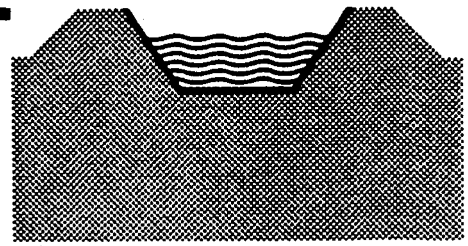
Design and Construction of Liners for Municipal Wastewater Stabilization Ponds



Alberta
ENVIRONMENT
Standards and Approvals Division
Municipal Engineering Branch

DESIGN & CONSTRUCTION OF LINERS FOR MUNICIPAL WASTEWATER STABILIZATION PONDS

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NOTE:

FOREWORD

A major responsibility of Alberta Environment is to ensure that municipal wastewaters are treated and disposed of in an environmentally acceptable manner. To meet current water quality objectives and to protect the Province's water resources requires that wastewater treatment facilities be properly and effectively designed and operated. It is essential that appropriate technologies be incorporated into the design of wastewater treatment facilities to achieve maximum benefit from pollution control expenditures.

In Alberta, lagoons are the most widely used method of treating municipal wastewaters. Lagoons can provide a high level of wastewater treatment if properly designed and operated. In the design, construction and maintenance of lagoon systems, the need to control seepage is well recognized. The purpose of this manual is to provide municipalities and engineering consultants with seepage control information to be used when planning, designing and maintaining municipal wastewater lagoon systems.

The manual outlines the seepage control criteria and the lagoon liner design standards that have been adopted by Alberta Environment for municipal wastewater lagoon facilities. The manual also outlines:

i) the investigative procedure to be followed when siting municipal wastewater lagoons;

ii) the types of linings available to control seepage;

iii) the relevant design considerations for the various types of liners; and

iv) the groundwater monitoring required to assess the effectiveness of seepage control measures.

The proper application of this manual should ensure that municipal wastewater lagoon systems do not pollute groundwaters or adversely effect adjacent lands, at the same time maintaining the necessary liquid depths to function effectively as wastewater treatment units.

Future editions of this manual will be issued as warranted by changing technologies and engineering practices.

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF MANUAL

The purpose of this document is to provide information and recommendations on the design of seepage control liners for municipal wastewater stabilization ponds (sewage lagoons) in Alberta. In addition, standards are presented for seepage control in sewage lagoons, as required by Alberta Environment. Recommendations and standards are given for preliminary site selection, site investigation procedures, selection of liner materials, liner system design and groundwater quality monitoring. Liner materials discussed in the manual include: natural undisturbed deposits; compacted clay; bentonite and sand admixes; asphalt; and flexible polymeric membranes.

The manual is intended for use by engineers and geologists familiar with the design of sewage lagoons and impoundment structures. Thus, it is expected that the user has a working knowledge of soil mechanics and earthworks design, hydrogeology, wastewater engineering, and related disciplines.

The contents of this manual are limited to seepage control and liner design for sewage lagoons, and do not deal with the principles of lagoon operation or system design; e.g., number, size, configuration and depth of lagoons; piping or mechanical systems; or retention time and effluent quality. Standards relating to these aspects of sewage lagoon design and operation are presented in the current edition of the Recommended Standards for Water Supply and Sewerage prepared by the Standards and Approvals Division of Alberta Environment.

The information and procedures presented in this document are based on recent available literature and discussions with engineers and researchers. However, it should be recognized that liner technology is a relatively new science that spans several disciplines. New materials, information, and long term performance data may render parts of the manual obsolete with time. Thus, the user is encouraged to keep up-to-date with current developments in liner technology.

Finally, it is believed that conscientious application of the information and procedures recommended herein will allow design and construction of lagoons that meet the design and performance standards of Alberta Environment. Nevertheless, good geologic interpretation of site conditions and sound engineering judgement will always play a key role in determining the success of any design.

1.2 FUNCTIONS OF SEEPAGE CONTROL LINERS

Wastewater stabilization ponds are designed to retain wastewater so that certain physical and chemical processes can occur; e.g., settlement of solids and decomposition of organics. Wastewater that seeps through the pond bottom may be discharging from the system before adequate renovation has taken place; in any case the discharge is generally uncontrolled and may have an adverse impact on surrounding groundwater, surface water, and land.

The function of a lagoon liner is to control the rate of seepage to a level that will not have an adverse impact on the environment. The seepage control liner is generally a layer of material, typically the pond invert material, that has a dominant influence on the rate of seepage; this may be a thick layer of moderately low permeability material such as clayey soil, or a thin layer of very low permeability material such as a polymeric membrane. The rate of seepage is also dependent to some degree on other factors such as the hydraulic properties of the underlying and surrounding sediments, and natural groundwater conditions.

Finite quantities of water seep through virtually all materials. Significant portions of impounded wastewater may seep through relatively porous soils, while very small quantities of wastewater pass through intact continuous materials such as polymeric membranes. However, in nearly all cases some seepage occurs and the lagoon designer should be aware of the probable seepage rate, where seepage will migrate, and what likely impact transported contaminants will have on the surrounding environment. Thus, while the liner itself is an important feature in liner design, the local hydrogeology is also important in determining the overall success of the seepage control design.

Liner materials and the surrounding hydrogeologic system (together they may be considered as the liner system) may interact to reduce the impact of seepage in several ways; that is, the liner layer reduces seepage rates, thus:

a) allowing dilution of contaminants in the natural groundwater system;

b) resulting in slower seepage velocities and contaminant migration rates, which in turn may allow die-off of pathogens and/or dilution of contaminants at the eventual discharge point;

c) reducing contaminant flux to levels that may be adequately attenuated by various physical and chemical mechanisms in the soil/groundwater system; e.g., filtration, precipitation, adsorption, chemical or biological degradation to inert forms.

In some situations multi-layered liner systems are constructed that allow total containment of the impounded liquids; these systems are very expensive and are generally only warranted for hazardous materials in sensitive environments.

It is apparent from the above discussion that the degree of renovation of seeping wastewater may depend on the distance the wastewater has travelled through the sediments underlying the liner. The perceived or allowed extent of the liner system will affect decisions regarding monitor well locations and groundwater quality standards. The Standards and Approvals Division of Alberta Environment has determined that the liner system should be designed so that seeping wastewater is renovated by the time it crosses the lagoon property boundary.

1.3 SEEPAGE CONTROL STANDARDS

Environmental regulatory agencies set seepage control standards for impoundments such as lagoons in order to protect the quality of groundwater and surface water resources in the area, and to maintain aesthetic conditions around the impoundments. There are two types of seepage control standards; those that pertain to performance and those governing the lagoon design and construction. A performance standard is generally a requirement that contaminant levels be kept below a certain maximum in neighbouring water resources. A typical requirement might be that seepage from the lagoon not cause degradation of groundwater or surface water quality outside of the lagoon property. Design standards relate to the properties of the lagoon structure or site conditions. For example, a design standard might specify a maximum allowable hydraulic conductivity for the pond bottom material.

In most jurisdictions a combination of performance and design standards is set. The design standards help to ensure both a minimum level of performance (although not necessarily adequate in all situations to meet environmental quality objectives) and a certain standardization of design. The performance standards make the designer responsible for taking normal precautions to ensure that any additional measures required to meet environmental quality objectives are taken.

A review of seepage control standards in other provinces and states was made during the preparation of this manual. Special attention was paid to areas with concerns and conditions similar to those in Alberta. The various regulatory agencies were contacted by telephone and through correspondence. In addition, information on seepage control standards and practices is available in published literature; e.g., Canter and Englande (1969) and Weston (1978). It should be recognized that regulations and standards are up-dated from time to time; thus the information presented herein may be dated and the relevant agencies should be contacted directly as required regarding current standards for sewage lagoons.

Prior to the last decade there was generally less awareness of the importance of groundwater resources and the potential for groundwater contamination, or other problems related to seepage and contaminant migration. As a result, seepage control standards were generally minimal and frequently contained guidelines that were either unspecific with respect to seepage rate or had vague requirements such as "clayey soil" or "impermeable soil" for pond bottom material. At the time of the survey by Canter and Englande (1969), Minnesota had a maximum allowable seepage rate of 0.32 cm/day, Kansas required less than 0.32 to 0.64 cm/day, and Washington had a maximum allowable seepage rate of 0.64 cm/day. These seepage rates are equivalent to 1.2 and 2.3 m/year. If a pond designed to accumulate and store 2 m of wastewater over a one year period had a seepage rate in this range, then the majority of the pond water would seep from the pond.

There has been a trend in recent years toward more stringent seepage control standards for all types of impoundments containing potential contaminants. The U.S. Environmental Protection Agency now requires the use of flexible polymeric liners in all lagoons containing designated hazardous materials. Clay liners for municipal waste facilities are commonly required to have hydraulic conductivities of less than 10^{-9} m/s. The State of Minnesota guidelines for sewage lagoons now require liners that control seepage rates to less than 5.2×10^{-9} m/s, or less than 0.2 m/year compared to the 1.2 m/year maximum required previously. These seepage rates and values of hydraulic conductivity are approaching the best that can be realistically accomplished with well designed and constructed compacted clay liners. Alberta's new standards for seepage control in sewage lagoons (see Section 4.1.2) is similar to the Minnesota standard. It is believed that this standard is technically and economically feasible, and will help to prevent degradation of groundwater and surface water quality due to contaminant migration from sewage lagoons.

1.4 SUMMARY OF LINER DESIGN PROCEDURE

1.4.1 Preliminary Site Selection

The first step in the site selection procedure should be collection of available, pertinent data for the study area. This would include topographical, geological, hydrogeological, and geotechnical information. The study area should then be divided into areas of low, medium, and high suitability for lagoon siting on the basis of environmental and social factors. The most suitable sites are then evaluated and ranked on the basis of environmental and social acceptability and cost, and a prime site is chosen for more detailed (field) investigation. The requirements for seepage control and the costs of lining the lagoons should be considered in the site selection process.

Areas to be avoided include:

- flood plains
- buried channel aquifers
- hilly terrain
- high bedrock (less than 3 m below ground surface)
- high water tables

Preferred sites are generally located in low permeability clayey soils.

1.4.2 Site Investigation

The site investigation should include a reconnaissance of surface features (such as bedrock outcrops, drainage paths, vegetation, etc.) and subsurface exploration, preferably by drilling boreholes and taking soil samples. A minimum of 5 boreholes or 1 borehole per 2 hectares (whichever is greater) is recommended. If the facility will be large and geologic conditions are complex then more boreholes may be warranted. Boreholes should be drilled to a depth of about 6 m below the proposed lagoon invert elevation, and at least one borehole should be drilled to a depth of 20 m or to auger refusal in bedrock, whichever occurs first. Three boreholes should penetrate the groundwater table in order to determine groundwater flow direction and gradient.

Objectives of the investigation should include:

- interpretation of site geology to allow reasonable prediction of conditions between borehole locations;
- determination of hydraulic properties of sediments at the site;
- determination of foundation conditions below fill areas;
- location of borrow sources for dyke fill, underdrains, liner material, etc.

1.4.3 Seepage Control Criterion

The Standards and Approvals Division of Alberta Environment has adopted a seepage control criterion for sewage lagoons that relates the maximum allowed hydraulic conductivity of the pond liner to the liner thickness, as outlined in Section 4.1.2. This criterion only applies to particulate, porous materials such as compacted clay, bentonite and sand admixes, and asphalt concrete, where seepage rate is governed by Darcy's Law. Standards are also set for minimum liner thickness and construction procedures, depending on liner type.

1.4.4 Liner Selection

There are several materials that can be used to line lagoons, including:

- natural deposits with low in situ hydraulic conductivities;
- compacted clayey soil;
- bentonite and sand admixes;
- hydraulic asphalt concrete with spray-on bitumen membranes;
- flexible polymeric membranes.

Factors that influence the choice of liner include:

- ability to meet the seepage control criterion;
- availability;
- durability in the anticipated service environment;
- cost.

1.4.5 Liner Design Procedure

The liner design procedure will include:

- specification of material properties as applicable, including hydraulic conductivity, density, mix ratios, etc.;

- determining minimum liner thickness based on the seepage control criterion, design standards or anticipated service environment conditions;

- determining subgrade requirements;

- determining liner protection requirements;

- design of any required underdrain system to control groundwater seepage and/or provide venting of gas.

The design procedure should also include a prediction of the impact of any seepage on the surrounding environment, including:

- potential for seepage discharge at the dyke toes or in nearby depressions because of groundwater table mounding;

- potential for degradation of groundwater and surface water resources due to seepage.

1.4.6 Liner Design Standards

The following subsections summarize some of the design standards associated with the different potential liner materials (in addition to the seepage control criterion presented in Section 4.1.2). It is emphasized that these are minimum requirements and the onus is on the designer to recognize the need for more stringent requirements to meet the overall performance objectives of a) negligible degradation of surrounding groundwater and surface water resources, b) maintenance of aesthetic conditions in the vicinity of the lagoon, and c) maintenance of lagoon structural integrity.

1.4.6.1 Natural In Situ Liners

When the natural *in situ* sediments (in their undisturbed state) are utilized as the seepage control "liner", the designated liner deposit should be relatively uniform and homogeneous so that the properties of the liner deposit can be determined with confidence. The liner deposit should be at least 0.9 m at the thinnest point below the pond, and should be completely free of hydrogeologic windows such as sand or silt pockets that penetrate the liner deposit. The vertical hydraulic conductivity will control seepage through the pond bottom, while the horizontal hydraulic conductivity will control seepage through the sideslopes. In many cases it may be necessary to combine engineered sideslope liners with natural bottom liners because of high horizontal hydraulic conductivities caused by layering and desiccation in deposits near the ground surface.

1.4.6.2 Compacted Clay Liners

The hydraulic conductivity of compacted clay liner material should be determined in laboratory permeameters following accepted (e.g., ASTM) procedures (see Appendix 3). Hydraulic conductivity should be determined as a function of soil dry density and moulding water content. Variations in dry density and moulding water content can alter the hydraulic conductivity of compacted clay by up to 2 to 3 orders of magnitude (e.g. Mitchell *et al.* 1965). Materials used in the tests should represent the range of materials that will conceivably make up the actual liner. A weighted average for the K value may be appropriate. The actual hydraulic conductivity of the liner in the field should be assumed to be at least one order of magnitude greater than the laboratory value to account for variations in materials, macro-structure, test errors, etc. The assumed field K value should be used when determining whether the seepage control criterion is met (Section 4.1.2). The minimum liner thickness should be 0.6 m on the bottom and 1.2 m on sideslopes (measured perpendicular to the slope). A bottom liner thickness of at least 0.9 m would be preferred. The liner should be constructed in 15 cm horizontal and uniform lifts, compacted to the required density and within the required moisture content range to achieve the desired K value.

1.4.6.3 Bentonite and Sand Admix Liners

Bentonite and sand admix liners consist of powdered or granular bentonite mixed with native sand in ratios that result in a low permeability material (once the bentonite has hydrated and swollen to plug the voids in the sand). Only moderately to high swelling sodium bentonite (i.e., where sodium is the predominant exchangeable cation in the bentonite clay structure) is recommended. The admix liner should be at least 7.5 cm thick after compaction. The bentonite application rate required to produce a liner hydraulic conductivity that meets the seepage control criterion in Section 4.1.2 should be determined by laboratory permeability tests. The required application rate determined by the permeability tests should then be increased by 25% to allow for field conditions.

1.4.6.4 Asphalt Liners

Asphalt-based materials used for lagoon liners include hydraulic asphalt concrete (HAC), spray-on bitumen membranes, soil asphalt admixes, and asphalt panels. The only asphalt liners recommended in this manual are a) spray-on bitumen over HAC, and b) spray-on bitumen over soil asphalt. The admix bases provide support for the relatively weak membranes, while the bitumen membranes provide additional seepage control over the more crack-susceptible HAC and soil asphalt. HAC should have an asphalt content of about 7 to 9% and a porosity below 4% to have hydraulic conductivities that meet the seepage control criterion (Section 4.1.2). A 10 cm liner thickness is recommended, comprising two 5 cm lifts with staggered joints. Soil asphalt liners should be mixed with 15 cm of native sandy soil. In all cases asphalt should be placed over non-frost susceptible base-coarses or subgrades. The spray-on bitumen covering the HAC or soil asphalt surface should be catalytic air blown asphalt applied to produce a uniform 2 mm thick membrane.

1.4.6.5 Flexible Polymeric Membrane Liners

Seepage through flexible polymeric membranes occurs through holes and gaps in seams, as well as under chemical and vapour pressure gradients through the intact material. The amount of seepage occurring due to the latter mechanisms is generally negligible; therefore the primary concern with flexible polymeric membranes is to minimize the number of holes and improper seams. This is accomplished by selecting liner materials that are durable enough to withstand the construction and service environments and by exercising careful quality control during placement and seaming operations. A minimum membrane thickness of 20 mils is recommended. Membranes less than 60 mils thick should be covered with a 30 cm layer of fine-grained soil on the lagoon sideslopes to prevent liner damage. Care must be taken to avoid puncturing the liner during soil cover placement. PVC and other materials that are susceptible to rapid weathering when exposed should be covered with soil on both the sideslopes and lagoon bottom. The bedding layer below membrane liners should provide: 1) a smooth, uniform support surface, 2) venting of gas, 3) drainage of groundwater and seepage. A well-graded sand with less than 5% fines (passing No. 200 sieve) is generally adequate for these purposes. A seepage collection system (e.g., underdrain pipes and sump) may also be required. Gas vents should be installed in the liner near the dyke crests.

1.4.7 Groundwater Monitoring

Groundwater observation wells should be installed during the site investigation program (generally in exploration boreholes outside of the lagoon). Periodic measurements of groundwater levels and chemistry allow determination of seepage direction and velocities and monitoring of contaminant migration. One monitor well should be installed per 2 ha of site with a minimum of 5 installations per site. At least one well should be up-gradient from the lagoon to provide background water quality values.

2. SITE SELECTION

2.1 ACQUISITION OF AVAILABLE DATA

The first step in the site selection and design process is collection of available, pertinent data. This includes 1:50,000 topographic maps, surficial and bedrock geology maps, hydrogeologic maps, climatic information, stereo-paired aerial photographs, and any reports or studies or large scale maps of the region where the sewage lagoons are to be sited. Appendix 1 lists sources of available information.

2.2 PRELIMINARY SITE SELECTION PROCEDURE

2.2.1 General

Once available data for the study area has been acquired, a site is selected for more detailed investigation. For small villages or towns sites may be selected subjectively based on the judgement of the designer/consultant and the consensus of local inhabitants. Provided that the site meets certain criteria and Departmental approval, this approach may be adequate. For most town and cities, however, the potential area for locating the treatment facilities may be quite large with a number of different environmental, social, and economic factors affecting the decision on site location (Table 2.1). The suitability of a site with respect to each factor may be rated subjectively from poor to excellent. The site selection process is generally a compromise between these factors.

The following paragraphs and charts provide guidelines for evaluating the environmental suitability of a site in terms of surface and groundwater pollution potential due to seepage from lagoons. Sites are rejected if certain criteria are not met. On the basis of pollution potential, remaining sites are ranked as having a low, medium, or high

TABLE 2.1 FACTORS AFFECTING SITE SELECTION

ENVIRONMENTAL	- groundwater protection - surface water protection - land use - receiving waters (availability, sensitivity)
SOCIAL	- aesthetics - property value - land use - site security
ECONOMIC	- pipelines - pumping stations - access - construction conditions - liner requirements - maintenance - land use/cost

suitability for siting sewage lagoons. Procedures for evaluating the social and economic aspects of potential sites are beyond the scope of this manual, but may be addressed in a similar manner. These two factors should be combined with the environmental assessment for an overall rating and site selection decision. However, the overriding consideration should be protection of the environment.

2.2.2 Delineation of Study Area

The site selection procedure is shown in flowchart form on Fig.2.1. First the study area is delineated. Obviously unsuitable land within the study area should be indicated on the plan of the study area. Suggested environmental restrictions are listed in Table 2.2. A similar set of restrictions may be selected based on social and local issues, such as land use, wind direction, and set-back from roads.

2.2.3 Ranking of Available Land in Study Area

Available land in the study area should be ranked on the basis of environmental factors as shown in the rating chart on Table 2.3. Each factor may be delineated on a separate overlay sheet for the study area map. When the sheets are superimposed, areas with high ratings for all environmental factors will be indicated. A similar procedure may be followed for ranking on the basis of social and economic factors.

2.2.4 Economic Assessment of Sites

The next step in the preliminary site selection procedure usually involves economic assessment of the most environmentally and socially suitable sites. An example evaluation chart for summarizing the data for potential sites is shown in Table 2.4. Table 2.5 indicates probable lining requirements. Approximate costs for different liners are shown on Table 4.5 in Section 4.3.5. The Rapid Infitration Manual by Reid

FIGURE 2.1 SITE SELECTION FLOW CHART

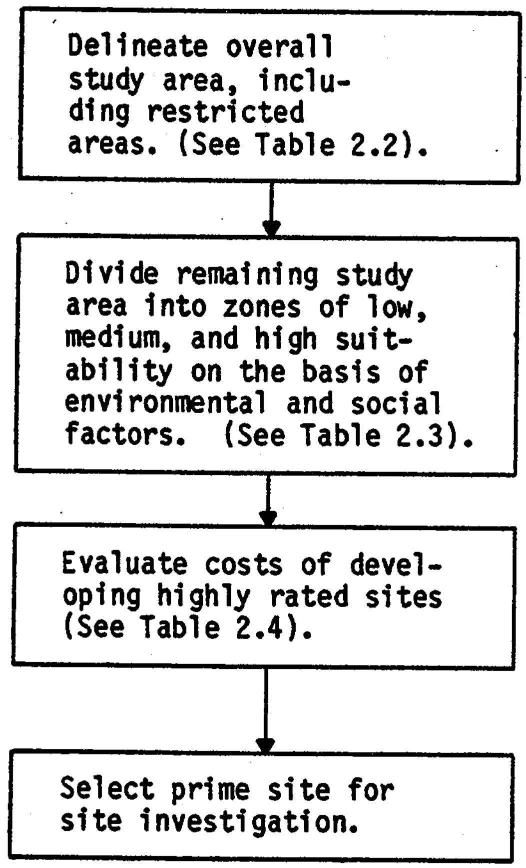

TABLE 2.2 PHYSICAL SITE CRITERIA

1. Flood Plains

River flood plains should be avoided. If siting in a flood plain is desirable for economic and/or social reasons, a hydrology study should be carried out to determine the position of flood levels and flow velocities around the lagoon structures. The potential influence of the lagoons on flood levels and flow velocities should be indicated. Designs should include sufficient freeboard and exterior erosion protection to prevent damage in the event of flooding.

2. Drainage Ways

Drainage ways should be avoided. If siting in a drainage way is desirable for economic and/or social reasons, a hydrology study should be carried out to determine surface runoff storm flows. The lagoon design must then provide suitable diversion of the storm water and adequate erosion protection.

3. Buried Channels

Do not locate lagoons over buried channel aquifers, or over aquifers in hydraulic communication with buried channel aquifers, unless it can be demonstrated that the aquifers are protected by a substantial thickness of low permeability material.

4. Groundwater Level

Avoid sites with high groundwater tables that might cause construction problems or cause flotation of membrane liners.

(CONTINUED ON NEXT PAGE)

5. Bedrock

The bedrock surface should be a minimum 3 m below the pond invert. A minimum depth of 10 m is recommended when the upper bedrock formations include coal seams, highly fractured or weathered rock, and other deposits with relatively high permeability. Do not locate lagoons over karstic bedrock.

6. Topography

Avoid hilly or steeply sloping terrain.

TABLE 2.3 RATING CHART FOR PHYSICAL ENVIRONMENTAL SUITABILITY OF SITES

	SUITABILITY FOR SITING LAGOONS		
PARAMETER	LOW	MEDIUM	HIGH
BEDROCK	Sandstone, coal seams, or weathered bedrock with less than 10 m of overlying surficial sediments	Sandstone, coal seams, or weathered bedrock with more than 10 m of overlying surficial sediments Shale or siltstone bedrock with less than 10 m of overlying surficial sediments	Shale or siltstone and more than 10 m of overlying surficial sediments
SURFICIAL SEDIMENTS (SOILS)	alluvial sands and gravels	lacustrine sand and silt, fissured till	lacustrine silt and clay, unfissured silt and clayey till
GROUNDWATER	major aquifer recharge or discharge zone	maximum groundwater elevation less than 1.2 m below pond invert	maximum groundwater elevation more than 1.2 m below pond invert
TOPOGRAPHY (% slope)	5%	1-5%	0-1%

TABLE 2.4 PRELIMINARY SITE EVALUATION CHART

SITE	DISTANCE FROM TOWN OR PLANT	TYPE OF BEDROCK	DEPTH TO BEDROCK	SURFICIAL SEDIMENTS	(1) EST.K	(2) DEPTH TO HIGH G.W.	(3) RECHARGE OR DISCHARGE ZONE	LAND USE	EST. COST	RANKING/ COMMENTS
1										
2										
3										
4										
5										
6										

NOTES: (1) Est. K = estimated hydraulic conductivity of natural sediments
(2) G.W. = groundwater table
(3) Groundwater flow system discharge and recharge zones

TABLE 2.5 PROBABLE LINER REQUIREMENTS

SOIL CONDITIONS	LINER REQUIRED?	LIKELY SUITABLE LINER TYPES[1] COMPACTED NATIVE CLAY	BENTONITE ADMIX	ASPHALT	POLYMERIC MEMBRANE
sands and gravels, silt	Yes	Maybe	Yes	Yes	Yes
fissured fine-grained sediments	Yes	Yes	Maybe[2]	Yes	Yes
unfissured fine-grained sediments	Maybe	Yes	Maybe[2]	Yes	Yes

[1] Subject to findings of site investigation and final design requirements.

[2] Bentonite admix liners may be suitable if sufficient sandy soil is available for mixing with the bentonite.

Crowther et al (1980) provides some data for estimating cost for pipelines, pump stations, and the like.

2.3 DISCUSSIONS WITH ALBERTA ENVIRONMENT

Once available data collection and preliminary site selection are completed, the designers and municipality should initiate discussions with the Standards and Approvals Division, Municipal Engineering Branch of Alberta Environment. The Department staff will review the information and comment on the general suitability of the procedure to date, and on the site(s) selected. Favourable comments at this stage should not be construed as tentative or final approval of a site. Final approval will only be issued to adequately designed facilities at the conclusion of the engineering studies. In addition, the Department will indicate whether a standard investigation and design procedure will be adequate, or whether a detailed study is required. Additional investigation and design details are required for detailed studies, as indicated in the following sections. At the discretion of the Department, a detailed study will generally be required for very large ponds or when the proposed site is in an environmentally sensitive area. It should be noted that a standard study may require up-grading to a detailed study if the findings of the site investigation and engineering study indicate this is necessary.

