:

a. those that are caused by soil particles migrating to an escape exit, causing piping and erosional failures; and

b. those caused by uncontrolled seepage patterns which lead to saturation, internal flooding, and excessive seepage.

Crouch (1978) found that so called tunnels, tunnel-gullies, or pseudokarsts will develop in dispersive soils where the soil-colloid bond strengths are low compared to the energy of water flowing through the soil. He found dispersive soils or those with low structural stability have been associated with tunnel erosion throughout the world. Other factors found to be related to tunnel erosion were ESP (exchangeable sodium percentage), soil cracks, low permeability, and hydraulic gradients.

In a study of the variables effecting piping, Landau and Altsehaeffl (1977) noted a strong interaction between the chemical composition of the eroding water and compaction water content. Ion concentration seemed to have little effect on soil piping susceptibility for mixed illitic and kaolinitic clay loam compacted dry of optimum. For the same soil compacted wet of optimum, soil piping susceptibility was highly related to ion concentration in the eroding water. When low ion concentration eroding water is combined with wet-of-optimum compaction, Landau and Altschaeffl (1977) reported low resistance to internal erosion.

Piping involves the slaking of soil particles. Slaking is defined as the disintegration of unconfined soil samples when submerged in a fluid. Moriwaki and Mitchell (1977) investigated the dispersive slaking of sodium and calcium saturated kaolinite, illite, and montmorillonite. All the clays slaked by dispersion when saturated with sodium with the process proceeding faster with sodium kaolinite and sodium illite. Sodium illite swelled slightly while dispersion of sodium montmorillonite was preceded by extensive swelling. Sodium kaolinite underwent no visible swelling while dispersing. For the calcium saturated clays, illite dispersed much more slowly while the rate of dispersion increased for kaolinite and montmorillonite. Calcium kaolinite was thought to disperse faster because of its higher permeability relative to sodium kaolinite. Sodium montmorillonite was thought to disperse slowly because the large degree of swelling it underwent would lower permeability, thus slowing water entry and retarding dispersion.

Compaction has been shown to decrease the electrolyte content of expelled interlayer water (Rosenbaum, 1976). Such a lowering of fluid electrolyte concentrations in sodium-saturated clays may cause substantial swelling and dispersion (Hardcastle and Mitchell, 1974). This dispersion causes particle migrations. If there are fluid conducting pores large enough to transport these dispersed clay particles, permeability increases and soil piping may result (Aitchison and Wood, 1965).

It is important to note that piping would initiate on the underside of a clay liner where clay particles could migrate into a substrata with larger pore diameters. The soil pipe would then progress upward through the clay liner until it finds an opening into the waste impoundment. Clay particles have been shown to migrate through porous media containing less than 15% clay (Hardcastle and Mitchell, 1974). Consequently, clay liners underlain by soils containing less than 15% clay may be susceptible to soil piping.

Four laboratory tests for the determination of soil susceptibility to dispersive erosion have been developed by the U.S. Soil Conservation Service. A major conclusion of a recent symposium on soil piping was that these four

tests should be performed on soils where piping would cause unacceptable damage (Sherard and Decker, 1977). The four tests are the pinhole test, a test of dissolved salts in the pore water, the SCS dipsersion test, and the crumb test. For the test methods and extensive test data see ASTM Special Technical Publication #623.

4.3.4 Alteration of the Permeability of "as-compacted" Soil

Possible changes in the "as-designed" permeability can occur due to unpredicted volume changes taking place in the soil liner during the operation caused by piping or directly by a change in soil structure without any change of soil bulk volume.

As discussed in Section 3.2.3.2, the soil liner should be compacted at a high moisture content, relatively high density and, a dispersed clay-structure; this situation will lead to the lowest possible permeability in "as-compacted" state. In this condition, soil matric potential will have a relatively high value (in the low negative range) and the incoming fluid (waste leachate) will arrive as an unsaturated flow; therefore, the rate of wetting of the soil element will be quite low. Under these circumstances, the initial swell of the soil would be quite limited.

If, on the other hand, the waste fluid is very different in its chemical composition from the water used during compaction, the incoming solution will replace the original clay interlayer water and establish a new flocculation/dispersion equilibrium. This may lead to volume changes, the significance of which cannot be inferred from general knowledge. A rigorous testing program can partially reveal soil volume change vulnerability and complementary consequences on flow properties.

The occurrence of piping in dams is to a considerable extent connected with the sharp boundary between two very different soil materials making up the structure. If, in the case of a soil liner the underlying soil is texturally very different, a high probability of piping can exist. If, associated with this, the chemical composition of the waste fluid is such that internal erosion of the soil liner is possible, the integrity of the liner becomes questionable. However, the most relevant change resulting in the overall increase in soil liner permeability is probably the modification of soil structure.

In different parts of this Manual, particularly in Section 4.2, emphasis is placed on the importance of the soil structure as reflected by the pore size distribution. Also, the testing program discussed in Section 5.2 should reveal the chemical sensitivity of the soil, i.e. its tendency to modify its internal structure without mechanical deformation.