3. SITE INVESTIGATION

Recommended site investigation procedures for lagoons are described in this Section. Drilling methods, number of boreholes, depth of boreholes, type of samples, sample frequency, laboratory tests, and *in situ* tests are discussed. This Section is not intended to be an exhaustive manual of drilling and sampling methods. Various techniques and equipment are described elsewhere, e.g. Hvorslev (1948), Terzaghi and Peck (1967), Campbell and Lehr (1977).

3.1 FIELD PROGRAM

3.1.1 General

Subsurface investigations may be performed by drilling boreholes, digging test pits, and geophysical surveys. Boreholes are recommended because drill rigs can normally penetrate overburden quickly and to the necessary depths with minimum disturbance to the site geology. Test pits are not recommended where the pit will create an artificial hydrogeological window in natural seepage barriers. Loosely backfilled pits in berm foundation areas can lead to differential settlement, which may create weak or fractured zones in the containment structure. Test pits outside of the lagoon area (e.g. for borrow investigations) are generally acceptable and economical. In general, geophysical surveys (in addition to drilling) would only be warranted for larger lagoons.

3.1.2 Drilling Equipment and Methods

Power auger drills are generally ideal for geotechnical subsurface investigations. Solid stem augers are suitable in firm to hard cohesive soils and in compact granular soils above the groundwater table. Hollow stem augers are required in soft cohesive soils, and in loose silts and sands above the groundwater table. For fine, loose sands and silts below the water table or when very hard tills are to be drilled, a rig capable of driving casing and advancing by wash-boring techniques may be required. Becker hammer rigs (reverse air circulation) are recommended in coarse granular sediments.

The drill rig used should be capable of obtaining disturbed but representative samples from the borehole. Auger cuttings alone are not generally sufficient, particularly below the water table; similarly wash samples alone are not generally sufficient.

Sonic drill rigs may be used to recover continuous core samples of the subsoil. The soil is partially disturbed by the vibratory action of the core barrel, but lamination and other structure is generally preserved.

3.1.3 Number and Location of Boreholes

Available topographic, geologic, and hydrogeologic information should be reviewed when planning site investigations so that borehole locations may be selected on the basis of anticipated conditions. The number and location of boreholes should not be arbitrary or random, but should be selected to efficiently investigate anticipated subsurface conditions. In general, boreholes should be located to: identify each anticipated lithologic unit of significance; identify the orientation of lithologic boundaries; identify the variation of properties over the site, and any special features that may effect the structural or hydraulic integrity of the lagoon. It is also desirable to develop one or more geological cross-sections through the middle of the lagoon, preferably parallel to the anticipated groundwater flow direction.

If conditions are uniform, or heterogeneous but uniform on a macro-scale, a random sample approach to predicting subsurface conditions may be successful. Normally, however, selective sampling and interpretation is required because of non-uniform conditions. Typical augered boreholes located on 8 m centres will encounter only 0.05% of the total soil volume, while samples at typical intervals will include only 0.001% of the total soil volume. Thus, reliance on statistical representation alone would not be prudent.

The spacing of borings should be close enough to allow reasonable prediction of conditions between boreholes, to the extent that variations in conditions encountered during construction can be accommodated without major alterations to the design of the ponds or lining system.

In general, when available information indicates that geologic conditions are likely to be unchanged over the site, fewer boreholes should be planned with more emphasis on good samples and laboratory testing. When more irregular conditions are anticipated, more boreholes with less thorough sampling is usually a better way to expend the investigation budget. In the latter case, it is more important to obtain a good understanding of geologic conditions at the site rather than detailed properties of only a few samples.

The investigation program should be flexible enough to allow for alterations based on the results of the first few boreholes and the interpretations of the project engineer/geologist.

In addition to locating boreholes to best interpret site geology in the most efficient manner, the following should be objectives of the investigation program:

i) to determine the hydraulic properties of sediments below the pond invert in order to assess the ability of the sediments to naturally control seepage rate;

ii) to determine foundation conditions below the fill areas of dykes;

iii) to determine the location of suitable borrow material for dyke construction (usually from the cut), liners, drains, and the like; and

iv) to utilize boreholes outside of the immediate pond area as monitor well installations.

Table 3.1 may be used as a guideline to estimate minimum borehole requirements. The number of boreholes may be reduced for very large facilities at the discretion of the engineer or geologist. In sensitive or complex geologic environments the number of boreholes should be greater.

The number of boreholes suggested in Table 3.1 result in average borehole spacings of 100 to 200 m, although as discussed previously, boreholes should not be drilled on an arbitrary grid. Hvorslev (1948) suggested a spacing of about 30 m for relatively large structures such as dams and highways, and larger spacings of about 120 to 150 m for more uniform conditions. However, the number of boreholes on Table 3.1 are probably appropriate as a preliminary estimate. At least 3 boreholes are required to determine the orientation of any geologic plane, such as the bedrock surface, or groundwater table. Three of the boreholes should approximate an equilateral triangle.

3.1.4 Depth of Boreholes

The hydraulic properties of the sediments in the first 3 or 4 m below the lagoon invert will have a predominant influence on the rate of seepage from the lagoon and on renovation of seeping wastewater through filtration, oxidation-reduction, adsorption, and other mechanisms (see Appendix 13). Therefore, a minimum borehole depth of about 5 to 6 m below the pond invert elevation is recommended.

TABLE 3.1 MINIMUM NUMBER OF BOREHOLES FOR SITE INVESTIGATION

	GEOLOGICAL & TOPOGRAPHICAL CONDITIONS	
	Uniform	Irregular
	- ground moraine - fine grained lacustrine - sheet glacial outwash - level to undulating topography	- ice contact deposits - hummocky morraine - coarse grained lacustrine - channel glacial outwash - recent fluvial - rolling to hilly
STUDY TYPE	RECOMMENDED NUMBER OF BOREHOLES (bh)	
STANDARD	1 bh/2 hectares (min 5 bh)	1 bh/2 hectares (min 5 bh)
DETAILED	1 bh/hectare (min 5 bh)	1 bh/hectare (min 8 bh)

At least 3 holes should penetrate the groundwater table in order to provide information on flow direction and gradients. At least one hole should be drilled to auger refusal in the bedrock or to a depth of 20 m, whichever occurs first. When detailed investigations are required, the bedrock should be cored for a minimum depth of 3 m. Deeper coring may be desirable if the bedrock is highly fractured, weathered, and/or relatively permeable below the upper 3 m.

In general, it is desirable to completely penetrate any significant aquifers immediately below the pond in order to determine the aquifer thickness and transmissivity for hydrogeological analyses. This would include unconfined aquifers immediately below the proposed lagoon invert, unconfined and confined aquifers below a surficial clay cap (aquitard), and possibly lower aquifers in detailed studies.

Some boreholes will likely be useful as monitor wells. These should meet the requirements for monitor well installations as discussed in Section 5.

Test pits may be dug outside of the lagoon location, and may be useful in exposing layering, fissures, cobbles, and other features that are difficult to detect or assess in borehole samples. Small backhoes can usually reach a depth of about 3 to 4 m, and larger backhoes can excavate to depths of 5 or 6 m. Personnel should not enter unsupported, vertical cuts deeper than 1.5 m, as specified by the Occupational Health and Safety Act of Alberta. Samples can be obtained from the bucket in deeper holes, and depths measured from the surface. A strong flashlight or mirror reflection may be useful for observing features in deep holes.

3.1.5 Borehole Samples and Drill Log

The hydraulic properties of natural sediments can be dominated by small features such as fissures, sand and silt seams, and layering. Careful sampling and observation by the field engineer/geologist is required to ensure that these features are identified. Ideally, boreholes would consist of continuous, large diameter, undisturbed samples. Unfortunately, economics and technology usually limit investigations to intermittent, small diameter, disturbed or partially disturbed samples. These limitations should be compensated as much as possible by (a) at least one continuously sampled hole, (b) a combination of sampling techniques, and (c) careful observation and logging of drill progress, water return, auger cuttings, etc., between samples.

More common sampling techniques are listed in Table 3.2.

Intermittent sampling is generally performed at 1.5 m intervals and when drilling progress indicates a change in soil conditions between sample depths. Continuous samples should be taken in all holes over the 3 m interval below the proposed pond invert if it is anticipated that the *in situ* sediments may be suitable as a natural liner (see Section 4.2.1).

One of the objectives of the field investigation should be to determine the probable maximum high groundwater level. Ideally this is determined by long term observation of monitor well or piezometer levels. Time constraints, however, may require augmenting short term monitoring of water levels with geologic evidence such as the depth of colour change (from aerobic to anaerobic weathering environments), oxidation along fissures and partings, moisture conditions, and the like.

Disturbed samples should be sealed in heavy polyethylene bags or sample jars with screw tops. Thin-wall tube samples should be sealed at both ends with paraffin wax and handled with care. In the winter, unfrozen samples should be protected from freezing. Preservation of frozen samples may be useful for observing the effects of freezing on surficial sediments (e.g. extent and orientation of fissures due to ice lensing), particularly if these sediments are considered for compacted clay liners.

TABLE 3.2 BOREHOLE SAMPLE TYPES

SAMPLE TYPE	SUITABLE SOILS	DEGREE OF DISTURBANCE	INFORMATION THAT MAY BE DERIVED FROM SAMPLES
51 mm O.D., 35 mm I.D. split spoon	All surficial sediments except coarse gravels. Sand catchers required for loose, wet, fine sands.	Disturbed	particle size distribution to maximum I.D. of tube, colour, grain properties, Atterberg limits of cohesive soils, moisture content of fine-grained soils
Thin-wall tube or Shelby tube 51 to 76 mm I.D.	Soft to firm cohesive soils.	Relatively Undisturbed	all the above plus density, strength, deformation and hydraulic properties, as well as soil structure
Vibra-core or sonic tube samples	Silts, sands, and fine gravels	Disturbed, layering may be preserved	particle size distribution, layering, grain properties, colour
Auger samples	Most soils	Very disturbed, possibly mixed, depth uncertain	colour, particle size distribution, grain properties, Atterberg limits of cohesive soils
Wash samples	Sand and fine gravel	Completely disturbed, fines washed out, depth uncertain	colour, grain properties of recovered sizes, estimate siltiness from wash water colour

3.1.6 In Situ Tests

In situ tests can give rapid measurements of subsurface conditions. Although frequently more crude and empirical than laboratory tests, a number of *in situ* tests can give a better idea of the variation in subsurface conditions than a few laboratory tests on samples. The applicability of common *in situ* tests are discussed in Appendix 2.

3.1.7 Abandonment of Boreholes

Boreholes drilled within the proposed lagoon area should be grouted to prevent preferential seepage paths through any low permeability soils. Even disturbance of layering in silts and sands can increase vertical hydraulic conductivity by an order of magnitude or more. A mixture of Portland cement, sand, and water should be adequate in most cases. Care should be taken to fill the hole with grout from the bottom using a tremie tube to prevent bridging of grout in the hole, formation of voids, and segregation of the cement in standing water.

Boreholes located well beyond the containment structure may generally be backfilled with available soil, or may be used as monitor wells (Section 5).

3.2 LABORATORY PROGRAM

3.2.1 Index Tests and Classification

Standard visual classification of all samples should be carried out, and laboratory index tests should be performed on selected representative samples to confirm visual classification. A recommended schedule is shown on Table 3.3. The Unified Soil Classification System (Wagner 1957) is recommended and is described in most soil mechanics texts. ASTM[1] test standards are recommended for index and other tests when

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1 ASTM - American Society for Testing and Materials.

applicable. Sample descriptions should be augmented by field observations when possible, e.g. SPT or field vane test results; presence of large gravel, cobble or boulder sizes; suspected thin sand layers in cohesive deposits, etc.

3.2.2 Strength Tests

Most of the province of Alberta is covered by dense glacial till deposits (Pawluk and Bayrock 1969). The strength of foundation materials is generally not critical for small lagoons founded in these materials. Foundation materials consisting of loose or soft sediments can occur, however, and would be identified by simple consistency or strength tests. Development of these sites may not be desirable for several reasons. However, if a lagoon is to be constructed in soft or loose ground conditions, more sophisticated strength and compressibility tests are likely warranted. A discussion of soft ground design and construction techniques is beyond the scope of this manual.

In compact or dense granular soils, an estimation of relative density based on SPT or CPT soundings should be adequate for most lagoons. In stiff to hard cohesive soils, field vane shear strengths, lab vane shear strengths, torvane strengths, or pocket penetrometer strengths should be adequate for assessing the intact strength of the soil. In highly fissured soils, lower bound residual strength values may be estimated from plasticity indices (e.g. Skempton 1964).

3.2.3 Compaction Tests

Moisture-density relations should be determined for potential dyke fill and compacted clay liner material (ASTM D698).

TABLE 3.3 SUGGESTED LABORATORY TEST ROUTINE

PARAMETERS & PROCEDURES		SAND & GRAVEL	SILT	CLAY	COHESIVE TILL	LOESS	ORGANIC SOIL	PEAT
GENERAL INFORMATION	COLOUR	A[5]	A	A	A	A	A	A
	VISUAL CLASSIFICATION[1]	A	A	A	A	A	A	A
	LABORATORY CLASSIFICATION[1]	R	R	R	R	R	R	-
UNDISTURBED SAMPLES	NATURAL MOISTURE CONTENT[2]	A	A	A	A	A	A	A
	STRENGTH TESTS[3]	-	R	R	R	R	R	-
	STRUCTURE IDENTIFICATION[4]	A	A	A	A	A	A	A
	HYDRAULIC CONDUCTIVITY TESTS	-	-	S	S	-	-	-
LABORATORY COMPACTED MATERIAL	MOISTURE-DENSITY RELATION (PROCTOR TEST)	S	-	S	S	-	-	-
	HYDRAULIC CONDUCTIVITY	S	-	S	S	-	-	-

1. Following Unified Classification System

2. Moisture content should also be determined for fine-grained split spoon samples.

3. Laboratory torvane, pocket penetrometer, unconfined compression tests, etc. should be sufficient in most firm to cohesive soils. More sophisticated triaxial and oedometer tests may be warranted for sites where large dykes will be located on soft or firm soils.

4. Layering,laminations,fissures,inclusions,oxidation,root holes, etc.

5. A - all samples, R - random samples, S - where specifically required to evaluate borrow for clay liners, underdrain sand, etc.

3.2.4 Hydraulic Conductivity Tests

Laboratory hydraulic conductivity tests should be carried out on undisturbed samples of native clay that might comprise a natural geologic liner (see Section 4.4.2). In addition, hydraulic conductivity tests are required for compacted clay liner material and bentonite-sand admixes (see Appendices 4 and 5). A discussion of hydraulic conductivity testing is presented in Appendix 3.

3.2.5 Soil and Groundwater Chemistry Tests

Certain soil chemistry tests may be pertinent to the design of compacted clay liners and to the evaluation of the attenuation capacity of the native sediments. The sodium absorption ratio (SAR) of porewater in clays gives an indication of the potential for dispersion when percolated by water with a relatively low total dissolved solids (TDS) content. The SAR is defined by:

$$\text{SAR} = [\text{Na}]/(([\text{Mg}] + [\text{Ca}])/2)^{1/2} \qquad (3.1)$$

where concentrations are measured in meq./L. A discussion of the significance of dispersion in clays is presented in Appendix 4.

The cation-exchange-capacity (CEC) of clayey soils provides a general indication of clay mineralogy and the potential for attenuation of heavy metals and other polyvalent cations.

Other tests that may be relevant to attenuation capacity of the native sediments (or compacted clay liners) include soil pH, calcium content, and organic content. More detailed discussion of these tests and the application of test results in assessing attenuation potential are discussed by Reid Crowther *et al* (1980).

Chemical analyses are required to establish the groundwater quality prior to sewage lagoon operation. This baseline information will be used to evaluate the changes in groundwater quality as a result of the lagoon operation. A minimum requirement would be collection of at least one water sample from each aquifer encountered during the geotechnical site exploration. The water samples should be preserved, stored and tested according to the standard methods of Alberta Environment. A recommended scope of groundwater analysis is shown on Table 3.4.

TABLE 3.4 RECOMMENDED SCOPE OF GROUNDWATER ANALYSIS

Main Ions:	Calcium Magnesium Sodium Potassium Iron Chloride Sulphate Bicarbonate
Nutrients:	Total Ammonia Nitrate Nitrite Phosphate
Organics:	Total Organic Carbon
Bacteriological:	Total Coliforms Fecal Coliforms Standard plate-counts
Other Parameters:	Temperature (field) pH (field) Salinity Total Dissolved Solids Colour Odour

4.0 DESIGN OF LAGOON LINER SYSTEMS

4.1 SEEPAGE CONTROL

4.1.1 Definition of a Liner

For the purposes of this document, a liner is described as the layer below a pond invert that controls the seepage rate from the pond. If more than one layer has a significant influence on seepage rate, then the layer with the lower hydraulic conductivity is generally defined as the liner. As discussed in Section 4.2, this layer may be a natural soil deposit, compacted clay, an admix material, or a polymeric membrane. The liner material must meet the seepage criterion in Section 4.1.2.

4.1.2 Seepage Control Criterion

The Standards and Approvals Division of Alberta Environment has adopted a seepage control criterion for municipal wastewater stabilization ponds (excluding rapid infiltration lagoons). The criterion, shown on Fig. 4.1, specifies a maximum hydraulic conductivity, K_L, for the pond liner as a function of the liner thickness, L, by the equation:

(4.1)

$$\text{Maximum } K_L \text{ (m/s)} = \frac{5.2 \times 10^{-9} \text{ m/s} \times L \text{ (m)}}{2 + L \text{ (m)}}$$

where all units are in metres and seconds.

For example, a compacted clay liner that is 1 m thick must have a hydraulic conductivity of about 1.7×10^{-9} m/s or less. The procedure for determining K_L for compacted clay liners is outlined in Appendix 4. Similarly, a bentonite admix liner or hydraulic asphalt

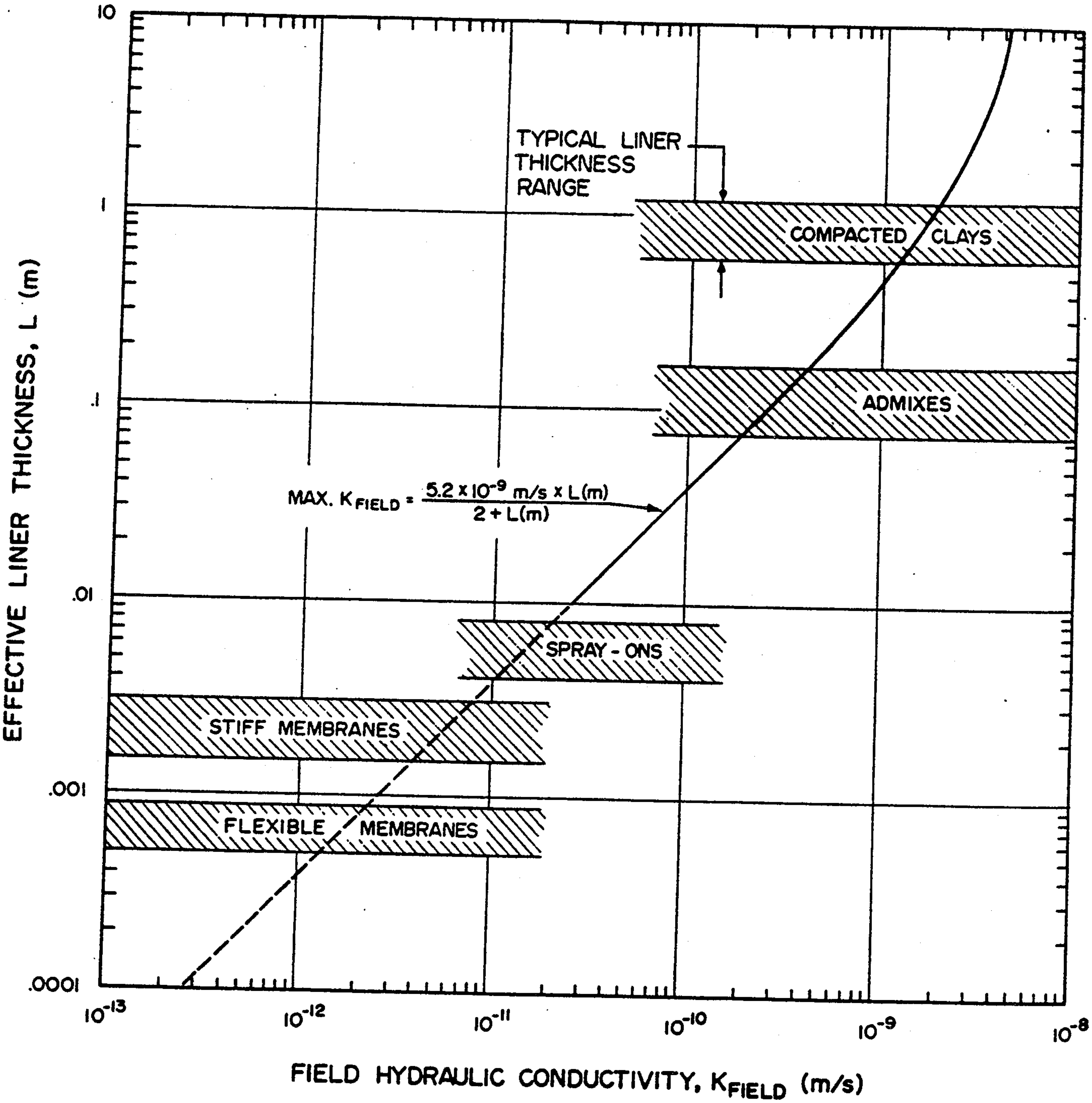

FIG. 4.1 MAXIMUM FIELD HYDRAULIC CONDUCTIVITY FOR COMPACTED CLAY AND ADMIX LINERS AS A FUNCTION OF LINER THICKNESS AS REQUIRED BY ALBERTA ENVIRONMENT FOR MUNICIPAL WASTEWATER STABILIZATION PONDS

concrete liner that is 0.1 m thick must have a hydraulic conductivity of 1.7×10^{-10} m/s or less.

Equation (4.1) is based on Darcy's Law for seepage through porous media, an average wastewater depth of 2 m, atmospheric pore pressures below the liner, and a maximum allowable seepage rate of 5.2×10^{-9} m/s. Darcy's Law is defined by:

$$v = Ki \tag{4.2}$$

where v is the discharge velocity or Darcy flux, and i is the hydraulic gradient. The relation is not valid for turbulent flow conditions (e.g., in coarse gravel). Darcy's Law will apply to natural sediments, compacted clay, admix liners, and all other particulate materials with interconnected voids. The relationship may not directly apply to continuous polymer materials in which fluid transmission occurs under electro-chemical and vapour pressure gradients rather than hydraulic gradients. Therefore the criterion is shown as a dashed line on Fig. 4.1 for liner thicknesses of less than 0.01 m.

Typical ranges of hydraulic conductivity and liner thickness for different liner materials are also shown on Figure 4.1, based on laboratory test data and observations of field performance reported in the literature (e.g. Kays 1977, Matrecon 1980). Natural compacted clay and admix liners may be designed to meet the criterion on Fig. 4.1 by using materials with suitable properties and following appropriate methods of construction. The hydraulic conductivities shown for spray-on and polymeric membrane liners are <u>apparent</u> values, estimated from observed seepage rates and assuming that Darcy's Law applies (i.e. assuming seepage occurs by hydraulic flow through discontinuities in the liners).

It should be noted that the rate of increase in maximum allowable hydraulic conductivity, with increasing liner thickness, decreases significantly above a thickness of about 1 m. This is because the wastewater depth becomes less significant with respect to liner thickness and the seepage gradient asymptotically approaches 1. Thus, if the _in situ_ sediments comprise a natural liner then the first few metres will dominate seepage performance in uniform materials.

In practice, lower average wastewater heads, groundwater table mounding (see Section 4.4.3), and possibly other conditions, may reduce the rate of seepage below the maximum rate allowed by Equation (4.1). These conditions may not apply during the first few years of operation, however, and should not be used as rationale for increasing the maximum allowed K_L values.

4.2 TYPES OF LINERS

4.2.1 Natural In Situ Liners

An _in situ_ clay or till deposit may meet the seepage control requirements for a pond liner, as outlined in Section 4.1.2. To meet the criterion for liner hydraulic conductivity, the hydraulic conductivity of the _in situ_ sediments must be less than about 2 to 4 x 10^{-9} m/s, depending on the minimum thickness of the deposit that is designated as the liner. It is likely that natural liners will be limited to relatively unfissured clayey soils. Relatively deep cuts may be required to avoid upper, more dessicated and fissured material. Surface compaction or proof-rolling of the native soil should be carried out to minimize the effects of construction disturbance.

If the hydraulic conductivity of the native, undisturbed soil exceeds the maximum allowed value of K_L in Section 4.2.1, surface compaction alone will generally be insufficient to reduce seepage rates to the required level (e.g. see Blight 1966).

4.2.2 Engineered Liners

There are several natural and synthetic materials which may be placed in a pond to form an engineered (as opposed to in situ) liner. The most common liner materials are listed on Table 4.1. Different types of liner materials are discussed briefly below. Detailed information on liner material properties is presented in Appendices 4 to 8.

4.2.2.1 Compacted Clayey Soil

Clayey soil from the pond excavation may meet the seepage control criterion if broken up and recompacted in thin lifts to form a compacted soil liner. Soil conditions may require the use of selected soil from the cut to form a uniform, relatively homogeneous liner. The clay may also be borrowed from a nearby location. Although generally cost effective, compacted soil liners require careful design and strict quality control to ensure satisfactory performance.

4.2.2.2 Admixes

Admix liners consist of low permeability matrix material mixed into a thin layer of native, sandy soil. The admix, such as powdered bentonite clay or asphalt, controls seepage while the sandy soil acts as a filler to improve strength, increase the liner thickness, and to facilitate construction. Hydraulic asphalt concrete, simlar to paving asphalt, is also classified as an admix material, although both the bitumen and the aggregate may be imported to the site. Soil cement and soil asphalt are other admix materials; however, these are recommended as soil stabilizers below spray-on membranes rather than as liners by themselves.

4.2.2.3 Flexible Polymeric Membranes

Polymeric membranes are synthetic plastic or rubber sheets that are seamed in the field using solvents, adhesives, or welding processes to form continuous membrane liners. There are several polymers and resultant compounds that are made into liner sheets, and these have a

TABLE 4.1 TYPICAL LINER MATERIALS
(from Folkes, 1982)

Class	Typical materials	Remarks
Compacted fine-grained soil	Local clayey soil	Porous, discontinuous liner, economical, typically 0.3–1.2 m thick
Admixes	Bentonite Soil cement Soil asphalt Hydraulic asphalt concrete (HAC)	Low permeability binder mixed in with native soil, typically 5–10 cm thick layer
Polymeric membranes:		
Thermoplastics	Polyvinyl chloride (PVC) Chlorinated polyethylene (CPE) Chlorosulfonated polyethylene (CSPE) (Hypalon)* Elasticized polyolefin (ELPO)	Continuous liner, discontinuous where damaged, relatively expensive, typically 0.5–2.6 mm thick, may be reinforced with polyester scrim
Vulcanized elastomers	Butyl rubber Neoprene (CR) Ethylene propylene diene monomer (EPDM)	
Crystalline thermoplastics	Low density polyethylene (LDPE) High density polyethylene (HDPE)	
Spray-ons	Catalytically blown asphalt Emulsified asphalt	Continuous liner, discontinuous at pinholes, cracks, typically 4–8 mm thick
Sealants	Polyacrylamide Liquid vinyl polymer	Sprayed, dusted or ponded, may result in nonuniform coverage
Chemisorptives		Function is to absorb contaminants, experimental

*Registered trademark of DuPont.

wide range of material properties. The most common polymeric liner materials are polyvinyl chloride (PVC), chlorinated polyethylene (CPE), chloro-sulfonated polyethylene (Hypalon[1]), and high density polyethylene (HDPE). Membrane thicknesses range from about 15 mils[2] for PVC up to 100 mils for some HDPE liners. The hydraulic conductivity of intact membranes is generally not measurable, although membranes are susceptible to punctures and faulty seams if strict quality control is not exercised. Some materials, such as PVC, must be protected from sunlight to prevent degradation while others are weather-resistant.

4.2.2.4 Spray-ons

Spray-on liners include: bitumen, which is generally sprayed over an admix material such as soil asphalt or hydraulic asphalt concrete; rubber or urethane extended bitumens which solidify to form elastic membranes that are somewhat similar to polymeric membranes; and other polymeric liquids such as polyacrilamide. The hydraulic conductivities of spray-on liners are generally very low, similar to those of polymeric membranes and asphalt liners. It may be difficult, however, to ensure good coverage, uniform thickness, and to avoid pinholes. An advantage of a well-installed spray-on liner is the lack of seams.

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1 Hypalon is a registered trade-mark of DuPont, and is available from several liner manufacturers and suppliers.

2 A mil is equal to 0.001 inch and is the common unit for describing membrane thickness. A thickness of about 39 mils is equal to 1 mm.

4.2.2.5 Sealants

Sealants are classified as powders or granular materials that are dusted on the subgrade surface, or are allowed to settle through the water to the pond bottom. These materials, such as bentonite and polyacrilamide, plug up the voids in the upper layer of the soil, and generally swell to form a gel. It is very difficult to obtain complete coverage of the pond bottom and sideslopes with sealants (Matrecon 1980). While the reduction in seepage rate due to sealants may be beneficial in terms of economic loss (e.g. irrigation water), the resultant overall permeability of the liner may not be adequate for pollution control. Chemisorptives are similar to sealants, except they are meant to react with the migrating fluid to either capture contaminants (e.g. through cation exchange) or to alter solutes to an inert form. Many of these materials are still experimental (Kays 1977).

4.3 LINER SELECTION

4.3.1 General

There are usually several alternative liner materials that can be used at any given site. However, while each material may be capable of achieving the seepage control standards outlined in Section 4.1.2, the durability and installed cost of the liner materials may vary significantly. The selection of a suitable liner material should be based on :

a) the ability of the installed liner to meet the seepage control standards of Section 4.1.2,
b) the availability of the material,
c) the durability of the material in the expected service environment, and
d) the material cost (installed).

These factors are discussed below. Suggested design procedures for several common liner materials are presented in Appendices 4 to 8.

4.3.2 Seepage Control Properties

4.3.2.1 Compacted Clay

The hydraulic conductivities of cohesive soils vary by several orders of magnitude. The hydraulic conductivity of a particular clay may also vary by more than an order of magnitude depending on degree of compaction, moulding moisture content, and pore water chemistry.

Matrecon (1980) suggests that the clay content[1] of compacted clay liner materials should be about 25% or greater, although soils with lower clay contents may be adequate if a significant proportion of the clay minerals is montmorillonite. In general, laboratory hydraulic conductivity tests on the compacted clay will be required to assess the suitability of the soil as a liner material (see Appendix 4).

The borrow deposit for the compacted clay should be relatively uniform and of sufficient volume to construct a liner of at least 0.6 m in thickness, or as otherwise required by the design engineer or Alberta Environment.

4.3.2.2 Bentonite Admix

Powdered or granular bentonite can usually be mixed with sand or silt to form a low permeability liner meeting the requirements of Section 4.1.2. Important considerations are the ability to thoroughly and uniformly mix the bentonite with the native soil and the quantity of bentonite required to create a liner with a sufficiently low value of

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[1] Clay content is usually defined as the percent by dry weight of the soil with an effective grain size of less than 2 μm.

hydraulic conductivity. In general, hydraulic conductivity tests will be required to determine the bentonite application rate. Bentonite suppliers can usually provide typical application rates for a given liner thickness and material type, and will often provide laboratory testing as a customer service. The application rate should be confirmed by an independent laboratory during final design (see Appendix 5).

The successful performance of a bentonite admix liner will also be highly dependent on the degree of quality control during construction.

4.3.2.3 Asphalt Liners

Intact hydraulic asphalt concrete (HAC) with asphalt contents of about 7 to 9% and porosities lower than about 4% should have very low hydraulic conductivities meeting the seepage control standards of Section 4.1.2. However, HAC liners are susceptible to cracking from frost action, ageing, and differential settlement of the subgrade, which may lead to unacceptable seepage rates. Thus, HAC liners are not suitable over frost susceptible soils, although designs that mitigate the potential for frost heave may be acceptable (see Appendix 6). In general, a spray-on asphalt membrane is recommended over the HAC, and the composite liner should be covered with a protective layer of soil.

In some cases <u>soil asphalt</u> may provide a suitable base for a spray-on asphalt membrane. The soil asphalt must overlie non-frost susceptible soils.

4.3.2.4 Flexible Polymeric Membranes

Flexible polymeric membranes are frequently called "impermeable" liners. This is a misnomer, since finite quantities of fluid transmission will occur through virtually all materials over an area the size of most lagoons. However, if the membrane is well-installed and free of defects, fluid passage only occurs through vapour transmission and osmosis. The equivalent seepage rate would be very small and well within the required seepage requirements. Because the rate of fluid passage by the various mechanisms is difficult if not impossible to quantify, the design procedures and requirements presented herein for flexible polymeric liners are based on material type, thickness, and installation procedures rather than values of hydraulic conductivity, gas permeability, etc.

4.3.3 Availability

4.3.3.1 Clay Soils

Approximately 70% of the interior plains region of Alberta is covered with a veneer of ground and hummocky moraine (Pawluk and Bayrock 1969), which in many areas has 20% or more clay size particles with plasticity indeces, I_p, in the 15-30% range (Fig. 4.2). Thus, the till in many areas may have sufficient clay content to create an adequate compacted clay liner, depending on measured K values and method of construction. Glaciolacustrine clay deposits are present in several areas, particularly around Edmonton and north of Red Deer. These may be suitable for liner construction if sufficiently free of silt and sand layers.

4.3.3.2 Bentonites

High swelling, sodium bentonite deposits exist in Alberta and a few northern states, notably Wyoming. These are commercially produced and are available in powdered or granular form for bentonite admix liners. Some products are treated with polymers to improve resistance to liner degradation in saline waters, acids, and other industrial fluids. Untreated bentonites should be suitable for typical municipal wastewaters (see Table A12.1), but higher application rates may be required than for treated bentonites. In general, the bentonite should contain a high percentage of montmorillonite, and have sodium as the dominant exchangeable cation. Lower quality bentonites, such as bentonites with calcium as the predominant exchangeable cation, will require higher application rates to achieve the desired level of seepage

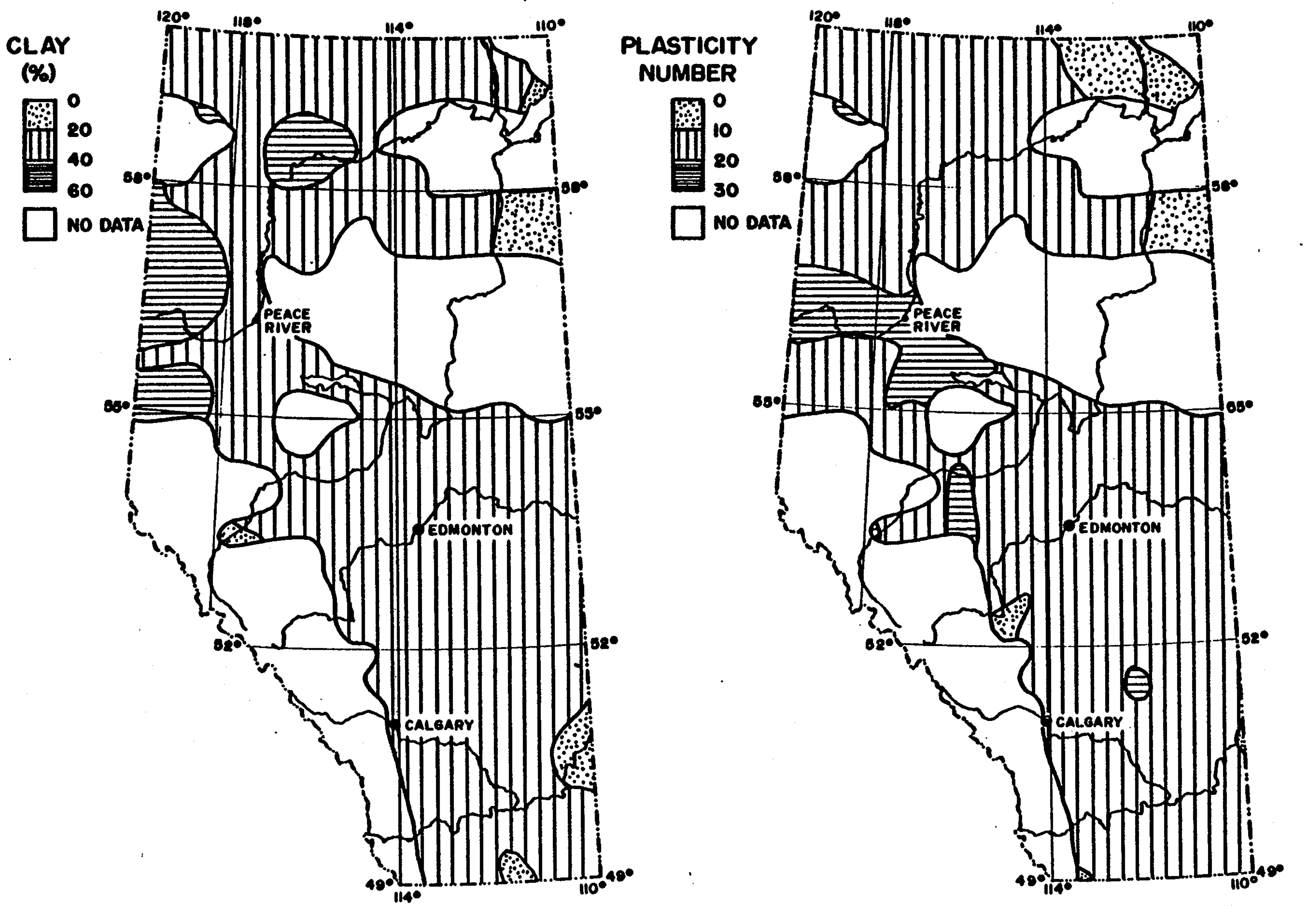

FIG. 4.2 DISTRIBUTION OF CLAY CONTENT (BY DRY WT.) AND PLASTICITY IN ALBERTA TILLS (FROM PAWLUK AND BAYROCK 1969)

control and will have reduced self-healing capabilities (lower swell potential).

4.3.3.3 Asphalt

Asphalt materials should be readily available in most areas of southern and central Alberta.

4.3.3.4 Flexible Polymeric Membranes

Most common flexible polymeric membrane liners are readily available in Alberta although delivery may take several weeks in peak demand periods, particularly if large quantities are required. PVC and CPE are manufactured in Canada, while Hypalon and HDPE generally come from the U.S. or South Africa.

It is recommended that polymeric membranes be purchased from suppliers who also supervise installation of their products and who have considerable experience installing the selected material. The supplier should be prepared to have an experienced foreman or supervisor on the site at all times, although local labour is generally used to assist in the work.

4.3.4 Durability

The liner material must be able to withstand the physical and chemical environment of the sewage lagoon without significant loss of seepage control properties. Physical failure[1] mechanisms that should be considered in liner design are shown on Table 4.2. In general, suitable protection can be incorporated in the design of each type of liner material to withstand potential failure mechanisms. However, the need for extensive subgrade preparation, earth covers, erosion protection,

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1 Liner failure occurs when the seepage rate through the liner exceeds the maximum allowable value.

and the like will affect the overall cost of the liner system. Typical liner protection requirements are listed in Table 4.3 for common liner materials.

The sideslope area of the liner is generally the most vulnerable to weathering because of exposure, fluctuating water levels, and wave action. In large shallow ponds the sideslope area is only a small portion of the overall area. However, increased seepage through the sideslopes and dykes could lead to surface seepage problems at the dyke toes. In some cases a hybrid liner might be considered, consisting of a weather resistant liner on the side slopes (e.g. HDPE) and a less expensive and less weather resistant material on the bottom (e.g. compacted clay).

The liner material must also be compatible with the wastewater. Typical wastewater properties are listed on Table A12.1 in Appendix 12. In general, wastewater with constituents within the ranges shown on Table A12.1 are not very aggressive toward most liners. Possible concerns with some materials are discussed in Appendices 4 to 8. Wastewaters containing significant volumes of industrial effluents may have constituents in concentrations that are aggressive toward certain liner materials.

For example, oils and hydrocarbons tend to dissolve asphalt-based liners while high salt contents may cause shrinkage in clay and bentonite liners. Table 4.4 indicates the effects of various types of industrial effluents on certain liners. More detailed information for individual liner materials is provided in Appendices 4 to 8 and 14. There is a general lack of information on liner-waste compatibility and the compatibility of the selected liner material should be checked for each specific application. Previous experience at other facilities or liner immersion tests are common ways to test liner/waste fluid compatibility.

TABLE 4.2 RESISTANCE OF LINERS TO PHYSICAL WEATHERING

MECHANISM	COMPACTED CLAY	BENTONITE ADMIX	PVC	HYPALON/CPE	HDPE	HYDRAULIC ASPHALT CONC.	SPRAY-ON BITUMEN
Sunlight, UV	drying cracking	no direct effect	liner breakdown	excellent resistance		accelerated aging	accelerated aging
Wet/dry cycles	shrinkage cracking	resists drying, self-healing	may accelerate aging		no effect	algae growth at water line	algae growth at water line
Freeze, thaw	loss of density	rupture of thin liner	floating ice may damage thin membranes			cracking, rupture	cracking, rupture
Waves	erosion	erosion	possible slumping of subgrade			little erosion	little erosion
Hydrostatic & Gas Uplift	N/A	generally N/A	uplift of membranes if no underdrain/vent			heaving if no underdrain	heaving if no underdrain
Coarse textured subgrade	possible piping	possible piping	liner puncture			irregular liner thickness	difficult and expensive to cover

(CONTINUED NEXT PAGE)

TABLE 4.2 (CONTINUED)

MECHANISM	COMPACTED CLAY	BENTONITE ADMIX	PVC	HYPALON/CPE	HDPE	HYDRAULIC ASPHALT CONC.	SPRAY-ON BITUMEN
Soft subgrades, settlement	tension cracks, poor compaction	tension cracks, rupture	local stressing of liner due to settlement			cracking of brittle liners, poor compaction	cracking or rupture
Installation Damage	poor quality soil, too thin, poor compaction, too dry	application rate too light, poor coverage, mixed too deep	tearing and puncturing, poor seams, stress points due to wrinkles			insufficient asphalt, poor compaction too thin	pinholes, too thin
Clean out of sludge	generally little damage	may damage	may tear liner			generally little damage	may rupture liner
Vegetation	root holes, drying	root holes, drying	some grasses may penetrate thin membranes			cracking	cracking
Wildlife	burrowing	burrowing	puncture, ingestion			little damage	puncture
Vandalism	generally not susceptible		possible puncture			generally not susceptible	possible puncture

TABLE 4.3 RECOMMENDED LINER PROTECTION FEATURES

PROTECTION FEATURE	COMPACTED CLAY	BENTONITE ADMIX	PVC	HYPALON/CPE HDPE	HYDRAULIC ASPHALT CONC.	SPRAY-ON BITUMEN
Earth Cover	No	Yes	Yes	optional	recommended on sideslopes	Yes
Erosion Protection	Yes	Yes	Yes	only if soil cover used	Yes	Yes
Well Compacted Subgrade	Yes	Yes	Yes	Yes	Yes	Yes
Underdrain System	No	optional	Yes	Yes	protects against frost heave if free-draining	protects against frost-heave if free-draining
Smooth Liner Bedding (no gravel)	should meet filter criteria	should meet filter criteria	Yes	Yes	not required	preferred
Level Subgrade	Yes	Yes	Yes	Yes	Yes	Yes
Sterilant Below Liner	No (short root grasses only on surface of liner)	No	Recommended	Recommended	Yes	Yes
Minimum Water Level At All Times To Prevent Freezing Of Bottom	Yes	Yes	Generally Recommended	Generally Recommended	Yes	Yes
Control of Hoofed and Burrowing Animals	Yes	Yes	Yes	Yes	Yes	Yes

TABLE 4.4 LINER-WASTE COMPATIBILITY
(from Kays 1977)

	Type of Lining										
Substance	PE	Hypalon	PVC	Butyl Rubber	Neoprene	Asphalt Panels	Asphalt Concrete	Concrete	Steel	CPE	311(
Water	OK	OK	OK	OK	OK	OK	OK	OK	CP	OK	OK
Animals oils	OK[3]	OK	ST	OK	OK	Q	Q	NR	OK	OK	OK
Petroleum oils (no aromatics)	OK[3]	Q	NR	NR	SW	NR	NR	OK	OK	OK	OK
Domestic sewage	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Salt solutions	OK	OK	OK	OK	OK	OK	Q	NR	NR	OK	OK
Base solutions	OK	OK	OK	OK	OK	OK	OK	Q	OK	OK	OK
Mild acids	OK	OK	OK	OK	OK	OK	OK	NR	NR	OK	OK
Oxidizing acids	NR	NR	NR	NR	Q	NR	NR	NR	NR	NR	NR
Brine	OK	OK	OK	OK	OK	OK	OK	Q	NR	OK	OK
Petroleum oils (aromatics)	Q	NR	NR	NR	NR	NR	NR	OK	OK	NR	NR

[1]OK = generally satisfactory, Q = questionable, NR = not recommended, ST = stiffens, SW = swells, CP = cathodic protectic suggested.
[2]It is recommended that immersion tests be run on any lining being considered for use in an environment where a question exis concerning its longevity. Consult the lining manufacturer or an experienced testing laboratory when in doubt.
[3]Must be a one piece lining.

4.3.5 Costs

The unit cost of liner material supply and installation is an important design consideration. Costs should also be determined for any special subgrade preparation, earth covers, erosion protection, etc., in order to properly compare liner methods. Other factors are the availability of local or on-site materials for use as underdrain, admix, cover material, etc., which may favour the use of a particular liner. Maintenance requirements, life expentancy, and the like should also enter into economic considerations.

Approximate unit costs for liner materials (installed) are shown on Table 4.5 in 1982 Canadian dollars. Factors which may influence costs include site location, the supplier location, the thickness of the liner, tax exemptions, and local contractor rates for earth moving.

4.3.6 Summary Table

There are several pro's and con's for each type of liner material. The site geology or material availability may eliminate some materials and others may be ruled out as not being sufficiently reliable or durable in the particular service environment. The final selection from the few short-listed alternatives is generally based on overall costs.

The following summary table (Table 4.6) of liner materials and properties is a general assessment of liner suitability for typical municipal wastewater ponds. The suitability of any particular material may vary for specific sites, thus the final evaluation must be made by the design engineer. In addition, the summary table is based on available liner technology at the time of preparation of this manual and may not reflect new information, materials, or processes that arise in the future.

TABLE 4.5 INSTALLED LINER MATERIAL COSTS (1982)

LINER MATERIAL	THICKNESS	TYPICAL COST RANGE
Compacted Clay	1 m	$2.50 - $6.00/m^2
Bentonite Admix[1,2]	0.1 m	$2.50 - $6.00/m^2
PVC[1]	20 mil	$3.00 - $5.00/m^2
Hypalon(R)[1]	36 mil	$12.00 - $15.00/m^2
CPE(R)[1]	30 mil	$9.00 - $11.00/m^2
HDPE[1]	80 mil	$10.00 - $12.00/m^2
Asphalt Concrete Plus Spray-on Seal	0.1 m 0.05 m	$15.00 - $18.00/m^2 $9.00 - $12.00/m^2
Soil Asphalt and Spray-on Seal	0.15 m	$7.00 - $10.00/m^2

1. The above costs assume locations in central or southern Alberta with ready access to the site. Subgrade preparation, underdrains, and earth covers are not included.

2. Assumes 10% by weight bentonite to sand ratio. This may vary from a minimum of about 8% to much higher percentages, depending on the quality of the bentonite, the native soil, and level of construction contol.

TABLE 4.6 RATING OF LINER MATERIALS FOR SEWAGE LAGOONS

MATERIAL	SEEPAGE CONTROL	PHYSICAL DURABILITY	CHEMICAL COMPATIBILITY	COST	OVERALL RATING
COMPACTED CLAY	FAIR to VERY GOOD depending on in situ properties	GOOD on bottom, fair on sides	Dispersion may improve, flocculation may fissure	Low if onsite material available	GOOD, if well designed and constructed
BENTONITE/ SAND MIX	FAIR to GOOD depending on thickness, mix ratio, quality control	GOOD if protected from erosion & swelling	GOOD if pre-treated	Medium	FAIR to GOOD if well designed and constructed
SOIL CEMENT	FAIR	Subject to cracks	GOOD	Medium	FAIR if maintained
SOIL ASPHALT	FAIR TO GOOD	Subject to cracks	FAIR	Medium	FAIR if maintained
HYDRAULIC ASPHALT CONCRETE	FAIR TO GOOD	Subject to cracks	FAIR	Med-High	FAIR if maintained
SPRAY ON BITUMEN OVER SOIL ASPHALT OR HAC	FAIR TO VERY GOOD	Subject to cracks	FAIR	High	FAIR to GOOD but expensive

(CONTINUED ON FOLLOWING PAGE)

TABLE 4.6 RATING OF LINER MATERIALS FOR SEWAGE LAGOONS (CONTINUED)

MATERIAL	SEEPAGE CONTROL	PHYSICAL DURABILITY	CHEMICAL COMPATIBILITY	COST	OVERALL RATING
PVC	GOOD	FAIR, should be covered	GOOD if well seamed	Med-High (incl. cover)	GOOD if well-installed
LDPE	GOOD if not damaged	POOR, susceptible to punctures, must be covered	GOOD but seams may deteriorate	Med-High (incl. cover)	POOR
CPE	GOOD	GOOD	GOOD if well seamed	High	GOOD but expensive
HYPALON	GOOD	GOOD	" " "	Very High	GOOD but expensive to very expensive
BUTYL	GOOD	GOOD	GOOD, seams may deteriorate unless welded	Very High	" " "
HDPE	VERY GOOD to EXCELLENT	GOOD to VERY GOOD depending on thickness	VERY GOOD to EXCELLENT	High	VERY GOOD but Expensive
CHEMICAL SEALANTS	POOR to GOOD depending on method of application	FAIR, usually thin	Depends on product Generally GOOD	Med. to High	NOT RECOMMENDED AT PRESENT

4.4 LINER DESIGN PROCEDURE

4.4.1 General

Design of the liner system includes specification of liner thickness, material properties, subgrade requirements, protection features such as soil covers; and design of any necessary underdrain systems. The objective of the design should be to meet the seepage control criterion outlined in Section 4.1.2 and to protect the liner against potential failure mechanisms in a cost effective manner.

Suggested design procedures for typical liner systems are presented in Appendices 4 to 8.

4.4.2 Assessment of Natural In Situ Liners

The potential for native deposits to serve as natural *in situ* liners should be investigated when sites are located in relatively low permeability clay or till. This will require good delineation of the natural liner deposit and assessment of the *in situ* hydraulic conductivity of the deposit. A combination of laboratory and field tests may be necessary to determine the *in situ* hydraulic conductivity (see Appendix 3). However, it should be noted that a study by Olson and Daniel (1981) indicates that the *in situ* value of hydraulic conductivity is likely to be higher than the value measured in laboratory tests because of the effects of macro-structure (fissures, sand and silt lenses, and the like). In 90% of the cases examined by Olson and Daniel, the ratio of K (*in situ*) to K (laboratory) ranged from 0.38 to 64. Therefore, it is recommended in this manual that the *in situ* K value for soil liners be assumed to be at least 10 times the laboratory value of K, unless there is strong evidence to the contrary.

The *in situ* K value of the liner deposit should be measured at the proposed liner depth, i.e., below the proposed invert elevation. It is recommended that the natural liner be no less than 0.9 m thick at any point below the completed pond. All boreholes drilled in the natural liner deposit should be well sealed with bentonite to avoid hydrogeologic windows in the liner.

Consideration must be given to potential seepage through the sideslope areas of the pond. The horizontal hydraulic conductivity of a deposit is generally higher because of layering, sand and silt lenses, etc. In addition, the upper portion of a fine-grained deposit is more likely to be desiccated because of weathering. Therefore it is likely that an engineered liner such as compacted clay will be required in the sideslope area of the excavation. An engineered liner will be required on the inner slopes of any dyke structures.