4.3.5 Slope Stability

When the topography of the waste-disposal site is flat, soil strength characteristics are of little consequence. However, environmental and economic criteria often prevail in the process of choosing a waste disposal site; environmentally, such a site should be placed as far away as possible from

highly populated areas, and since these are mostly flat areas; the disposal sites will be pushed into hilly dissected topographical regions; economically, a hilly region has its advantages in the sense that placing a waste in a ravine, for instance, may involve generally less earth work and result in a higher efficiency of waste-storage per unit disposal area. Consequently, in many situations, the waste-disposal site floor will be sloping and so will the soil liner. A similar situation occurs even when the floor of the waste-disposal site is flat, but the waste is buried under the ground surface. In this case the waste-disposal "site" will have a trapezoidal cross section with lateral slopes. The consolidation of the waste during the storage time can result in uncovering of slopes, relief of lateral pressures, and possibly an unstable condition. For these situations, an investigation of stress-strain and strength properties of the soil has to be conducted. The difficulties associated with the analysis of slope stability are not associated with the usual difficulty in determining properly the cohesional and frictional characteristics of the soil, but rather with a proper estimation of the characteristic hydrology of the site under operation condition; in other words, changes in the hydrology have to be estimated, average and "worst" patterns have to be identified, and all information integrated into a factor of safety using pertinent methods of analysis.

Although slope stability considerations are important, we believe that due to the vast amount of available information on this subject, the design of the slope can be done in such a way that a factor of safety larger than 1.4 - 1.5 can be generated. The problem is slightly more complicated when there is a partial replacement of soil water by waste-leachate; the design of the slope cannot be done without considering this factor.

In Section 5.2.2.5, the influence of different factors upon soil strength are presented and suggestions are made regarding the testing of the soils for strength.

The failure of a sloped soil liner can occur as a slippage of the whole compacted layer over the undisturbed soil or bedrock. Another mechanism is the creep of very cohesive saturated clays; by this mechanism tension can be generated in a slope and cracks perpendicular to the slope can enhance failure.

We define all these conditions as "failure", since they will lead to changes in the integrity of the soil liner with adverse effects upon bulk soil flow-properties.

4.3.6 Miscellaneous

There are a variety of situations that may increase the permeability of clay liners other than those discussed above. The phenomena causing the permeability increase may not be fully understood, but they are presented here for their possible usefulness in future research.

Miller et al. (1975) found that the permeability of a soil increased as water flushed out an earlier application of surfactant.

Grubbs et al. (1973) found that methyl alcohol increased the permeability of a core previously injected with oil-base wastes. He also noted the use of solvents, organic acids, surfactants, alcohols, and emulsion breakers for permeability enhancement in deep well injection operations.

Letey et al. (1962) observed for water-repellent soils an increase in the infiltration rate with time. He felt this was due to the progressive neutralization of the soils' water repellency as the depth of infiltration increased. In a later study, Miller et al. (1975) found that permeability increases with time if there is a substance in the soil that would dissolve into the water and decrease its surface tension.

Brant (1968) found an increase in the water permeability after a soil was treated with para-tert-butylcatechol. He postulated that the increase was due to the soil matrix being rendered more stable to water flow yielding a decrease in the migration of soil particles.

Watson (1968) found surfactants acted to stabilize soils against dispersion and swelling, thereby preventing a decrease in permeability values at certain surfactant concentrations.

Wolstenholme (1977) stated that solvents of low viscosity are "by their very nature" leachable and able to extract organic components from otherwise dry waste. Low viscosity would significantly increase a fluid's permeability according to Darcy's Law.

4.4 Effects of Waste Fluids on Flexible Polymeric Membrane Liners

4.4.1 Introduction

Background information obtained in the field on the effects of waste fluids on flexible membrane liners is limited. Consequently, most of this discussion of liner materials to various waste fluids will be based upon laboratory and simulated exposures. The principal experience in the field with membrane liner materials has been with water impoundment and conveyance. There also has been some experience with the impoundment of brines. The experience with waste fluids, however, is relatively recent although membrane lined impoundments for waste water have been used since the 1960's.

Use of polymeric membrane liners for water conservation was started in the 1940's and the first membranes were used for lining canals in 1948. These liners were butyl-coated fiberglass. Later, a number of water reservoirs, catchment basins, canals, and ponds were lined with butyl sheeting (A.R. Dedrick, 1980; W. S. Smith, 1980; Lauritzen, 1967). A variety of other polymeric membrane liners were also developed based upon such polymers as: polyvinyl chloride (PVC), polyethylene (PE), chlorosulfonated polyethylene (CSPE or CSM), and chlorinated polyethylene (CPE). These were all used in the conservation, collection, storage, and conveyance of water.

Use of polymeric membrane liners for lining waste disposal sites began in the early 1970's principally because their low permeability appeared to be effective for preventing the migration of toxic constituents from waste