4.4.3 Prediction of Groundwater Table Mounding

Once the liner system has been designed to meet the seepage control criterion in Section 4.1.2, the potential for mounding of the groundwater table below the lagoon should be assessed. Mounding can result from even a small amount of unsaturated seepage below the liner, as illustrated on Fig. 4.3. The potential for mounding increases if the area of the lagoon is large with respect to the overall depth of the underlying aquifer. If the lagoon is located in a natural liner deposit, or if the *in situ* sediments below the liner have a relatively low hydraulic conductivity, a saturated seepage front will occur below the pond. In this case a mound will always develop, although it may take a long time for the seepage front to reach the groundwater table (Fig. 4.4).

Procedures for predicting the potential height of a groundwater table mound below a lagoon are presented in Appendix 9.

One of the effects of a groundwater table mound that intercepts the bottom of the pond is a reduction in hydraulic gradient through the

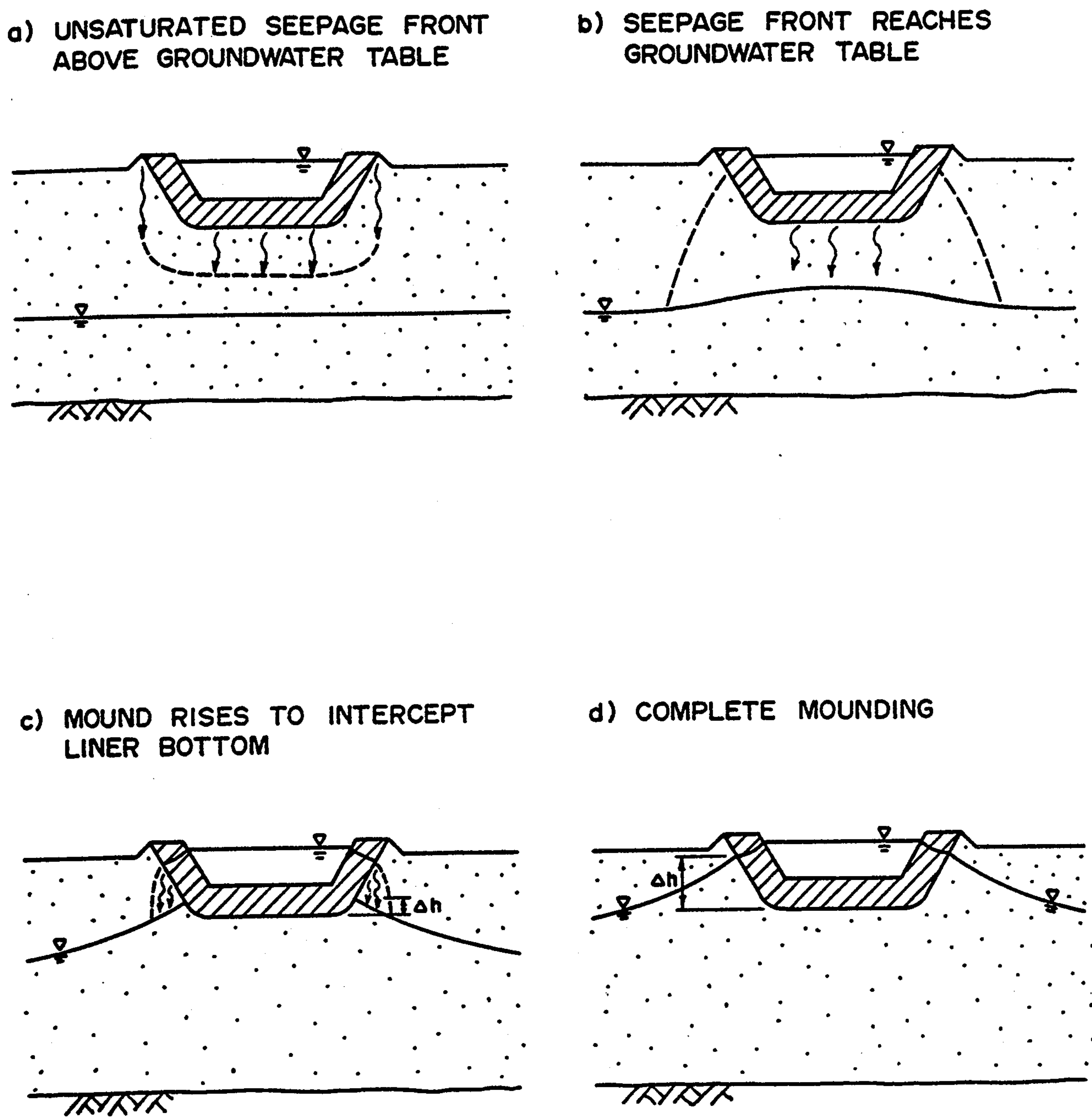
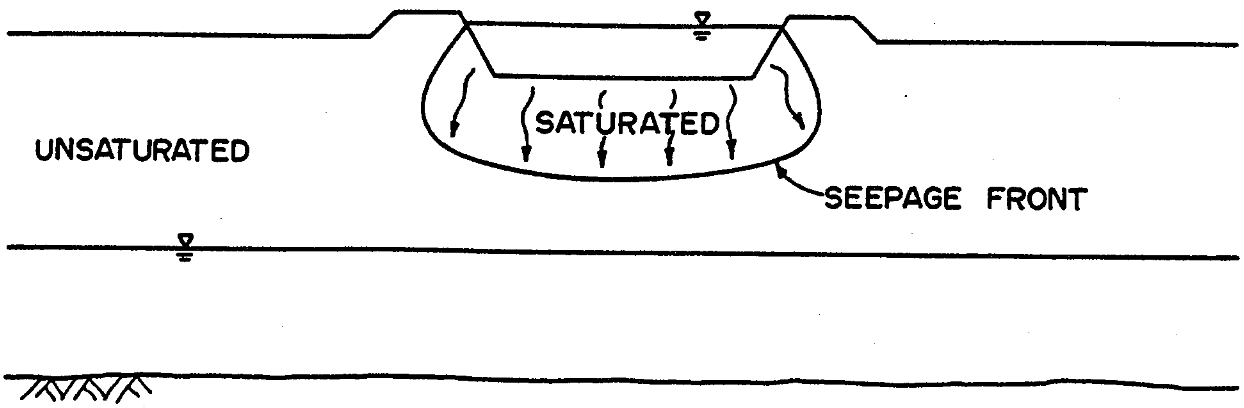
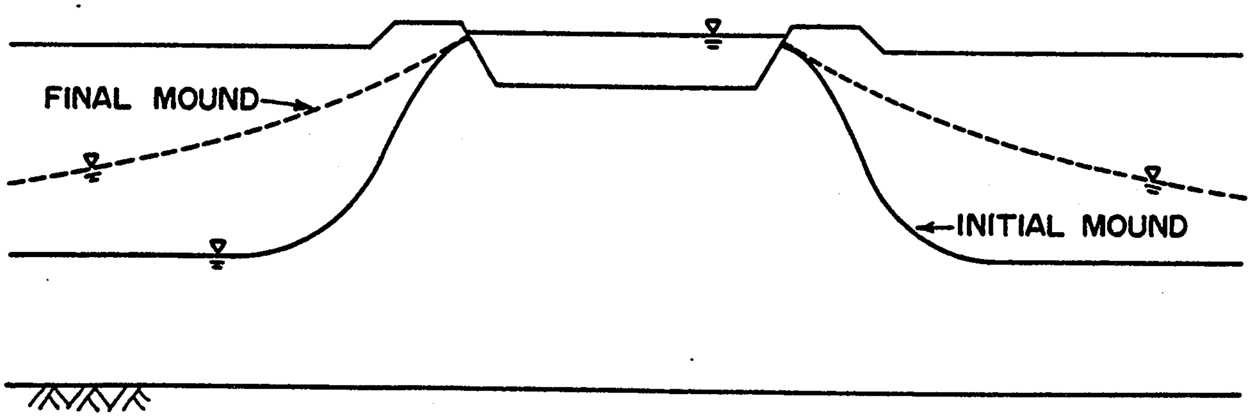


FIG. 4.3 DEVELOPMENT OF A GROUNDWATER TABLE MOUND DUE TO UNSATURATED SEEPAGE BELOW A LINER



a) SATURATED SEEPAGE FRONT ABOVE GROUNDWATER TABLE.



b) SEEPAGE FRONT REACHES GROUNDWATER TABLE, FORMING MOUND

FIG. 4.4 DEVELOPMENT OF GROUNDWATER MOUND DUE TO SATURATED SEEPAGE liner and slower seepage rates. Disadvantages of high mounds include loss of an aeration zone in the ground below the lagoon, vegetation kill, and boggy conditions and seepage at the dyke toes or in nearby depressions.

4.4.4 Control of Seepage at Dyke Toes

Potential seepage at or near the dyke toes may be controlled in two relatively simple ways:

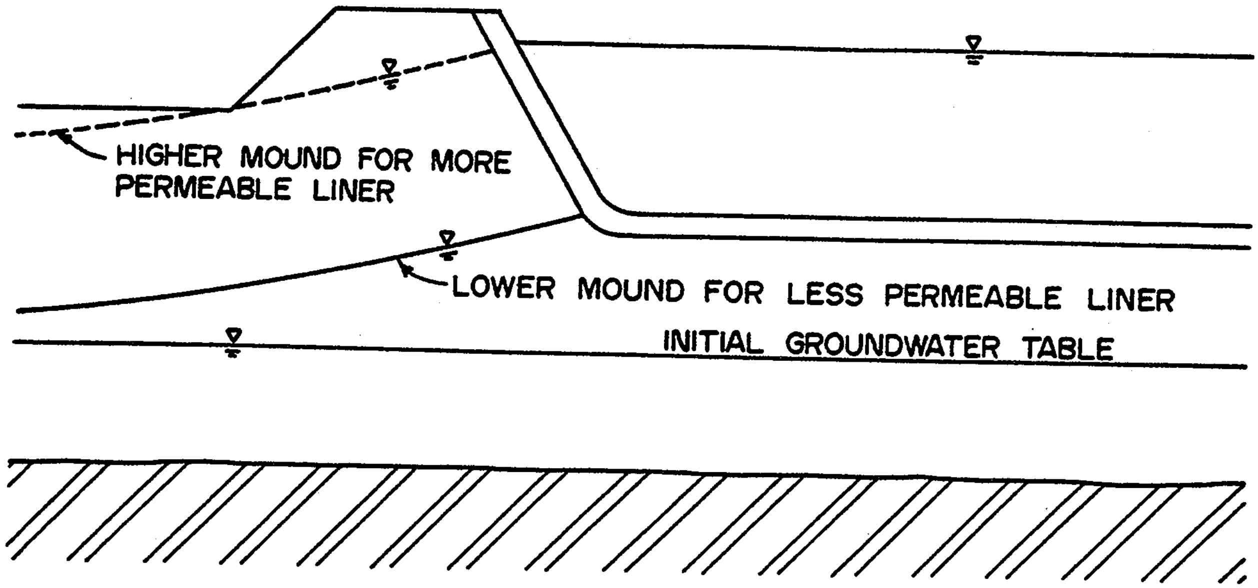
a) reduced seepage by the use of very low permeability liners, e.g., HDPE
b) drainage systems

These are shown schematically on Fig. 4.5. The effectiveness of a drainage system should be assessed by the use of flow nets or computer models. Provisions must be made for collection of seepage from the drainage system. Maintenance may be required if biological activity causes clogging of drains. Cedergren (1976) discusses various procedures for cleaning out clogged drains.

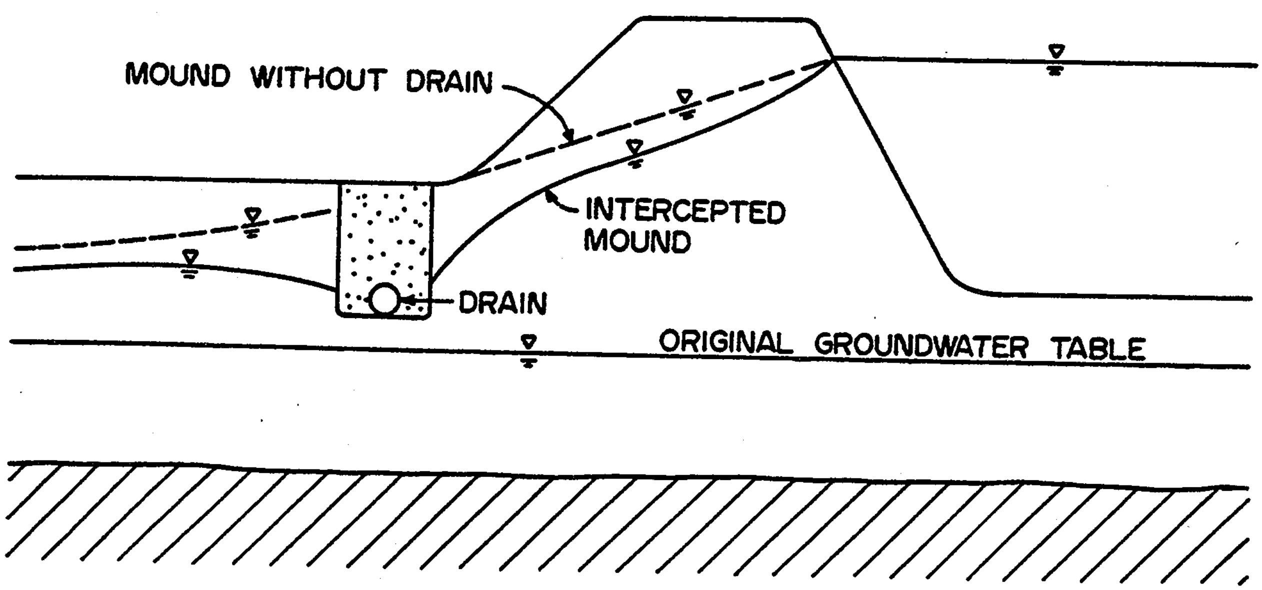
4.5 ASSESSMENT OF IMPACT ON GROUNDWATER QUALITY

4.5.1 General

An evaluation should be made of the capability of the hydrogeological regime to accept and renovate the anticipated seepage quantities without causing degradation of groundwater resources (aquifers), surface waters (streams, lakes, ponds) and land (agricultural, recreational). Assessment of impact includes identifying resources that might be affected, the possible modes of hydraulic communication, and the attenuation or renovation mechanisms. In most cases this will be a qualitative or subjective assessment based on empirical attenuation



a) REDUCED LINER HYDRAULIC CONDUCTIVITY



b) DRAINAGE DITCH

FIG. 4.5 METHODS OF CONTROLLING HEIGHT OF GROUNDWATER MOUND AT DYKE TOE data, calculations of seepage rates and times, simplified models of subsurface conditions and engineering judgement. For more critical cases (e.g. areas with sensitive groundwater uses) it will be desirable to use more sophisticated computer models of contaminant migration. A certain degree of monitoring is required to substantiate predictions and to warn of the need for remedial action.

4.5.2 Attenuation Mechanisms

The primary role of the liner in most cases will be to contain most of the wastewater in the ponds so that it will undergo the required biological treatment process prior to discharge to a stream and renovation in the environment, thus preventing contamination of surface water resources. A secondary role is to limit seepage rates to a level low enough to allow the soil to treat the wastewater.

The major renovation mechanisms of wastewater in the soil are:

- filtration of suspended solids and precipitates,
- adsorption of cations onto clay mineral surfaces,
- dilution of nitrates, anions by diffusion, dispersion, inflow of groundwater from other sources (e.g. due to natural gradients, infiltration of precipitation),
- chemical reaction with gases, organics, metals, etc. in natural groundwater.

These mechanisms are discussed further in Appendix 13, with respect to typical municipal wastewater constituents and potential contaminants. A summary of renovation processes in the soil is shown on Fig. 4.6 (Makeig 1982).

4.5.3 Evaluation Procedure

A plan of the site and vicinity should be prepared showing the lagoon location(s), land use, and resources (e.g. water wells; lakes, streams,

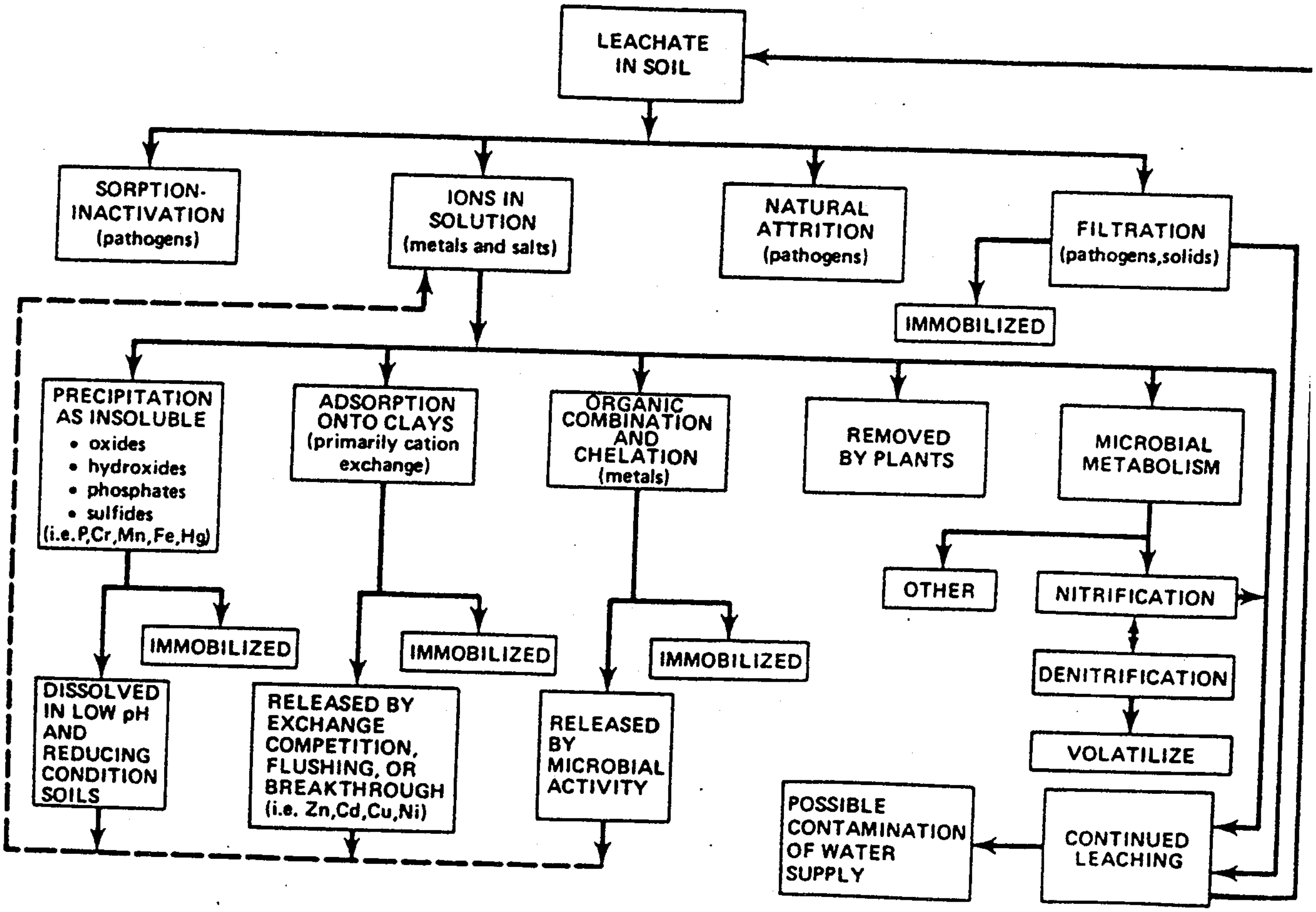


FIG. 4.6 ATTENUATION OF MUNICIPAL SLUDGE LEACHATE IN THE GROUNDWATER SYSTEM (FROM MAKEIG 1982).

ponds, runoff features; agricultural and recreational areas; aquifers and buried channels)

A detailed plan of the site and immediate vicinity should be prepared indicating the direction and gradient of the natural groundwater flow and any significant changes in flow direction and gradient anticipated due to groundwater table mounding.

A discussion should then be prepared outlining the mechanisms that are expected to cause renovation of seeping wastewater and thus prevent pollution of surrounding land and groundwater resources. Remedial work required to prevent pollution of local resources, such as drainage ditches or seepage cut-offs, should be identified on the plan.

4.5.4 Set-Back of Pond From Property Line

The toes of the pond dykes should be set back from the property line a sufficient distance to:

a) allow vehicle access around the dyke perimeter,
b) allow installation of monitor wells, either during construction of the pond or at a later date,
c) allow room for installation of drainage ditches, seepage cut-offs, or dewatering wells, either during pond construction or at later dates as remedial measures.

It is suggested that 15 m be provided as a minimum set back in all cases for operational purposes as outlined above. Alberta Environment requires that existing lagoons be set back 1000 ft. (300 m) from existing dwellings. Other minimum horizontal setback distances from waste stabilization ponds to various land uses are outlined in the Alberta Environment publication entitled Standards for Water Supply Facilities and Sewage Treatment Works.

4.6 EARTHWORKS DESIGN

4.6.1 Balance of Cut and Fill

The depth of the pond cut and height of the pond dykes above original ground level will affect earth moving costs, and may also affect liner requirements. For example, lowering the pond invert may allow the use of a low permeability clay stratum as a bottom liner. Alternatively, raising the pond invert and building higher dykes may keep the bottom of the pond within a suitable clay deposit, or may avoid groundwater table problems. The liner cost savings, however, must be balanced with any additional earth moving costs.

The total volume of earth moved, and the earth moving costs, are approximately minimized by balancing the quantity of cut and fill. For small ponds it may be more advantageous to excavate more of the pond and have some waste. The difference in unit cost of cut and waste versus cut and fill volumes will affect the optimum ratio of excavation depth and dyke height above original ground level.

Charts for quick estimation of cut and fill requirements are presented in Appendix 10. More accurate volume calculations should be made during final design.

4.6.2 Excavation

The geotechnical engineer should be present during excavation of the pond cut to compare soil and groundwater conditions with those anticipated in design, and if a compacted clay liner is specified, to approve suitable liner material.

4.6.3 Fill Placement and Compaction

The following discussion relates to dyke design and construction. Construction of compacted clay liners is discussed in Appendix 4.

The pond dykes should be well constructed to avoid settlement, slumping, and erosion, and to provide good support for liners, erosion protection, and vehicles. Excessive settlements result in loss of freeboard, liner stress, poor trafficability, and unsightly appearance. Slumping and erosion (due to runoff, wave action, and seepage) may jeopardize the integrity of the dyke and could lead to failure in extreme cases.

A typical schematic cross-section is shown on Figure 4.7. Foundation preparation consists of stripping topsoil and any soft, compressible or otherwise unsuitable materials from the dyke area, and proof-rolling the scarified surface to at least 95% of the maximum dry density (ASTM D698). Heavy, self-propelled sheepsfoot rollers should be used in cohesive soils and vibratory smooth-drum rollers should be used in granular soils. Hard, smooth foundation soils should be scarified and re-compacted to ensure a good bond between the fill and foundation soil. It may also be desirable to key large dykes into the foundation soil to cut off potential seepage paths at the dyke/foundation interface.

The dyke fill should be free of organics, organic soil, debris, cobbles over 15 cm diameter, snow, ice, or soft, compressible materials. The fill should be placed in level, uniform lifts in a direction parallel to the dyke axis. Maximum loose lift thicknesses should be 15 to 20 cm, depending on the type of compaction equipment. The fill should be compacted to at least 95% of the maximum dry density (ASTM D698) or as otherwise required by the design engineer. If the dyke consists of separate zones, (e.g. dyke fill, compacted clay liners, drainage material) these zones should be built up at the same rate. Care should be taken to avoid mixing of materials in different zones, and to achieve good compaction at the interfaces. All materials subject to seepage should meet applicable filter criteria, as outlined in most soil mechanics and groundwater texts.

The interior dyke slopes should be no steeper than 3 (horizontal) to 1 (vertical). Flatter slopes may be undesirable for operational reasons, e.g. flat slopes may promote emergent weed growth. The exterior slopes may range from 2.5:1 to 4:1, depending on soil conditions, mowing

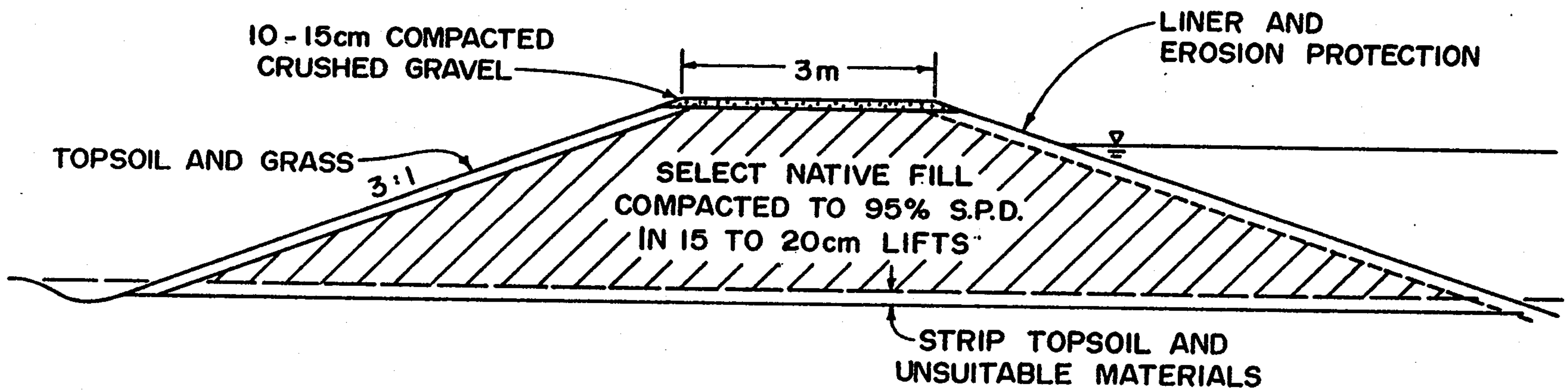


FIG. 4.7 SCHEMATIC CROSS - SECTION OF TYPICAL SEWAGE LAGOON DYKE
(FOR ILLUSTRATION PURPOSES ONLY
AND NOT TO BE USED FOR DESIGN)

requirements, etc. Slopes of 3:1 are common. The exterior slopes should be covered with 15 - 20 cm of seeded topsoil. Shrubs and trees shold not be planted on the dykes.

The dyke crest should be about 3 m wide to allow vehicle access and be topped with 10 - 15 cm of compacted crushed gravel as a trafficable surface. The dyke crest should also have a slight camber to promote runoff.

Minimum freeboards of 0.6 m should be suitable for small ponds (less than 0.5 ha) while 0.9 to 1.2 m may be required for large ponds. Anticipated wave heights for different pond lengths are shown on Table 4.7. Erosion protection requirements are discussed in Section 4.6.4.

TABLE 4.7 WAVE HEIGHTS IN LAGOONS (after Kays 1977)

Wind Speed (km/h)	Lagoon Length (m)	Expected Wave Height (m)
80	300	0.3
130	300	0.5
80	600	0.4
130	600	0.7

4.6.4 Erosion Control Features

4.6.4.1 Compacted Clay Liners

Minimum erosion protection of compacted clay liners should consist of 10 cm of topsoil with an established growth of short-rooted grass down to the minimum water level. Gravel, small riprap, or artificial erosion protection systems should be placed around all piping entrances and exits.

Larger ponds may require additional gravel or riprap protection on the side slopes to prevent wave erosion. The riprap or an acceptable substitute should be placed from the dyke toe to a minimum height of 0.6 m above the maximum water level.

4.6.4.2 Bentonite Admix Liners

Bentonite admix liners should have a soil cover of at least 30 cm to protect the erodable liner material. This soil cover will also require the same erosion protection as compacted clay liners.

4.6.4.3 Asphalt Liners

Soil covers are recommended on the side slopes of HAC and spray-on membrane liners and soil asphalt and spray-on membrane liners. The soil covers should be protected from erosion as in Section 4.6.4.1.

4.6.4.4 Polymeric Membrane Liners

Soil covers are recommended on the side slopes of weather resistant membranes less than 40 mils in thickness, and around piping entrances and exits. Soil covers are recommended over the entire liner area of PVC membranes and over other materials that are susceptible to rapid weathering. Soil cover should be protected from erosion as in Section 4.6.4.1. Where only the sideslopes are covered, the soil cover should extend onto the bottom well beyond the riprap material in order to prevent riprap from washing directly onto the liner material and to facilitate cover placement. Riprap and soil materials should be placed from the bottom upwards using small vehicles on wide pads or by clam-shell.

Polymeric membranes over 40 mils thick, (e.g. high density polyethylene) do not require erosion protection. A soil cover with suitable erosion protection may be desirable to protect the liner sideslopes from vandalism, hoofed animals, etc. or to avoid a slippery side slope surface in wet weather.

4.6.4.5 Exterior Slope Riprap

Riprap may be required on exterior slopes where storm runoff or flooding may cause erosion of the dykes. This will include any facility constructed in a natural runoff channel, or within the flood plain of a river, or as deemed necessary by the design engineer or the Department.

4.7 REPORT TO ALBERTA ENVIRONMENT

4.7.1 General

An engineering report on the design and construction of wastewater stabilization ponds is required as part of the application for a Permit to Construct for a wastewater treatment facility. The engineering report should outline the site selection and investigation procedure, present the liner design, discuss potential environmental impact on surrounding land and water resources, and specify construction and monitoring procedures. Borehole logs and hydrogeological data such as groundwater levels should also be included in the report.

4.7.2 Required Data

4.7.2.1 Plans

The engineering report should include a 1:50,000 scale plan of the site area (5 to 8 km radius minimum) with 7.5 m elevation contours, showing the area to be served by the facility and the proposed site location. The report should also include a larger scale plan of the proposed site showing property boundaries, pond locations, borehole and test pit locations, monitor well locations, drainage courses (receiving waters) and other pertinent information.

4.7.2.2 Data

Data relating to the items listed on Table 4.8 should be included in the engineering report, and shown on the report drawings where practical.

TABLE 4.8 SUGGESTED DATA LIST REQUIREMENTS

	STANDARD STUDY	DETAILED STUDY
TOPOGRAPHY AND LAND USE		
Within 5 km of site:		
topographic contours	X	X
municipal boundaries	X	X
roads	X	X
buildings	X	X
services, pipelines	X	X
parks and recreation areas	X	X
agricultural areas	X	X
mines, open pits, quarries	X	X
oil and gas wells	X	X
landfills, other waste facilities	X	X
SURFACE WATER HYDROLOGY		
Within 5 km of site:		
rivers, creeks, seasonal streams	X	X
lakes, ponds, swamps, springs	X	X
reservoirs, irrigation ditches	X	X
runoff directions in site vicinity	X	X
water quality of water resources		X

(Continued on following page)

TABLE 4.8 (CONTINUED)

	STANDARD STUDY	DETAILED STUDY
GEOLOGY		
Within 5 km of site:		
depth to bedrock	X	X
bedrock type	X	X
extent of weathering in upper bedrock layers	est.	X
bedrock strike and dips (approx.)		X
faults		X
joint planes		X
SOIL		
Within 5 km of site:		
soil types and distribution	X	X
In site vicinity, for each major soil type:		
unified classification	X	X
grain size distribution curves	X	X
density of granular soils	X	X
consistency of cohesive soils	X	X
plasticity of cohesive soils	X	X
weathering, dessication	X	X
extent of fissures, fractures	X	X
hydraulic conductivity	X	X
porosity	X	X
cation exchange capacity		X
sodium absorption ratio		X

(Continued on following page)

TABLE 4.8 (CONTINUED)

	STANDARD STUDY	DETAILED STUDY
HYDROGEOLOGY		
Within 5 km of site:		
water supply well locations	X	X
potable aquifers	X	X
municipal water supply	X	X
monitor wells	X	X
In site vicinity:		
delineate unconfined and confined aquifers	X	X
depth to water table (measured and anticipated)		
maximum	X	X
minimum	X	X
perched water table	X	X
horizontal and vertical flow gradients	X	X
direction of flow	X	X
rate of flow	X	X
recharge and discharge zones	X	X
water quality	X	X
CLIMATOLOGY		
Monthly average:		
precipitation		X
evaporation potential		X
temperature		X
wind velocity		X
Flood frequency (if applicable)	X	X

5. MONITORING REQUIREMENTS AND PROCEDURES

5.1 GROUNDWATER

Groundwater monitoring should be performed to determine the effects of seepage from a sewage lagoon on natural groundwater flow conditions and on groundwater chemistry. In the event that unacceptable conditions develop, monitoring would not only allow the problem to be identified, but would also provide the necessary data on which to base the remedial works program and a rational re-evaluation of the pond design.

In order to adequately assess the impact of a sewage lagoon, both hydrogeological and geochemical parameters should be monitored.

The main hydrogeologic parameter to monitor is depth to water. If depth to water measurements are available at a number of locations, the surface of the water table and groundwater flow directions can be determined. Ideally the monitoring system employed to measure water levels should allow some direct measurement of the hydraulic conductivity of the surficial sediments (see Appendix 3). The results of these in situ measurements of hydraulic conductivity should be compared to other available hydraulic conductivity measurements to provide a best estimate of the hydraulic conductivity. This value could be used to quantify the groundwater flow system.

Increased nutrient levels are the principal groundwater quality problem associated with migrating municipal wastewater. Samples should be collected from sampling stations on a regular basis and submitted to a laboratory for nitrate, nitrogen, phosphorous and coliform analyses.

More detailed information on groundwater monitoring methods is presented in Appendix 11.

5.2 STRUCTURAL INTEGRITY

5.2.1 Dykes

Regular inspections of dyke integrity should be made by the engineer or technician responsible for the lagoon operation. The lagoon designers should provide written guidelines for inspection, including recommended frequency of inspection. Observations should be recorded and any of the following occurrences should be reported to the lagoon designers:

i) Large or localized settlements of the dyke surface;
ii) Failures, ravelling, or other degradation of interior or exterior dyke slopes;
iii) Cracking along the dyke crests or slopes;
iv) Gullying, under-cutting, loss of erosion protection, or any other type of erosion on dyke surfaces;
v) Visible seepage on exterior dyke slopes, particularly when the seepage water is cloudy in appearance or is causing noticeable erosion;
vi) Cloudy or dirty water emerging from underdrain collection pipes;
vii) Burrows in dykes.

Annual inspection of dyke integrity by a geotechnical engineer would be desirable, particularly with large facilities or lagoons in environmentally sensitive areas.

5.2.2 Liner Systems

Most liner systems will require some degree of maintenance over their lifetime (flexible polymeric membranes are often warrantied on a pro-rated basis for 20 to 25 years, although it is likely that many materials, properly installed and protected, will last much longer). Inspections of liner integrity should be made regularly, ideally at low pond levels when more of the slope surface is exposed. The condition of the sideslope liner will likely be a good indication of the liner integrity since the sideslope environment is generally the harshest.

Observations should be made of erosion or slumping of soil liners (compacted clay and bentonite admix) and soil covers. Weather sensitive liners (eg. PVC) can degrade very rapidly when exposed to heat and wetting and drying. Asphalt liners should be inspected for cracking, creep, rupture, and erosion (pitting, exposure of aggregate, etc.). Flexible polymeric membranes should be checked for separated seams, tears, ripping where joined to concrete and steel structures, and general degradation. The tops of folds and along the water line are good areas to inspect for cracking, disintegration, or other noticeable changes in property. Seams should not show signs of peeling when pulled in tension by hand.

Floating of membrane liners may occur due to gas accumulation, hydrostatic uplift, or seepage below the liner. Any indications of liner floating should be reported immediately to the lagoon designers.

Other forms of liner degradation that should be observed and reported are: visible and extensive cracking of compacted clay or bentonite admix liners; heavy weed growth on interior slope surfaces where roots may penetrate, dry-out, or otherwise degrade the liner; lifting of membrane liners due to wind suction.

Inspection of the liner on the lagoon bottom should be carried out whenever the lagoon is emptied. Long term exposure of the bottom liner is not recommended, particularly if it is not weather resistant, if temperatures may fall below freezing, or if drying of clay or bentonite liners could occur.

Appropriate footwear recommended by the liner supplier should be used when walking on flexible polymeric membrane and spray-on liners. Care should be taken to avoid slipping on the liner. In general, walking on wet membrane liner sideslopes should not be permitted.

6.0 REMEDIAL PROCEDURES

6.1 EVALUATION OF EXCESSIVE SEEPAGE

When it is suspected that an operating lagoon has inadequate seepage control to a) operate effectively as designed, b) maintain aesthetic conditions outside the lagoon, or c) prevent degradation of groundwater resources, an evaluation should be made of the extent of the seepage, the impact of the seepage on the environment, and the remedial alternatives.

6.1.1 Observed Performance

There may be several indicators of poor seepage control in operating lagoons. The most evident of these is the inability of the lagoon to retain wastewater. In some cases the volume of wastewater in the lagoon at any time may be significantly less than the inflow volume; in other cases the lagoon may remain dry, indicating that virtually all of the wastewater entering the lagoon has infiltrated the underlying sediments. Another indicator of seepage problems is evidence of seepage discharge around the perimeter of the dykes. This may show up as vegetation kill or wet ground. A more serious indicator of poor seepage control is contamination of local groundwater resources.

6.1.2 Groundwater Monitoring

Groundwater monitoring is usually the best way to evaluate seepage conditions around a lagoon. Degradation of groundwater quality at monitor well locations will indicate the extent and rate of seepage from the lagoon, as well as the degree of renovation occurring in the groundwater/soil system. Care should be taken, however, to ensure that contamination of groundwater at a well is due to seepage from the lagoon and not some other source, such as infiltration of contaminated surface water down the well annulus, contaminated sampling equipment, or another facility. In addition, groundwater contamination may be caused by fertilizers, feedlots, and road salt. In many cases the source of contamination can be identified by the type of contaminants. For example, nitrate is a common indicator of municipal wastewater seepage; however, nitrates may also be present from fertilizers.

The source and extent of contamination is more positively identified by establishing the flow direction and gradient of the groundwater in the contaminated aquifer(s). At least three monitor wells in a triangular configuration will be required to determine flow direction and gradient. In many cases additional monitor wells will be required to establish the areal and down-gradient extent of the contaminant migration. These additional wells may be installed after contamination is indicated by the original wells.

Once the flow direction and gradient of the seepage is established, the average K value of the soil deposit may be back-calculated from the time required for the seepage front to reach the monitor wells. This information can be compared to assumptions made during the pond design, and can be used to estimate the seepage rate and probable extent of seepage with time. In some cases the seepage may be localized (e.g., due to the presence of a sand lens). If an adequate number of wells are installed to define the shape and position of the seepage plume, then the general area of seepage in the lagoon may be identified.

6.1.3 Geophysical Methods

Geophysical monitoring methods such as resistivity surveys may be used to detect zones of contamination if the contaminants sufficiently alter the electrical properties of the groundwater. In general, a high total dissolved salts (TDS) content in the contaminated water will allow delineation of the seepage plume. In many cases, however, the TDS in municipal wastewater may be too low to allow differentation from the natural groundwater.

The advantage of geophysical surveys is that they allow relatively continuous coverage of a large area in a short time. Small, localized zones of seepage that might not be intercepted by a monitor well can be detected by geophysical surveys. Geophysical surveys can also be useful for planning the location of additional monitor wells.

6.2 REPAIR OF LINERS

6.2.1 Compacted Clay Liners

The repair of a compacted clay liner may include repair of physical degradation due to wave erosion or slumping, as well as improvement of the seepage control properties of an intact compacted clay liner.

Repair of physical damage to a compacted clay liner will entail draining of the pond (at least to a level that is below the damaged area) and allowing the drained surface to dry out sufficiently to allow access of construction vehicles. Discing of the soil and/or the use of wide pads may allow earlier access to the pond. Sludge, muck, or any other materials unsuitable for liner construction should be stripped from the repair area. Existing clay liner material may be used for repair of the damaged area, but it will likely be necessary to allow the soil to dry to a moisture content suitable for compaction. In general, the cause of the damage should be identified so that suitable measures can be taken to prevent a re-occurrence.

Repair measures to improve the seepage control properties of an intact, compacted clay liner might include locating and replacing zones of permeable material, increasing the liner thickness, or discing and re-compacting the liner surface to increase the soil density or eliminate fissures. Locating zones of poor quality material can be difficult, particularly if the liner has been in use for some time and sludge has accumulated on the surface. Increasing the liner thickness may be effective if the original liner was thin; however, if the additional thickness of liner has the same K value as the original liner, then the decrease in seepage rate will only be inversely proportional to the increase in liner thickness. In general, recompacting the upper surface of a clay liner will not significantly improve the seepage control properties of the liner, since the depth of compaction is limited and the decrease in K value will likely be less than an order of magnitude (Blight 1966). Recompaction of the liner surface may have the effect of restoring the liner to its original thickness if weathering has fissured the upper portion of the liner. Again, the resultant decrease in seepage rate is inversely proportional to the increase in effective liner thickness. In most cases the clay liner is likely to have swollen while in service, making compaction of the existing material difficult. It may also be difficult to compact a new layer of clay on top of an existing, swollen liner.

Chemical additives have been used to reduce the *in situ* K value of cohesive soils (see Appendix 8). The effectiveness of these additives, however, is very dependent on clay and pore water chemistry. Laboratory tests should be carried out using the *in situ* soil, and the effect of reduction of K on the seepage rate should be calculated. Although chemical additives can be effective in reducing the K value of moderately permeable soils, it may be difficult to reduce K to the level required by the seepage control criterion in Section 4.1.2.

6.2.2 Bentonite Admix Liners

It may be more difficult to repair physically damaged bentonite admix liners than to repair compacted clay liners, primarily because admix liners are thin. All materials must be removed in the damaged area, the subgrade must be restored to the required lines and grades, and a new admix layer must be placed or created so that it is continuous with the undamaged portion of the original liner. In some cases damage to a bentonite admix liner may indicate that the liner is unsuitable for the particular service environment. In any case, the cause of liner damage must be identified so that a re-occurrence can be prevented.

Improving the seepage control properties of a bentonite admix liner will likely require the application and mixing of additional bentonite. This can only be carried out if the admix liner is not covered with a protective soil layer. Thus, it is important that the adequacy of a bentonite admix liner be well established prior to placement of the soil cover.

6.2.3 Asphalt Liners

The most common damage to asphalt liners such as HAC is likely to be cracking due to cold, ageing, or frost action, particularly in the exposed sideslope area. Unless the extent of cracking is severe (e.g., vertical displacements along cracks or loss of material due to erosion), repair of the liner will likely entail the application of bitumen in the crack areas.

6.2.4 Flexible Polymeric Membrane Liners

In many cases flexible polymeric membranes are probably the easiest to repair because of accessibility, relative ease of locating damaged areas, and ease of most repairs. Small punctures and tears will require a patch (using the same material and proper bonding techniques). Separated seams may be re-seamed, or covered with a strip of new material. More extensive damage due to wind or flotation may require the replacement of one or more sheets of liner material, or in extreme cases, replacement of the entire liner. These repairs will be very costly unless covered by a warranty.

Some materials vulcanize or cross-link with time (e.g. Hypalon), so that the material becomes difficult to seam and repair.

Thermoplastic materials (e.g., HDPE) may become highly stressed in cold weather if insufficient slack is left in the material during installation. This results in lifting of the liner at the toe of the slope and may cause over-stressing and splitting of seams. Repair of these liners will require the addition of filler strips to provide more slack in the liner.

In all cases it is important that the cause of the damage be identified to prevent a re-occurrence. For example, separated seams may be due to poor seams, stressing, chemical incompatibility between the adhesive and the wastewater, or any combination of the above factors. The original supplier and installer should perform the work, particularly if the liner is under warranty. All repairs should be made using the same material as the liner and proper bonding techniques.

6.3 RE-LINING OF EXISTING LAGOONS

This Section discusses design and construction considerations when flexible polymeric membranes are used to line existing lagoons that originally have inadequate native or compacted soil liners.

Once the lagoon has been drained the existing soil liner or in situ sediments must be re-worked to form a smooth and firm support for the flexible polymeric membrane and any required underdrain system. This will generally entail allowing the soil to dry, perhaps in conjunction with stripping of sludge and muck and discing of the soil to increase the rate of drying. Consideration should be given to disposal of any waste-contaminated materials. Possible alternatives include disposal in a nearby sludge lagoon or other applications as approval by the Department (landspreading, landfilling).

The existing soil liner or native sediments are likely to be relatively soft and swollen due to inundation and may be difficult to work and/or compact. A subgrade density of at least 90% of maximum dry density (ASTM D698) should be adequate for most facilities, although a minimum of 95% of maximum dry density is preferred. Moisture contents of undisturbed samples from the liner should indicate the degree of compaction likely to be attainable in the field, based on the moisture - density relation (ASTM D698).

In some cases the existing lagoon invert may be below the natural groundwater table. Resultant inflow of groundwater (after attempting to drain the pond) will make subgrade restoration impossible unless the subgrade is adequately dewatered. This in itself can be an expensive and difficult proposition. Possible solutions may include installing a french-drain system, or placing free-draining gravel over a geotextile layer. The gravel should be dewatered so as to act as a blanket drain, and ideally would consolidate the underlying subgrade and serve as a firm support for the new polymeric liner. The geotextile layer separates the subgrade and gravel and also improves the support capacity of the gravel layer if the subgrade is soft.

Underdrainage, gas venting, and liner bedding requirements must be met in the re-lined lagoon, as discussed for new facilities in Appendix 7.

7. REFERENCES

AISENSTEIN, B., DIAMANT, E, and SAIDOFF, J. 1961. Fat clay as a blanketing material for leaky reservoirs. Proc. 5th Int. Conf. on Soil Mechanics and Foundation Engineering, Vol. 2, pp 523 - 529.

AITCHISON, G.D. and WOOD, C.C. 1965. Some interactions of compaction, permeability, and post-construction deflocculation affecting the probability of piping failure in small earth dams. Proc. of the 6th Internation Conference on Soil Mechanics and Foundation Engineering, Canada, Vol. 2, pp 442 - 446.

ANDERSON, D. and BROWN, K.W. 1981. Organic leachate effects on the permeability of clay liners. In Land Disposal: Hazardous Waste, Proc. of the Seventh Annual Research Symposium, U.S. Environmental Protection Agency, pp 119 - 130.

ANDERSON, D.C., BROWN, K.W. and GREEN, J. 1982. Effect of organic fluids on the permeability of clay soil liners. In Land Disposal of Hazardous Waste, Proc. 8th Annual Research Symposium, U.S. Environmental Protection Agency, Ft. Mitchell, KY, pp 179 - 190.

ASPHALT INSTITUTE. 1974. Asphalt for wastewater retention in fine-sand areas. MISC-74-3, May.

BARDEN, L. 1974. Consolidation of compacted and unsaturated clays. Geotechnique 24, No. 4, p 267.

BIANCHI, W.C. and MUCKEL, C. 1970. Groundwater recharge hydrology. U.S. Department of Agriculture, Agriculture Research Service, ARS 41-161.

BILTON, G. 1975. Adsorption of viruses onto surfaces in soil and water. Water Resources (9), pp 473 - 484.

BISHOP, A.W. and GIBSON, R.E. 1963. The influence of the provisions for boundary drainage on strength and consolidation characteristics of soil measured in the triaxial apparatus. ASTM STP 361, American Society for Testing and Materials, Philadelphia, pp 435 - 451.

BLIGHT, G.E. 1966. Seepage from clay-lined ponds. Civil Engineer in South Africa (8) 7, pp 215 - 221.

BOLTON, R.L. and KLEIN, L. 1971. Sewage treatment, basic principles and trends. 2nd Edition, Ann Arbor Science Publishers, Ann Arbor, Michigan.

BOUWER, H. 1964. Resistance networks for solving groundwater problems. Groundwater (2) 3, pp 26 - 32.

BOUWER, H. 1979. Groundwater Hydrology. McGraw-Hill Book Co., New York, NY.

CAMPBELL, M.D. and LEHR, J.H. 1973. Water Well Technology. McGraw-Hill Book Co., New York, NY.

CEDERGREN, H.R. 1976. Seepage, Drainage, and Flownets, 2nd Edition. John Wiley and Sons, New York, 534 pp.

CHERRY, J.A. and JOHNSON, P.E. 1982. A multilevel device for monitoring in fractured rock. Groundwater Monitoring Review, Vol. 2, No. 3, pp 41 - 44.

FOLKES, D.J. 1982. Fifth Canadian Geotechnical Colloquium: control of contaminant migration by the use of liners. Canadian Geotechnical Journal, 19, pp 320 - 344.

FREEZE, R.A. and CHERRY, J.A. 1979. Groundwater. Prentice-Hall, Inc., Englewood Cliffs, N.J.

GARCIA-BENGOCHEA, I., LOVELL, C.W. and ALTSCHAEFFL, A.G. 1979. Pore distribution and permeability of silty clays. ASCE Journal of the Geotechnical Division, Vol. 105, No. GT7, pp 839 - 856.

GORDON, B.B. and FORREST, M. 1981. Permeability of soils using contaminated permeant. In Permeability and Groundwater Contaminant Transport, ASTM STP 746, Zimmie and Riggs, Eds., American Society for Testing and Materials, pp 101 - 120.

GOODALL, D.C. and QUIGLEY, R.M. 1977. Pollutant migration from two sanitary landfills near Sarnia, Ontario. Canadian Geotechnical Journal, 14, pp 223 - 236.

GRIFFIN, R.A. and CHIAN, E.S.K. 1979. Attenuation of water-soluble polychorinated biphenyls by earth materials. Illinois State Geological Survey, Environmental Geology Notes, No. 86.

GRIFFIN, R.A. and CHOU, S.F.J. 1980. Attenuation of polybrominated biphenyls and hexachorobenzene by earth materials. Illinois State Geological Survey, Environmental Geology Note 87.

GRIFFIN, R.A. and SHIMP, N.F. 1978. Attenuation of pollutants in municipal landfill leachate by clay minerals. Illinois State Geological Survey, prepared for U.S. Environmental Protection Agency, Report No. EPA-600/2-78-157.

GRIM, R.E. 1968. Clay Mineralogys 2nd Edition, McGraw-Hill Book Co., Inc., New York, NY.

GRIM, R.E. and GUVEN. N. 1978. Bentonites: Geology, Mineralogy And Uses. Elsevier Scientific Publishing Co., NY.

GUNKEL, R.C. 1981. Membrane liner systems for hazardous waste landfills. In Land Disposal: Hazardous Waste, Proc. of the Seventh Annual Research Symposium, U.S. Environmental Protection Agency, pp 131 - 139.

HAGEDORN, C., McCOY, E.L. and RAHE, T.M. 1981. The potential for groundwater contamination from septic effluents. Jn. of Environmental Quality (10), pp 1 - 8.

HAXO, H.E., Jr. 1981. Durability of liner materials for hazardous waste disposal facilities. In Landfill Disposal: Hazardous Waste, Proc. of the Seventh Annual Research symposium, U.S. Environmental Protection Agency, pp 140 - 156.

HVORSLEV, J. 1948. Subsurface exploration and sampling of soils for civil engineering purposes. Report of Committee on Sampling and Testing, ASCE.

JOHNSON. 1972. Groundwater And Wells, 2nd Edition. Johnson Division, Universal Oil Products Co., St. Paul, Minn. 55165.

JOHNSON, V.R.. 1983. Monitoring of groundwater at cyanide leach mining sites. Groundwater Monitoring Review (3)1, pp 144 - 147.

KAYS, W.B. 1977. Construction of linings for reservoirs, tanks, and pollution control facilities. John Wiley and Sons, New York, NY.

KIRBY, P.C. 1979. The selection of membrane materials for water distribution and allied applications. Water Research Centre, Technical Report 113.

KUGELMAN, I.J. 1976. Status of advanced waste treatment. In Handbook of Water Resources and Pollution Control, Gehm and Bergman, Eds., Van Nostrand-Reinhold.

LANCE, J.C. 1975. Fate of nitrogen in sewage effluent applied to soil, ASCE Jn. Irrigation and Drainage Division, (101)IR3, pp 131 - 145.

MAKEIG, K.S. 1982. Natural buffers for sludge leachate stabilization. Groundwater (20), No. 4, pp 420 - 429.

MATRECON, INC. 1980. Lining of waste impoundment and disposal facilities. Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Report SW-870.

McWHORTER, D.B. and NELSON, J.D. 1979. Unsaturated flow beneath tailings impoundments. ASCE Journal of the Geotechnical Engineering Division, Vol. 105, No. GT.11, pp 1317 - 1334.

METCALF AND EDDY, INC., 1979. Wastewater Engineering, 2nd Edition. McGraw-Hill Book Co.

MITCHELL, J.K. 1976. Fundamentals of Soil Behaviour. John Wiley and Sons, Inc., New York, 422 pp.

MITCHELL, J.K., HOOPER, D.R., and CAMPANELLA, R.G. 1965. Permeability of compacted clay. ASCE Journal of the Soil Mechanics and Foundations Division, 91, SM4, pp 41 - 65.

MOORE, B.E., SAGIK, B.P. and SORBER, C.A. 1981. Viral transport to groundwater at a wastewater land application site. Jn. Water Pollution Control Fed. (53) 10, pp 1492 - 1502.

MORRISON, W.R., DODGE, R.A. and MERRIMAN, J. 1971. Pond linings for desalting plant effluents (supplement). Office of Saline Water, U.S. Department of the Interior, Report No. 734.

NEUMAN, S.P. and WITHERSPOON, P.A. 1970. Finite element method of analyzing steady seepage with a free surface. Water Resources Research, 6(3), pp 689 - 697.

NRC. 1982. Report to NRC Associate Subcommittee on Geotechnical Research by Task Force on Geotechnical Aspects of Waste Disposal, February, (unpublished).

OLSON, R.E. and DANIEL, D.E. 1981. Measurement of the hydraulic conductivity of fine-grained soils. In Permeability and Groundwater Contaminant Transport, ASTM STP 746, Zimmie and Riggs, Eds., American Society for Testing and Materials, pp 18 - 64.

PATERSON, R.J., FRAPES, K., DYKES, L.N. and McLEOD, R.A. 1976. Coring and squeezing techniques for the detailed study of subsurface water chemistry. Canadian Journal of Earth sciences (15), pp 162 - 169.

PAWLUK, S. and BAYROCK, L.A. 1969. Some characteristics and physical properties of Alberta tills. Bulletin 26, Research Council of Alberta, Edmonton, Alberta, 72 p.

PECK, R., HAWSON, W. and THORNBURN, T. 1974. Foundation Engineering, 2nd Edition. John Wiley & Sons, NY.

PICKENS, J.F., CHERRY, J.A., GRIZAK, G.E., MERRITT, W.F. and RISTO, B.A. 1978. A multilevel device for groundwater sampling and piezometric sampling. Groundwater (16)5, pp 322 - 327.

PREUL, H.C. 1968. Contaminants in groundwater near waste stabilization ponds. Jn. Water Pollution Control Fed., (40), 4, pp 659 - 669.

REID, CROWTHER & PARTNERS LIMITED. 1980. Hazardous wastes in Alberta: An inventory and review of practices and technology, Vol. 1: Inventory and practices. Report to Alberta Environment.

SABEY, B.R., 1973. Soil organisms: their role in soils used for waste recycling. In Proc. Symposium on Land Treatment of Sec. Effluent, CSU, Fort Collins, Colorado, pp 96 - 125.

SCAFE, D.W. 1975. Alberta bentonites. Economic Geology Report No. 2, Alberta Research Council, Edmonton, Alberta, 19 p.

SEED, H.B., WOODWARD, R.J. and LUNDGREN, R. 1962. Prediction of swelling potential for compacted clays. Jn. Soil Mechanics and Fnd. Div., ASCE (90) SM4, pp 107 - 131.

SKEMPTON, A.W. 1964. Long term stability of clay slopes. Geotechnique (14), p 77.

STEWART, W.S. 1978. State-of-the-art study of land impoundment techniques. Exxon Research and Engineering Company, prepared for U.S. Environmental Protection Agency, Report No. EPA-600/2-78-196.

TERZAGHI, K. and PECK, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition. John Wiley and Sons, New York, NY.

THOMAS, R. and LAW, J.P. 1977. Properties of Wastewaters. In Proc. of Symposium on Soils for Management of Organic Wastes and Wastewaters, Elliot and Stevenson, Eds., Soil Science Society of America, pp 47 - 72.

THOMPSON, E. 1983. Unpublished report to Komex Consultants Ltd., by Pavement Management Systems.

TODD, D.K. 1959. Groundwater Hydrology. John Wiley and Sons, New York, NY, 1959.

U.S. BUREAU OF RECLAMATION 1981. Performance of granular soil covers on canals, REC-ERC-81-7, Engineering and Research Center, Bureau of Reclamation, August, 62 p.

U.S. BUREAU OF RECLAMATION. 1974. Earth Manual, 2nd Edition. U.S. Department of the Interior United States Government Printing Office, Washington, D.C., 810 p.

WAGNER, A.A. 1957. The use of the unified soil classification system by the Bureau of Reclamation. Proc. of the 4th International Conference on Soil Mech. Found Eng., London, Vol. 1, p 125.

WILSON, L.G., CLARK, W.L. III, and SMALL, G.G. 1973. Subsurface quality transformations during the initiation of a new stabilization lagoon. Water Res. Bulletin (9)2, pp 243 - 257.

YONG, R.N. and WARKENTIN, B.P. 1975. Soil Properties and Behaviour. Geotechnical Engineering 5. Elsevier Scientific Publishing Co., New York, 449 pp.

APPENDIX 1
SOURCES OF AVAILABLE INFORMATION

Published information pertaining to factors affecting the design and construction of municipal wastewater treatment facilities in Alberta is available through a number of sources, as outlined in the following Sections. Some areas of the province are covered in greater detail than others with regards to this information. Many information sources are outlined in the Natural Resources Information Directory, published by and available from the Resource Evaluation and Planning Division, Alberta Department of Energy and Natural Resources. This publication, revised annually, describes available information sources, gives locations and contacts for these sources, and in each case details those parts of the province that are covered.

A1.1. GENERAL TOPOGRAPHIC INFORMATION

A1.1.1 Topographic Maps

The major sources of topographic information are the National Topographic System (NTS) maps. These maps detail spot elevations, elevation contours, surveys, access, hydrography, vegetation, and cultural features. Complete coverage of the province is available in 1:1,000,000, 1:500,000 and 1:250,000 scales, and except for some northeastern areas, 1:50,000 scale. Some maps are available at 1:25,000 scale. NTS maps are available at the Institute of Sedimentary and Petroleum Geology, Calgary, and at offices of the Map and Air Photo Distribution Services, Alberta Energy and Natural Resources.

Topographic maps are also available for Provincial Parks (Parks Mapping and Survey Unit, Alberta Recreation and Parks), rural municipalities (Duplication and Stock Advance, Alberta Municipal Affairs) and urban centres (Alberta Urban Map Series, from Bureau of Surveys and Mapping, Alberta Energy and Natural Resources).

A1.1.2 Aerial Photography

A listing of all provincial aerial photography is contained in the Aerial Photography Project Index, copies of which are available from Map and Air Photo Services, Alberta Energy and Natural Resources. An alternate source is the National Air Photo Library, Institute of Sedimentary and Petroleum Geology.

A1.2 LAND OWNERSHIP, USE AND PLANNING

Information on land ownership, including legal property descriptions, is available through the Regional Land Titles Offices in Edmonton and Calgary.

Land use and zoning information is available from Alberta Municipal Affairs. Detailed information concerning development, general plans, environmental and engineering studies, water supplies, etc., are available for municipalities covered by Regional and Municipal Planning Commissions. Land use and zoning bylaw information is also available for various districts not covered by planning commissions. Descriptions of environmentally-sensitive areas in Alberta are included in the Restricted Development Areas File, compiled by the Restricted Land Use Branch, Land Assembly Division, Alberta Environment.

Maps showing land use have been developed by Canada Land Inventory, Lands Directorate, Environment Canada, and are available through Map and Air Photo Distribution Services, Alberta Energy and Natural Resources.

The Alberta Land Use Forum is a compilation of public hearings and reports on land use which is available for most cities and large towns through the Alberta Energy and Natural Resources Library.

The Land Classification Section, Department of Energy and Natural Resources, Alberta, publishes Physical Land Classification Maps and Special Geomorphic Maps as a data base for resource planning and management.

A1.3. CLIMATIC INFORMATION

Climatic data is available for much of Alberta from a number of sources. Perhaps the most thorough collection of data is the Climatological Records File (compiled by the Atmospheric Environment Service, Environment Canada) in which hourly temperature and precipitation data are recorded for an extensive network of climatological stations. Sky conditions, visibility, atmospheric pressure, ambient and dewpoint temperatures, and wind speed and direction are recorded at principal stations. The above information is compiled in the following publications distributed by the Atmospheric Environment Service:

1) Monthly Record of Meteorological Observations
2) Canadian Weather Review
3) Canadian Normals
4) Climatic Perspectives

The Civil Aviation Branch, Transport Canada, compiles Meteorological Reports for various Alberta airports. These reports are available at the Transport Canada Library, Edmonton.

Detailed analyses and summaries of precipitation and lightning data from numerous stations in the province are made by the Alberta Forest Service, Alberta Energy and Natural Resources.

Thematic Resource Maps have been compiled by the Bureau of Surveys and Mapping, Alberta Energy and Natural Resources. These include summary maps showing precipitation, snowfall, mean temperatures, sunshine and frost. These and other climatic summary maps are found in the Atlas of Alberta and the National Atlas of Canada, available at most libraries.

A1.4. SURFACE WATER HYDROLOGY

River basin studies conducted by the Planning Division, Alberta Environment, give information regarding water resource development, control and protection in various river basins. Flood plain studies by the Technical Services Division, Alberta Environment, include historical flooding and hydrology data and flood plain analyses for several locations in the province. Information on both of the above studies are available at the Alberta Environment Library, Edmonton.

The Mines Branch, Department of Energy, Mines, and Resources published a series of Reports on the Industrial Water Resources of Canada covering many major watersheds in the years 1950-1965.

The Inland Waters Directorate, Department of the Environment, publishes Technical Bulletins, Reports, Surface Water, Historical Stream Flow Summaries, Guidelines for Surface Water Quality, Flood Damage Studies and series of scientific papers available from Ottawa (K1A OE7) and from their Regional Office in Calgary.

A1.5. GEOLOGICAL INFORMATION

A1.5.1 Bedrock Geology

Bedrock geology maps, outlining the distribution, lithology, age and structure of rock formations near the surface, are available from the Geological Survey of Canada (through the Institute of Sedimentary and Petroleum Geology) and the Alberta Research Council. The entire province is covered by small scale maps; however, larger scale maps are available for specific areas.

The Geological Survey of Alberta publishes bulletins and maps of bedrock topography, hydrogeology and surficial geology.

Bedrock topography maps covering South and Central Alberta are available from the Alberta Research Council. These maps contain elevation contours of various bedrock layers, thalwegs of buried valleys, and thicknesses of surficial deposits.

A1.5.2 Surficial Geology

Maps indicating the type, distribution, and origin of deposits overlying bedrock are available from the Geological Survey of Canada, and the Alberta Research Council. Coverage of the province is incomplete; however, maps are available for most of Southern Alberta.

Maps that summarize soil survey data are published in several series by the Alberta Research Council and Alberta Institute of Pedology. The Reconnaissance Soil Survey Map Series gives detailed information for large areas of Southern Alberta. The Exploratory Soil Survey Series gives broad regional classifications of soil types in Northern and West-Central Alberta. The Soil Survey of Urban Areas series includes maps with detailed soil information for areas of many towns and cities of Alberta.

The Urban Geology File of the Alberta Research Council gives detailed reports and maps, extensive drill hole records, field notes, and analyses of mineralogical, textural, and engineering features for the Edmonton area. A similar file for the Calgary area is in preparation.

The Alberta Research Council maintains a computer-accessible file of soil profiles (CANSIS). The file gives information on location, climate, vegetation, and soil morphology, as well as physical, chemical, engineering and mineralogical data.

A1.6. HYDROGEOLOGY AND GROUNDWATER INFORMATION

Groundwater and water supply information is collected by several divisions of Alberta Environment. The Earth Sciences Division maintains two files. The Water Well Inventory File details well locations and owners, the use and chemistry of the water, and geologic information for all known water wells in the provinces. A general file of water supplies, environmental impact, groundwater licensing, landfills, sewage effluent, soil drainage, seepage, and deep well disposal sites is also kept by the Earth Sciences Division. The Water Resources Management Division collects water well drillers' reports which detail the location, depth, amount and quality of water, and ground formations for each industrial and municipal well drilled. The Pollution Control Division maintains the Water Quality Surveys File and Water Quality Surveys of Major River Systems in Alberta File, in which chemical properties and bacterial content of water samples emitted from industrial and municipal waste treatment facilities are monitored.

A Central Groundwater Data File is maintained by the Earth Sciences Division, Alberta Environment. This file contains information on groundwater availability, production suitability, legal descriptions, drillers' logs, water analysis, water flows and completion reports for individual wells. Information on the above file may be obtained from the Groundwater Resources Information Centre, Edmonton.

The Groundwater Department, Alberta Research Council, publishes two series of maps of groundwater information. The Hydrogeological Reconnaissance Map Series indicates groundwater probabilities, expected well yields, chemistry and flow systems, and covers the entire province. The Hydrogeology, Groundwater Geology and Related Maps series show the distribution, amounts and quality of groundwater in surficial deposits and bedrock. Piezometric surfaces and water-well locations are also shown. The latter series gives scattered coverage of the province.

A1.7. REPORTS ON WASTE MANAGEMENT EXPERIENCE IN ALBERTA

Two publications contain helpful information for the design of waste treatment facilities in Alberta. Management and Disposal of Hazardous Waste in Alberta, by the Environmental Council of Alberta, is a collection of information relating to waste management in the province.

The Solid Waste Management Branch, Environment Canada, has a bibliographic data base, SOL, containing references to solid waste management recovery and utilization projects, both government and private. This data base is accessible through information systems at the Alberta Research Council, University of Calgary Library, and the Calgary and Edmonton Public Libraries.

APPENDIX 2
IN SITU TESTING OF SOIL PROPERTIES

A2.1 STANDARD PENETRATION TEST (SPT)

The SPT is performed in conjunction with split spoon sampling, and provides a valuable index parameter and profiling mechanism in granular soils. The resultant blow-count (N value)[1] is empirically related to relative density, friction angle, and other engineering parameters. Although the test is subject to many errors, it will indicate the variation in conditions across the site when performed by the same rig and crew. See the Canadian Foundation Engineering Manual (1978) for a thorough discussion of the SPT. Correlations with relative density and consistency are shown on Table A2.1.

A2.2 FIELD VANE SHEAR TEST

Field vane shear tests can be used to evaluate undrained shear strength conditions in soft to stiff clays. Vane tests are generally carried out between Shelby tube samples. See Schmertmann (1975) for a thorough discussion of the vane shear test.

A2.3 DYNAMIC CONE PENETRATION TESTS

Dynamic cone penetration tests are used to obtain continuous penetration resistance profiles through the overburden. The test is similar to the SPT, and provides a profile which indicates variations in subsurface

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[1] N is the number of blows of a 63.5 kg hammer, free-falling 76.2 cm, required to drive a 51 mm split spoon sampler 30 cm into the soil. The sampler is typically driven a total of 45 cm, and the N value is derived from the last 30 cm of penetration.

TABLE A2.1 CONSISTENCY AND RELATIVE DENSITY TERMS FOR COHESIVE AND GRANULAR SOILS
(After Peck, Hanson and Thornburn 1974)

A. COHESIVE SOIL

UNDRAINED SHEAR STRENGTH (kPa)	NUMBER OF BLOWS/30 cm in SPT, N (rather unreliable)	CONSISTENCY
< 12	< 2	very soft
12 - 24	2 - 4	soft
24 - 48	4 - 8	firm or medium
48 - 96	8 - 15	stiff
96 - 192	15 - 30	very stiff
> 192	> 30	hard

B. GRANULAR SOILS (less reliable when gravel sizes present)

NUMBER OF BLOWS/30 cm in SPT, N (fairly reliable)	RELATIVE DENSITY
< 4	very loose
4 - 10	loose
10 - 30	compact or medium
30 - 50	dense
> 50	very dense

conditions. Samples are not retained, however, making the dynamic cone test less valuable as a primary investigation technique. In general, a few additional boreholes would probably be of more value than a number of dynamic cone soundings for lagoon investigations.

A2.4 STATIC CONE PENETRATION TEST (CPT)

The static or quasi-static CPT involves pushing a standard cone tip at a constant velocity and recording soil bearing and side friction resistance. The resistance values and ratio may be used to empirically identify soil type, layering, relative density and friction angle of granular soils, and undrained shear strength and modulus of cohesive soils. The CPT is a rapid method of providing relatively complete information between boreholes, provided that there are sufficient borings to allow confident interpretation of CPT readings. More complex CPT tools may be equipped with electronic piezometers, which indicate pore water pressure response to driving and thus may provide information on silt layering in sands, and empirical estimates of consolidation parameters. The CPT is limited by available driving force, and may not penetrate hard or dense sediments.

A2.5 DILATOMETER

The dilatometer is a relatively new sounding device that measures the soil resistance to small lateral deformations. These *in situ* measurements have been empirically correlated with soil type, density, and friction angle or shear strength. Because the dilatometer has a limited empirical data base, caution is advised in interpretation, particularly in soft cohesive soils. Nevertheless, on large sites where the dilatometer is correlated with borehole data the device may be used as a convenient profiling tool to extend borehole information. In general, the applicability of the dilatometer to lagoon investigation may be limited when compared to CPT testing, but is less expensive.

A2.6 BOREHOLE INFILTRATION (HYDRAULIC CONDUCTIVITY) TESTS

<u>In situ</u> features such as layering, fissures, sand lenses, etc., can dominate the hydraulic properties of a deposit. The effect of these structures may not be apparent in samples or laboratory permeability tests, and therefore <u>in situ</u> permeability tests may be desirable, particularly when the <u>in situ</u> sediments are expected to contribute to seepage control. A detailed discussion of these tests and their applicability is presented in Appendix 3.

APPENDIX 3
FIELD AND LABORATORY MEASUREMENT OF HYDRAULIC CONDUCTIVITY

A3.1 INTRODUCTION

The purpose of the Appendix is to present information on various field and laboratory methods of estimating hydraulic conductivity. The most common procedures for measuring hydraulic conductivity are presented and the problems associated with each method are discussed.

The evaluation of hydraulic conductivity in fine grained soils is discussed in Section A3.4. The applicability of measurement techniques in coarser grained materials is discussed in Section A3.5. Finally, a comparison of laboratory and in situ field test results is made in Section A3.6. and conclusions are drawn regarding the interpretation of data for design of wastewater stabilization pond liners.

The information contained in this Appendix is not intended to be a step-by-step guide for conducting hydraulic conductivity tests. Rather, the information is a summary of pertinent literature and experience and is intended to serve as a reference source. This information will aid in selection of the appropriate test method and provide background data for more detailed studies.

A3.2 FACTORS AFFECTING HYDRAULIC CONDUCTIVITY

Hydraulic conductivity, K, is a coefficient of proportionality that describes the relationship between hydraulic gradient, i, and the Darcy flux, v, for a porous material, where

(A3.1) $v = Ki$

According to Darcy's law, K is a constant for laminar flow, for a given fluid and porous medium.

Multiplying v by the cross-sectional area of flow, A, gives the seepage quantity, Q. Thus:

(A3.2) $Q = KiA$

Theoretically, hydraulic conductivity depends on the properties of both the fluid and the porous medium. By far the dominant factor controlling K is the size and continuity of pore spaces in the soil. This factor alone makes K vary over many orders of magnitude (e.g. from 1×10^{-10} m/s or less for clayey materials to perhaps 1 m/s for clean gravel). In considering this tremendous variation in a single engineering parameter, it is often acceptable to establish "order-of-magnitude" estimates for preliminary studies.

On a practical basis, hydraulic conductivity estimates can be affected by a variety of factors in both field and laboratory studies. Modelling of <u>in situ</u> conditions is difficult due to complex geological and structural conditions, while laboratory tests are burdened with errors resulting from sample disturbance, saturation conditions, dispersion of fines, temperature and chemical action of permeant, etc.

In the following Sections, soils are grouped into two broad classes according to grain size. Fine-grained soils include clay and silt size particles, and coarse-grained soils include sand and gravel size particles. This is a convenient division for discussion of hydraulic conductivity measurements because of the similarity of test procedures within each group. It should be noted that such a simple material classifiction system rarely is applicable to field situations. When mixtures of fine and coarse grained materials are encountered, the relative proportion of each particle size will determine whether the hydraulic properties of the mass are controlled by the fine or coarse fraction.

A3.3 SITE GEOLOGY AND HYDROGEOLOGY

Hydraulic conductivity estimates obtained for design of wastewater stabilization ponds are used to represent fluid flow at a particular site in an analytical model. The applicability of the model and calculated seepage rates depend largely on an understanding of the geological and hydrogeological conditions at the site. The hydraulic properties of a carefully placed compacted clay liner may be determined within a narrow range; however, seepage entering the natural geological environment will usually be controlled by more complex and changing hydrogeological regimes. The geological and hydrogeological conditions should be established during the site investigation stage. Only then can one rationally decide which strata to sample or test _in situ_, and what boundary conditions (i.e. seepage pressures and flow directions) to model.

A3.4 HYDRAULIC CONDUCTIVITY MEASUREMENTS IN FINE-GRAINED SOILS

A3.4.1. Laboratory Versus Field Measurements

Silt and clay deposits, or materials containing significant quantities (say more than 20%) of fine-grained soils, tend to be nonhomogeneous because of deposition or modification processes. For example, root holes may exist in the near surface zone; bedding features, sand partings, fissuring, and the like can occur throughout the strata. For these reasons, it is often desirable to conduct hydraulic conductivity measurements _in situ_. Laboratory-scale tests have the inherent problem of limited sample size and disturbance.

On the other hand, prior to construction, compacted soils can only be tested in the laboratory.

Laboratory tests offer the advantage that a large number of tests can be performed at a relatively lower cost than field tests. Additionally, laboratory tests can be performed under controlled conditions, while boundary conditions are difficult to determine in field tests.

A3.4.2. Laboratory Tests

Laboratory hydraulic conductivity tests on fine-grained soils are typically carried out in consolidation cells (modified for falling head tests), compaction molds, standard triaxial cells, or specially constructed cells (e.g. lucite cylinders with appropriate plumbing).

The most common test method is a falling head test in which head loss in a small diameter tube is measured with time (e.g. U.S. Bureau of Reclamation Designation E-15). Constant head tests require a considerable volume of flow to obtain accurate hydraulic conductivity estimates (e.g. ASTM D2434); this test procedure is generally reserved for rather pervious soils as discussed in Section A3.5, or long term tests.

Other less common test methods which are used to measure hydraulic conductivity include the one-dimensional consolidation test (K is calculated indirectly from pore presssure dissipation observations), and radial flow tests (e.g. Bishop and Gibson 1963).

The sources of error in laboratory hydraulic conductivity tests are summarized in Table A3.1. Also shown on this Table are recommended procedures to minimize the effect of these errors and the influence of the error on hydraulic conductivity measurements.

The laboratory testing methods and procedures discussed above deal only with saturated soil conditions. For unsaturated soils, there are additional complexities in hydraulic conductivity measurements as discussed by Olson and Daniel (1981). It is not common to test partially saturated soils in the laboratory. Thus, test results on saturated samples are often used as upper bound hydraulic conductivity estimates. In general, however, most flow regimes in fine-grained soils will become saturated with time because of mounding (see Section 4.4.3).

A3.4.3. Field Tests

There are several in situ hydraulic conductivity tests described in the literature. The most common and useful methods are summarized in Table A3.2. These tests can be subject to a variety of errors and problems as discussed by Olsen and Daniel (1981) and others. Hvorslev (1948) and the U.S. Bureau of Reclamation (1974) provide detailed information on in situ hydraulic conductivity testing.

One of the difficulties in performing field hydraulic conductivity tests on fine-grained soils is that a considerable time may be required to complete the tests. The use of sealed porous probes offer an advantage for these tests in that drilling rigs are not placed on standby during field measurements.

Surface infiltration tests provide a qualitative estimate of seepage loss during the early stages of impoundment. Ultimately, however, saturation will be attained and tests as described above would be more applicable. A review of infiltration test procedures is provided by Reid Crowther et al (1980).

A3.5 HYDRAULIC CONDUCTIVITY MEASUREMENTS IN COARSE-GRAINED SOILS

Laboratory measurements of hydraulic conductivity in coarse-grained soils are generally restricted to compacted materials used for drains or filters. In situ estimates of hydraulic conductivity are best evaluated by field tests because of the difficulties in obtaining an undisturbed sample of granular sediments. The most common laboratory test method on coarse-grained soils is the constant head test (e.g. ASTM 2434). Many of the sources of error listed in Table A3.1 apply to laboratory tests on coarse-grained soils.

Field measurements of hydraulic conductivity in coarse-grained soils include methods and procedures listed in Table A3.2 as well as full-scale pumping tests. Radial flow pump test procedures and methods of analysis are described by Johnson (1975).

A3.6 INTERPRETATION OF HYDRAULIC CONDUCTIVITY MEASUREMENTS

There can be major differences between laboratory and field hydraulic conductivity measurements. These differences are likely to occur because of complex geological and hydrogeological conditions *in situ* and errors in measurement methods. Olson and Daniel (1981) evaluated hydraulic conductivity measurements at several sites where field and laboratory tests were conducted. They found that for the majority of the comparisons, the ratio of K (*in situ*) to K (laboratory) was in the range of 0.38 to 64. The major reasons for higher field values are: (1) laboratory tests are generally run on homogeneous, clayey samples; (2) sand seams, fissures and other macrostructures in the field are not present in laboratory samples; (3) measurement of vertical K in the laboratory and horizontal K in the field; (4) changes in soil structure, chemical characteristics of the permeant, air entrapment in laboratory samples, and other errors associated with laboratory tests.

TABLE A3.1

HYDRAULIC CONDUCTIVITY

Sources of Error in Tests Performed in the Laboratory*

Fine-grained soils

Source of Error	Cause or Effect of Error	Method of Minimizing Effect of Error	Influence on Hydraulic Conductivity, K, Measurement
1. Nonrepresentative samples	Variability of deposit, tendancy to select homogeneous zones	Thorough field investigation, development of accurate geological model	$\pm$ one order of magnitude or more
2. Sample preparation	Voids, stress relief, fissures, smear zones, variation in molding water content.	Careful trimming, apply stress to close fissures, use as large a specimen as possible, control moisture-density of compacted soils	$\pm$ one order of magnitude or more
3. Sample placement in cell	Dispersion of fines, irregular cell wall contact leading to piping during test.	Careful placement and tamping in cell, removal of pebbles in trimmed sample, use cell constructed of transparent material (e.g. lucite), apply sufficient cell pressure in triaxial tests.	$\pm$ one order of magnitude or more.

TABLE A3.1 (continued)

Source of Error	Cause or Effect of Error	Method of Minimizing Effect of Error	Influence on Hydraulic Conductivity, K, Measurement
4. Flow Direction	Sample generally oriented vertically whereas horizontal flow may dominate in site.	Orient laboratory specimen to model dominant flow direction in site.	K_H/K_V may range from 1 to 10 or higher.
5. Alterations in Clay Chemistry (permeant)	Leaching sample with distilled water, permeant with different chemical characteristics than in field.	Where possible, use permeant with same chemistry as field, recycle fluids, use tap water rather than distilled water.	$\pm$ two orders of magnitude in extreme cases.
6. Growth of Micro-organisms.	Clogging of flow channels by organic matter which grows in sample during prolonged tests.	Use disinfectant in permeant if microorganisms would not grow naturally in field situation studied.	$\pm$ 8 to 50 times K for prolonged tests. (measured values generally too low)

TABLE A3.1 (continued)

Source of Error	Cause or Effect of Error	Method of Minimizing Effect of Error	Influence on Hydraulic Conductivity, K, Measurement
7. Air in sample	Attempted saturation of compacted sample.	Apply backpressure to dissolve air into water, conduct test under a reasonably large pressure gradient (see Source of Error No. 9).	± 2 to 5 times K (measured values generally too low)
8. Equipment	Meniscus problems in capillary tubes, leaks, evaporation of permeant, volume changes of cell.	Measure pressure drops instead of heads in capillary tubes, tests should be performed in well constructed laboratory equipment, a dummy cell should be set up to measure evaporation and volume changes due to temperature fluctuations.	0.3 to 3 times K depending on equipment problems.
9. Excessive hydraulic gradients.	Used to reduce testing time, increase seepage quantity. May consolidate sample, cause piping.	Use gradients as close as possible to field conditions, use falling head test.	± 1 to 5 times K; may reduce k due to particle migration; limited influence for carefully run tests.

TABLE A3.1 (continued)

Source of Error	Cause or Effect of Error	Method of Minimizing Effect of Error	Influence on Hydraulic Conductivity, K, Measurement
10. Volume change of specimen due to stress change.	Large gradients increase effective stresses in lower half of sample and may cause consolidation. Upper half experiences reduced effective stress (unless loaded) and may swell.	Measure inflow and outflow, plot flow rate, q, versus time $t^{-1/2}$ and continue test until linear relationship is observed (steady state seepage condition).	Small, provided accurate measurements of sample swelling/compression are taken. (generally 1 to 20 times too high)
11. Temperature	Viscosity changes of permeant.	Perform tests at relevant temperature to field situation modelled.	Viscosity decreases 3 percent per ^{o}C rise; minor influence on K. (generally $\pm$ 0.5 to 1.5 times K)

* References: Olsen and Daniel (1981), Gordon and Forrest (1981)

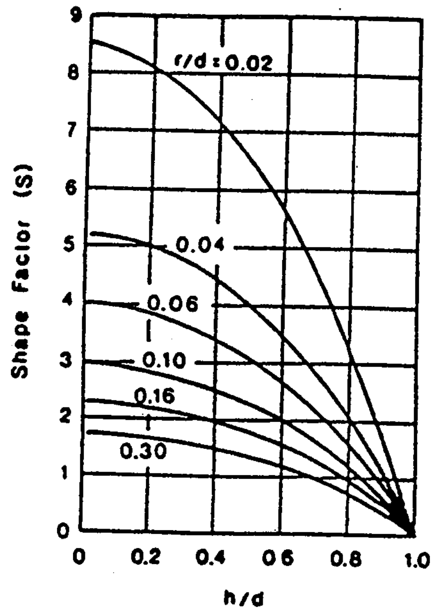
TABLE A3.2 HYDRAULIC CONDUCTIVITY - FIELD TESTING PROCEDURES

Test Method	Procedure	Limitations	Calculation Procedure	REFERENCES
1a. Open Auger Hole - below water table	pump water from hole, measure rise with time	- applies to incompressible soil, hole drilled to impervious base, no drawdown of water table - applicable only to moderately pervious soils - sloughing of soil	$k = \frac{a^2}{16}\ \frac{r}{Sd}\ \frac{h}{t}$ Where, k = conductivity r = radius of well S = shape factor* d = depth of hole below water table h = height of water in hole t = time elapsed since cessation of pumping	1
1b. Open hole above water table	pump water into hole, measure infiltration	- caving of hole	$k = \frac{0.5\ \ln\left(\frac{h}{r}\right)\ \frac{Q}{2\pi}}{h^2\left[\left(\frac{h}{T_u}\right)^{-1} - 1/2\ \left(\frac{h}{T_u}\right)^{-2}\right]}$ Where, h = height of water in well (ft) r = radius of well (ft) Q = discharge rate water from well (ft^3/min) T_u = distance above water table K = conductivity (cm/sec)	2
2. Cased Boring	constant head or falling/rising head water in boring	- few, most common field test in geotechnical engineering	constant head test: $k = q/_{FDh}$ Where, q = flow rate F = shape factor* D = diameter of hole h = head loss (head water above water table) falling or rising head test: $k = \frac{A}{FDt}\ \ln\ \frac{h_1}{h_2}$ Where, A = area of standpipe t = time for head change from h_1 to h_2	1
3. Using porous probes or piezometer	sealed well point or piezometer	- general limitations of sealing porous tips in borings	as above	2

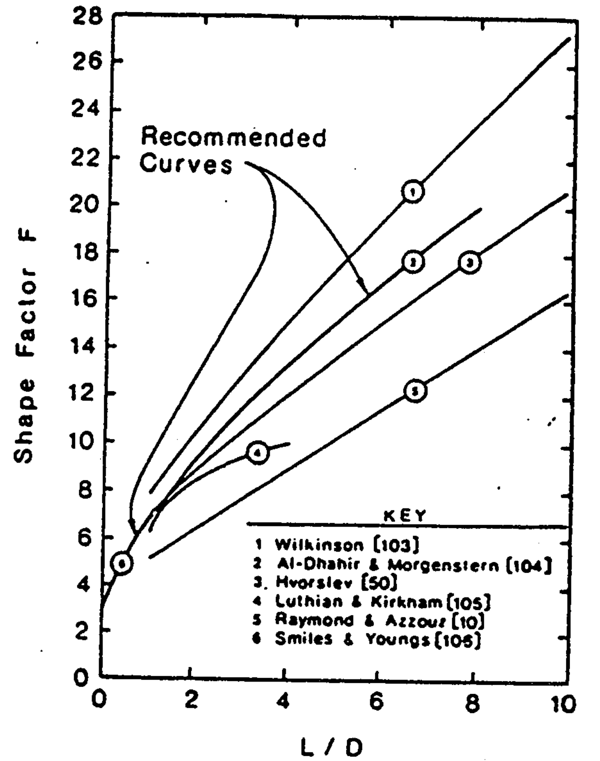
* See next page for shape factor.

1. Olson and Daniel (1981) and included references
2. U.S. Bureau of Reclamation (1974)

TABLE A3.2 HYDRAULIC CONDUCTIVITY - FIELD TESTING PROCEDURES (CONT'D)
(FROM OLSON AND DANIEL, 1981)



Shape factors for use with the auger method (see P.1 of this table for definition of parameters).



Shape factors for use with porous probes (see P.1 of this table for definition of parameters).
(L is length of porous probes)

Case	F*	Condition	Source
1	2π	spherical tip in an infinite soil	Samsioe [93] Dachler [94]
2	π	hemispherical tip extending below an impervious upper boundary	Samsioe [93] Dachler [94]
3	2	borehole with a flat bottom at an upper impervious boundary	Forchheimer [95] Dachler [94]
4	2.75	cased borehole with a flat bottom in the middle of a deep soil layer	Hvorslev [50] Harza [96] Taylor [7]
5	$\dfrac{2\pi(L/D)}{\ln\left(2L/D + \sqrt{1 + \left(\frac{2L}{D}\right)^2}\right)}$	borehole with a flat bottom extending a distance, L, below an impervious upper boundary, no casing	Dachler [94]
6	$\dfrac{2\pi(L/D)}{\ln\left(L/D + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right)}$	cased hole in a semi-infinite soil with an uncased section of length, L, below the casing	Dachler [94]

*Shape factor.

Shape factor for use with porous probes (see P.1 of this table for definition of parameters).

APPENDIX 4
DESIGN & CONSTRUCTION OF COMPACTED CLAY LINERS

This Appendix provides detailed information on the properties of compacted clay liners (Section A4.1), design and construction procedures (Section A4.2) and potential failure mechanisms that should be considered in design (Section A4.3).

A4.1 PROPERTIES OF COMPACTED CLAY AFFECTING LINER PERFORMANCE

A4.1.1 Mineralogy

The hydraulic conductivity of a compacted clay is primarily a function of the pore size distribution and the tortuosity of the resultant seepage channels. The smaller the clay-size[1] particles are, the smaller pore sizes will be, all other factors being equal. As a result, montmorillonite generally has a lower hydraulic conductivity than illite (hydrous mica), which in turn is less permeable than kaolinite (Fig. A4.1). The clay mineralogy may be determined by X-ray diffraction and to a certain extent by the clay activity. Activity is defined as the plasticity index divided by the percent by dry weight of the soil that is finer than 0.002 mm. Approximate values of activity for different clay minerals are listed in Table A4.1.

TABLE A4.1 ACTIVITIES OF CLAY MINERALS (from Mitchell, 1976)

Mineral	Activity
Smectites (montmorillonite)	1 - 7
Illites (hydrous mica)	0.5 - 1
Kaolinite	0.5

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1 Clay-sized particles are arbitarily defined as being less than 0.002 mm in effective diameter, and include most clay minerals and some non-clay minerals.

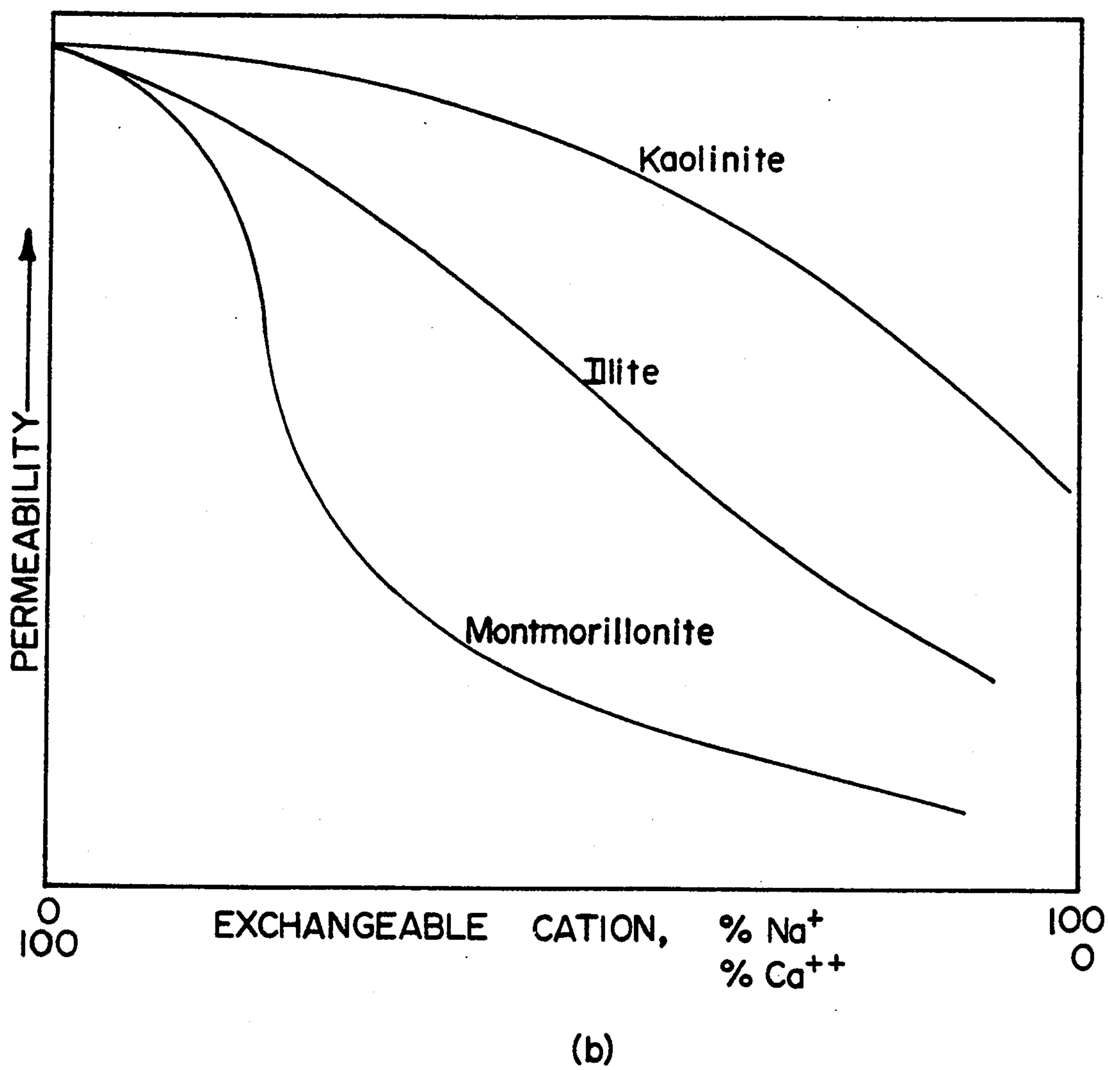

FIG. A4.1 HYDRAULIC CONDUCTIVITY OF CLAY MINERALS
(FROM YONG AND WARKENTIN 1975)

Clay soils with high activities will usually have relatively low K values and high swell potential, and thus may be good liner material; however, the workability of high plasticity clays is generally poor, particularly in wet weather.

Clay tills in Alberta generally have a high percentage of montmorillonite (Fig. A4.2), although activity values are generally less than 1 due to high concentrations of cations in the pore water (see Section A4.1.2). In most cases, simple determination of grain size distribution, Atterberg limits (plasticity index), and activity should be sufficient to characterize the soil and to establish the consistency of soil properties across the site. The real potential of the soil as a compacted clay liner material will depend on the results of laboratory hydraulic conductivity tests.

A4.1.2 Exchangeable Cations

Clay minerals are composed of layered sheets with negative net charges on the sheet faces. Cations such as Na, Ca, Mg, and K, are attracted by the negative charges and occupy the interlayer spaces with varying degrees of permanence, depending on clay mineral and cation type. Detailed discussions of the clay - water - electrolyte system are given by Grim (1968) and Mitchell (1976).

The concentration and valence of cations in the adsorbed water region of the clay minerals affect the clay mineral structure by either increasing or decreasing net electrical repulsive force between sheet faces. Both increased cation concentrations and increased cation valence cause a decrease in net repulsive forces and thus a tendency for flocculation of clay particles. Conversely, either decreased cation concentrations or lower cation valences cause an increase in net repulsive forces and a tendency for the clay particles to disperse. Flocculated clays tend to have a more open, card-house structure and thus a higher hydraulic

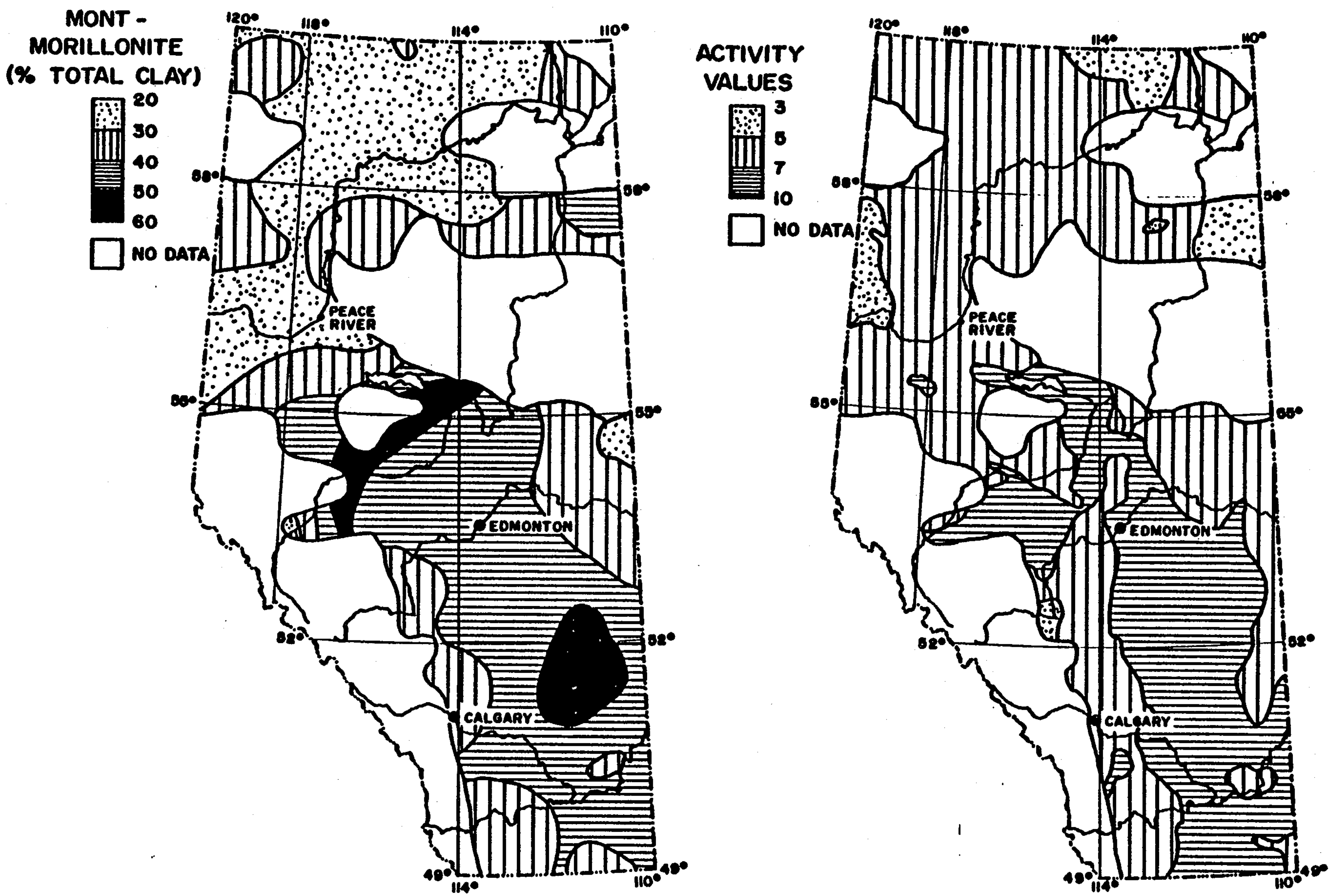

FIG. A4.2 DISTRIBUTION OF MONTMORILLONITE AND ACTIVITY VALUES
ALBERTA TILLS (AFTER PAWLUK AND BAYROCK 1969)

conductivity than dispersed clays. The effects of replacing monovalent Na cations with divalent Ca cations in montmorillonite, illite, and kaolinite are shown on Fig. A4.1. Increasing the Ca^{++} concentration results in more flocculated structures and higher hydraulic conductivities. Na-saturated[1] clays are dispersed and have lower hydraulic conductivities. Montmorillonite has a greater surface area available for cation exchange (termed cation exchange capacity) and thus is more affected by changes in cation species than either illite or kaolinite.

Cation concentrations in Alberta tills are relatively high (Pawluk and Bayrock 1969), resulting in flocculated structures. The state of flocculation in clay minerals has been experimentally related to the sodium absorption ratio (SAR) of the pore fluid and the total cation concentration of the pore fluid (Aitchison and Wood 1965). Relationships for montmorillonite and illite are shown on Fig. A4.3, together with values for Alberta tills as reported by Pawluk and Bayrock (1969). Points to the right of the appropriate clay mineral envelope indicate flocculated structures while points to the left of the curve indicate dispersed structures. The data shown on Fig. A4.3 suggest that most Albertan tills are flocculated.

A4.1.3 Compacted Clay Structure

Research over the past two decades (e.g. Michell *et al* 1965, Barden 1974, Garcia-Bengochea *et al* 1979) has indicated that the hydraulic conductivity of compacted clays is strongly influenced by the clay structure that develops during compaction. It appears that compacted clay structure is made up of clumps or aggregates of clay particles (sometimes called peds). The pore spaces between individual particles

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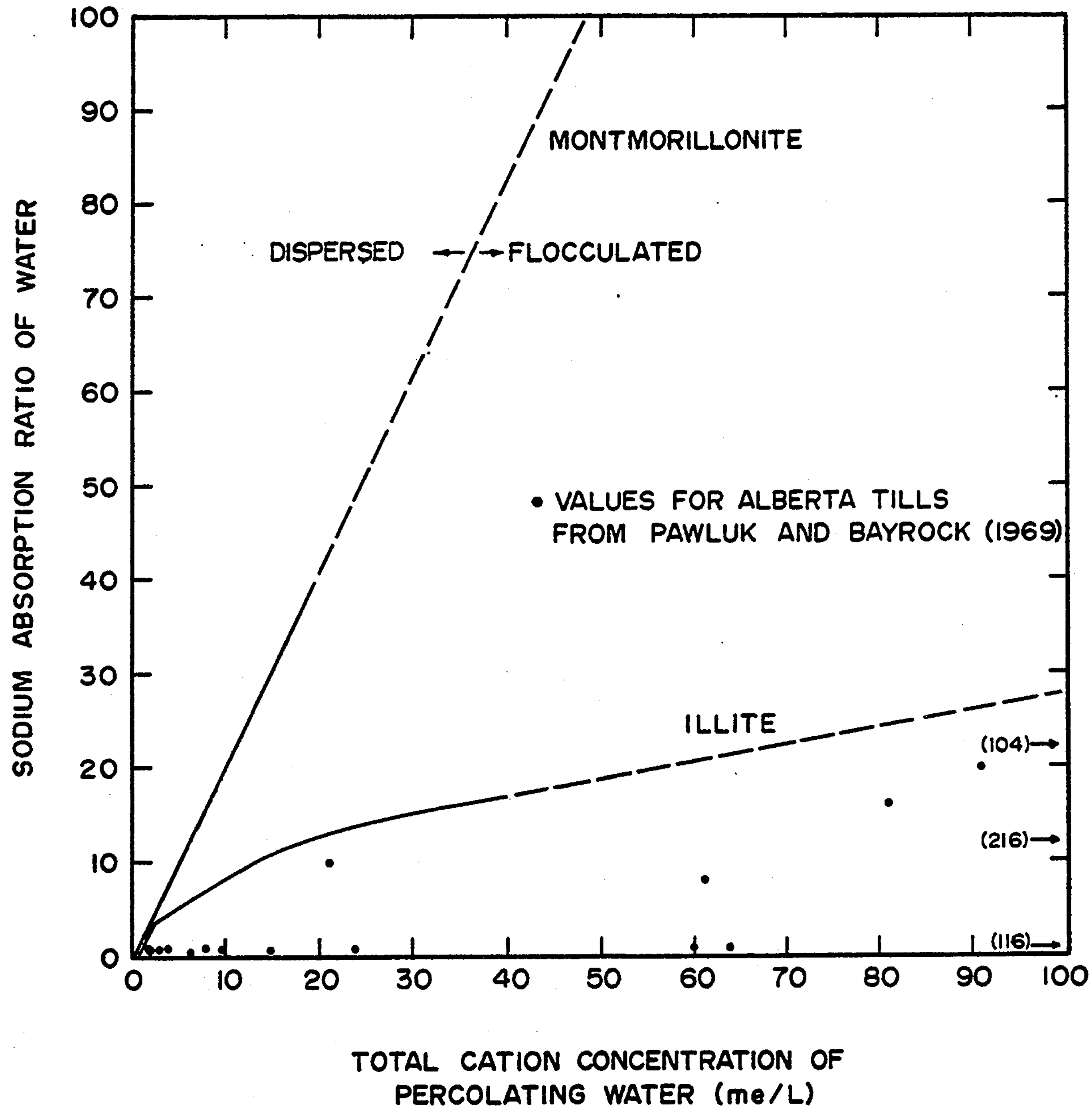
1 The term Na-saturated indicated that all of the available negative surface charges are occupied by Na cations, but it does not necessarily indicate Na saturation in the pore fluid itself.



(CURVES FOR MONTMORILLONITE AND ILLITE
FROM AITCHISON AND WOOD 1965)

FIG. A4.3 FLOCCULATION OF MONTMORILLONITE AND ILLITE AS A FUNCTION OF SAR within the aggregates are very small (micro-pores), while pore spaces between the aggregates are much larger (macro-pores). Thus, for a given total porosity, the hydraulic conductivity increases with a rise in the relative proportion of macro-pores. This is one of the reasons why hydraulic conductivity in clays does not correlate well with total porosity or void ratio (Garcia-Bengochea *et al* 1979).

Two factors appear to dominate the compacted clay structure; one is the moulding moisture content and the other is the mode of compaction. Clays compacted dry of the optimum moisture content have low degrees of saturation and relatively high negative pore pressures, which increase inter-particle effective stresses and thus increase shear strength. The aggregates or lumps resist breakdown during compaction, which results in a relatively high proportion of macro-pores. Clays compacted wet of optimum moisture content are nearly saturated and have lower shear strengths. Thus the aggregates break down more readily during compaction. As a result, the macro-pores become compressed, and hydraulic conductivity is lower than for dry of optimum moisture content conditions. Hydraulic conductivity tests by Mitchell *et al* (1965) on a low plasticity silty clay indicated that hydraulic conductivities of clays compacted on the dry side of optimum moisture content could be as much as 2-3 orders of magnitude higher than the hydraulic conductivity of clays compacted wet of optimum, as shown on Fig. A4.4.

Matrecon (1980) suggest that two general relationships may exist between moulding moisture content and hydraulic conductivity in compacted clays. These are illustrated on Fig. A4.5. The second case is the same as reported by Mitchell *et al* (1965) and shown in Fig. A4.4. The relationship between hydraulic conductivity and bulk density of the soil is also shown. Note that the relationship varies depending on whether the moisture content is wet or dry of optimum. The first case illustrates a different behaviour where hydraulic conductivity is controlled by soil density and has a minimum value at the peak density.

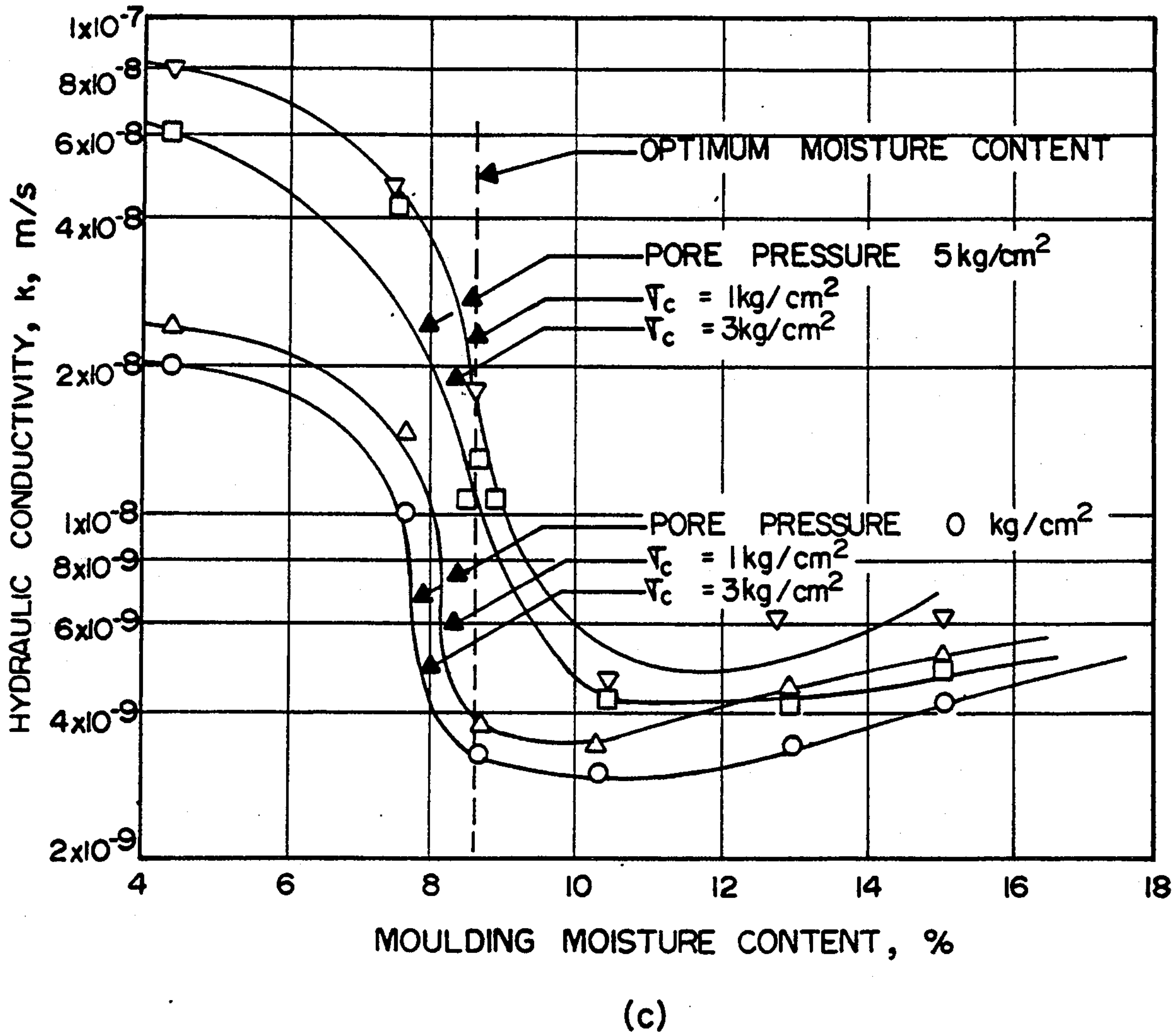


FIG. A4.4 EFFECT OF MOULDING MOISTURE CONTENT ON HYDRAULIC CONDUCTIVITY OF A COMPACTED CLAY (FROM MITCHELL ET AL 1965)

It is recommended that a series of carefully conducted hydraulic conductivity tests (see Appendix 3) be carried out on samples compacted at various moisture contents in order to establish the hydraulic conductivity - moisture content and hydraulic conductivity - density relationships as in Fig. A4.5. These relationships will then be useful tools in establishing quality control during construction.

The mode of compaction also influences hydraulic conductivity in compacted clays (Fig. A4.6). Static compaction, or application of a dead weight, is less effective in breaking down aggregates than kneading compaction (e.g., sheepsfoot roller) which works the soil and causes greater shear deformation. It should be noted that the behaviour described above is based on laboratory tests and may not directly relate to field compaction conditions. For example, clay is placed in discrete lumps in the field thus introducing a third order of structure. Moisture content conditions are more likely to be variable and compaction less uniform. Nevertheless, the same general principles will apply and these principles can be used to optimize liner properties in the field.

A4.1.4 Chemical Weathering of Compacted Clay

Liquids seeping through compacted clay soils with constituents different from the native pore water may alter the hydraulic properties of the clay in three ways. Caustic solutions, such as acids and bases, may cause dissolution of certain elements and minerals in the soil structure. Acids dissolve aluminum, iron, alkali metals, and alkaline earths, while bases dissolve silica (Grim 1953). Migrating fines released by the dissolution process may initially plug pore spaces and decrease hydraulic conductivity, but eventually channels may form in the clay and lead to piping and significant increases in hydraulic conductivity (Anderson *et al* 1982).

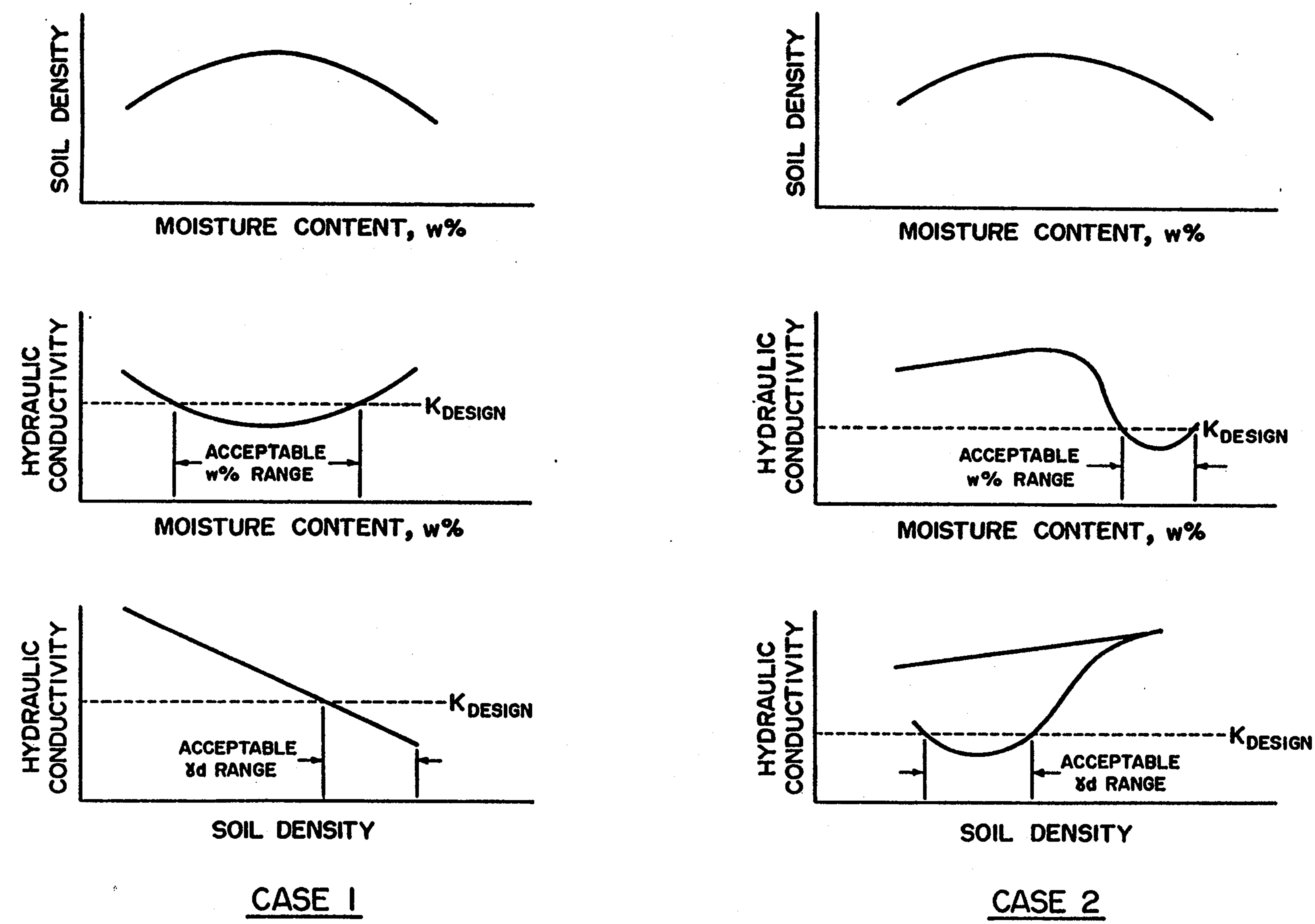
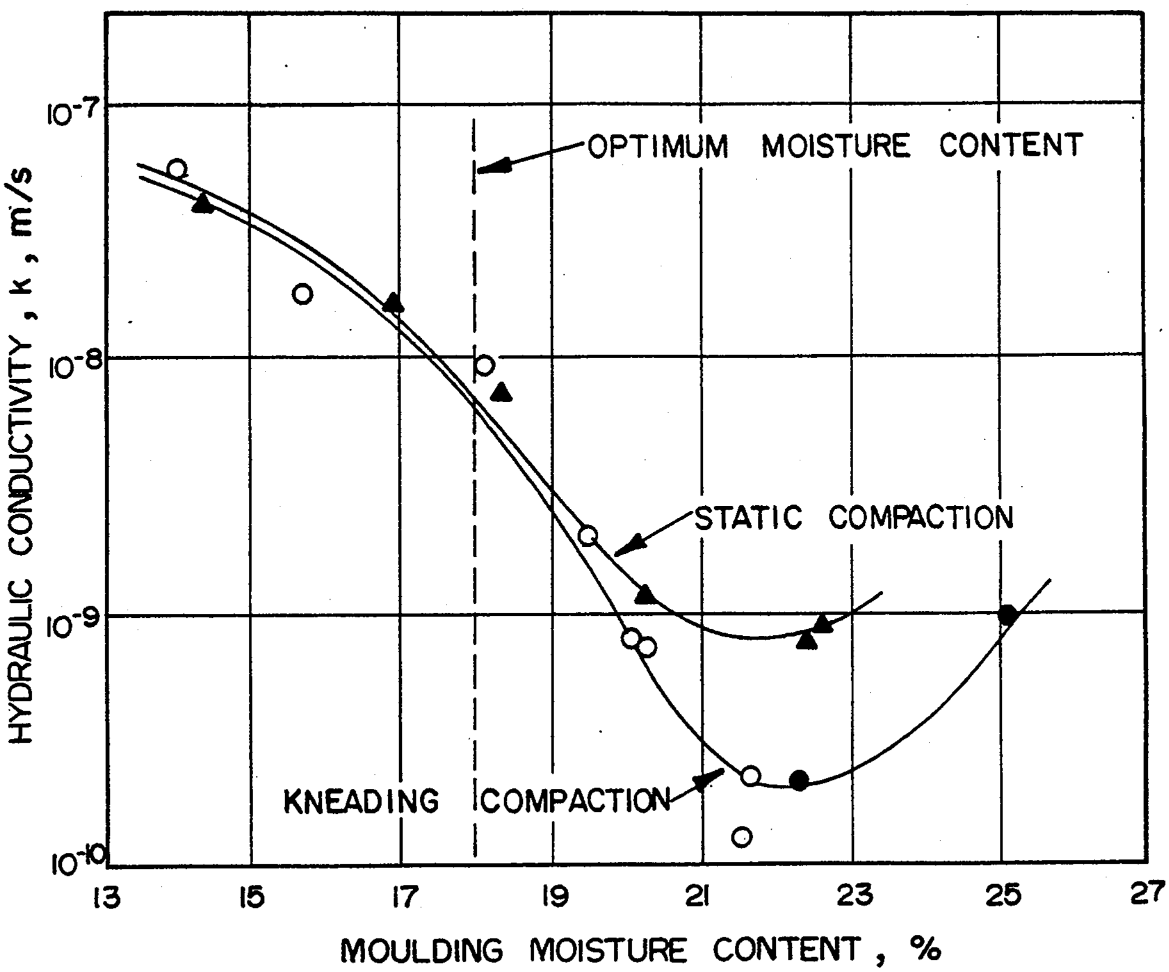


FIG. A4.5 RELATIONSHIPS BETWEEN SOIL DENSITY, MOISTURE CONTENT, AND HYDRAULIC CONDUCTIVITY OF COMPACTED CLAY (FROM MATRECON 1980)



(d)

FIG. A4.6 EFFECT OF COMPACTION METHOD ON HYDRAULIC CONDUCTIVITY OF COMPACTED CLAY (FROM MITCHELL ET AL 1965)

The second form of alteration is change in clay structure due to cation exchange in the adsorbed water region, or in the case of some organic liquids, replacement of the adsorbed water itself (Anderson et al 1982). For example, replacement of Na^+ with Ca^{++} results in a tendency toward flocculation (Fig.A4.1), while leaching of Na^+ by water with a low total cation concentration can cause clays with high initial SAR values to disperse (Fig.A4.3). Large increases in Na^+ or Ca^{++} concentration due to brine results in large reductions in net repulsive forces, causing shrinkage cracks and development of a blocky structure (Folkes 1982 and included references). Anderson et al (1982) reported that aniline, ethylene glycol, acetone, methanol, xylene, and heptane caused shrinkage and cracking in 4 different natural clays (calcareous and non-calcareous smectite, kaolinite, and illite) with resultant increases in hydraulic conductivity of up to 3 orders of magnitude before beginning to stabilize. The changes in hydraulic conductivity could not be accounted for by changes in fluid density and viscosity.

The third form of hydraulic property alteration is due to precipitation of heavy metals, salts, carbonates, etc. This contributes to a decrease in hydraulic conductivity by blocking pore spaces. The precipitation may be reversible, however, with changes in solute concentration, temperature, and pH.

The effects of various industrial waste liquids on compacted clay are included in the Tables in Appendix 14.

Typical municipal wastewater is less likely to cause radical changes in the hydraulic properties of compacted clay. Total dissolved solids concentrations are moderate (Table A13.1), and are unlikely to cause dispersion or blocky structures due to flocculation forces. Organic acid formation may result in slightly acidic pH levels (a pH of 5 or less could cause dissolution of clay minerals) but sewage pH levels of about 6 to 8 should not cause significant problems unless calcareous clays or aggregate are present. The effects on clay liners of chemical additives used for odour, insect, and algae control and the like, should be considered, particularly if additives contain high concentrations of Ca^{++} or other cations.

Whenever there are questions about the effects of a wastewater on a liner material, particularly when concentrations of industrial wastes are present, liner/wastewater compatibility tests should be carried out (Appendix 14).

A4.2 DESIGN AND CONSTRUCTION PROCEDURES

A4.2.1 Delineation of Borrow Deposit

The first step in designing a compacted clay liner is delineating a relatively uniform deposit of suitable borrow material, preferably from the pond cut, or from a nearby borrow area. The required volume of clayey soil is equal to the surface area of the pond interior times the liner thickness (measured perpendicular to the bottom and sideslope surfaces). A large reserve volume is recommended to ensure that there is indeed sufficient clay volume after removing silt and sand pockets and other unsuitable materials.

A4.2.2 Liner Thickness

Recommended minimum compacted clay liner thicknesses are 0.9 m on the pond bottom and 1.2 m on the sideslopes, to allow for weathering, variations in actual thickness, pockets of poor quality material that escape detection, etc. In some cases a minimum thickness of 0.6 m may be acceptable on the pond bottom, provided that the integrity of the liner can be demonstrated. A schematic cross-section of a compacted clay liner is shown on Fig.A4.7.

If a clay core in the dyke is preferred over an upstream clay blanket liner, then the core should be well keyed into the bottom liner. A

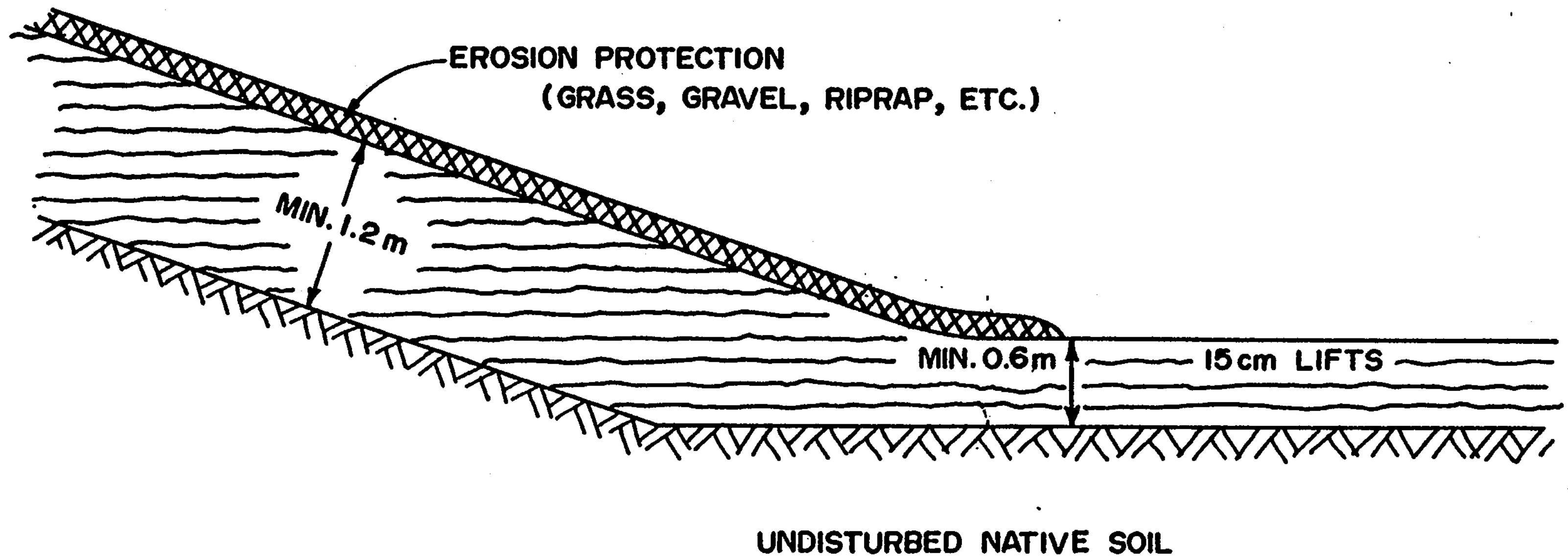


FIG. A4.7 SCHEMATIC CROSS-SECTION OF COMPACTED CLAY LINER minimum core width of 3 m is suggested to allow economic and proper placement and compaction of the clay using large earthmoving equipment. The U.S. Bureau of Reclamation (1974) recommends that the width of a clay core at any elevation not be less than the height of the dyke remaining above that elevation in order to keep hydraulic gradients through the core to less than 1. Filter criteria should be adhered to where applicable.

A4.2.3 Hydraulic Conductivity of Compacted Clay

The *in situ* hydraulic conductivity of the compacted clay liner should be predicted from laboratory tests on the proposed clay borrow material. Several samples should be selected representing the *range* of material within the designated borrow zone, not just the better material. Permeability tests should be performed on the samples compacted to the required density (e.g. 95% of standard Proctor maximum dry density) at a moisture content *anticipated in the field*. It is recommended that the sensitivity of the compacted clay hydraulic conductivity to variations in density and compaction moisture content be determined (see Section A4.1.3). The designer must be prepared to ensure that the soil is brought to the specified moisture content (e.g. by wetting), unless the natural moisture content is already suitable.

A laboratory value for K should be calculated from the weighted average of the individual tests. The weighting of each test value should be according to the estimated percent of the borrow volume that the individual sample represents.

It is recommended that the liner design be based on a K(*in situ*) that is one order of magnitude larger than the average K(lab), i.e.

$$K(\text{design}) = K(\textit{in situ}) = 10 \times \text{average } K(\text{lab})$$

The increase in the K value is a factor of safety to allow for the effects of macro-structure, poor quality borrow, etc., in the field. The K(design) and liner thickness values should meet the seepage criteria outlined in Section 4.1.2. If K(design) is too high, then more selective borrowing or adjustment of compaction moisture content could be investigated. Otherwise, an alternative liner material will be required.

A4.2.4 Subgrade Preparation

The subgrade surface below the compacted clay liner should be relatively level (to control liner thickness) and proof-rolled to provide a good base for compacting the first liner lift to the specified density. Soft pockets that would prevent sufficient compaction of the liner must be sub-excavated and replaced with suitable, compacted fill.
Clay should not be placed directly over gravel or other materials that do not provide an adequate filter to prevent piping erosion of the liner (e.g. see Aisenstein *et al* 1961).

A4.2.5 Liner Material Placement and Compaction

The clay should be placed in uniform, horizontal lifts of about 15 to 20 cm maximum loose thickness. The liner should be constructed in at least 4 lifts (e.g. four 15 cm lifts for a 0.6 m thick liner). Thin lifts ensure more uniform density, better bonding between lifts, and reduce the likelihood of continuous seepage channels existing in the liner. Large lumps, cobbles, and other undesirable materials are more easily identified in thin lifts. Lumps of soil greater than 10 cm in maximum dimension should be broken up prior to compaction. As far as practical, the liner should be built up in a uniform fashion over the pond area, in order to avoid sections of butted fill where seepage paths may develop.

Each lift should be compacted within the specified water content range to the required density using heavy, self-propelled sheepsfoot compactors. Lift surfaces that have been allowed to dry out should be scarified prior to placing of the next lift. Lift surfaces that have degraded due to precipitation, etc., should either be removed or allowed to dry to the required moisture content and then be recompacted.

The completed liner should be smoothed out with a smooth-barrel compactor to reduce the liner surface area exposed to water absorption and swelling. The liner base should not be allowed to dry out or be exposed to freezing temperatures. Ideally, the liner should be flooded as soon as possible after construction and acceptance.

A4.2.6 Construction Control

The most important form of quality control during construction of compacted clay liners will be observation and direction by the engineer. The characteristics of the desired liner material should be established in as much detail as possible (e.g. by color, texture, moisture content, plasticity, or characteristic features such as the mineralogy of pebbles in till). Quick visual or index test identification by experienced field personnel is probably the best way to detect poor quality material. An indirect but simple way of controlling liner quality is to perform frequent _in situ_ density and moisture content tests. The density and moisture content may then be related to hydraulic conductivity by the relationships established during the laboratory test program (see section A4.2.3). The frequency of tests should be increased when soil conditions are variable. The tests may be used to statistically evaluate the overall liner properties and to assess suspect zones in the liner.

In situ density and moisture content tests should be carried out on a routine basis for each lift. Tests should be conducted on a grid pattern (say 30 x 30 m to 60 x 60 m grids for large ponds and at closer spacing for small ponds) and in suspect areas.

The completed liner may be assessed by performing in situ infiltration tests, which may be theoretically related to hydraulic conductivity values (see Appendix 3). It should be noted that the compacted clay liner is most likely to be partially saturated at the end of construction. The presence of 5 to 10% air voids will result in an unsaturated K value that is somewhat lower than the saturated K value.

The completed liner may also be cored, and the hydraulic conductivity of a trimmed sample can be tested in a suitable permeameter, e.g., oedometer falling head tests or triaxial constant head tests (see Appendix 3). All holes created in the liner due to tests, stakes, or other circumstances should be backfilled with well-compacted liner material.

A4.2.7 Planning

The most important aspect of constructing a compacted clay liner may be the planning stage when the inspection engineer's role is defined, contract specifications are prepared, and construction strategies are worked out. The engineer must have an adequate degree of control over material selection and methods of placement. The work procedure must be flexible with respect to earth movement and the contract must have a fair and reasonable basis of payments so that the contractor will be cooperative.

Ideally, the borrow for a compacted clay liner would be the cut material just below the eventual pond invert. Thus, material may be cut and placed in a single operation for much of the pond liner area, although some stockpiling of borrow may be inevitable. The lower lift of the liner might consist of reworked native soil broken up by tilling and recompacted to eliminate fissures, etc. Nevertheless, the contract should allow for selective borrowing of cut material for liner use, for stockpiling, removal of undesirable materials, and possible additional borrowing outside of the cut area.

A4.3 POTENTIAL FAILURE MECHANISMS

A4.3.1 Liner Stresses

Stresses and deformations caused by gravity forces, capillary forces, thermal forces, and ice forces can increase the hydraulic conductivity of compacted clay liners by loss of density or by creation of fissures. Gravity may cause settlement of foundation soils and dykes and creep on sideslopes, resulting in deformation of the liner and tensile cracks. Capillary and thermal forces due to wetting and drying and freeze-thaw cycles may result in loss of density and/or fissuring. The depth of this weathering depends on the depth of drying and/or freezing. Both conditions may be avoided on the pond bottom by maintaining a minimum depth of wastewater at all times and having thicker liners than required by the seepage criteria to allow for some weathering. The sideslope area is more difficult to protect in ponds with fluctuating fluid levels. Field observations have indicated that the hydraulic conductivity of the sideslope area of compacted clay liners can be significantly higher than on the base (Folkes 1982). Thick liners and the use of gravel protection rather than vegetation will improve resistance to drying. In some situations hybrid liners with compacted clay on the base and a weather-resistant material on the side slopes may be desirable.

A4.3.2 Swelling Clays

Swelling is likely to occur when highly plastic, compacted clays are inundated with water. This behaviour was observed in montmorillonite clay liners in Israel (Aisenstein et al 1961) where the swelling of 0.5 and 1.1 m thick liners resulted in increased hydraulic conductivity in the upper half to two-thirds of the liners, and thus an effective decrease in the liner thicknesses (Fig.A4.8). The lower portions of the liners did not swell because of the concentrated seepage forces resulting from hydraulic gradients of 20 and higher. The high gradients eventually caused piping of the liner material into the coarse subgrade

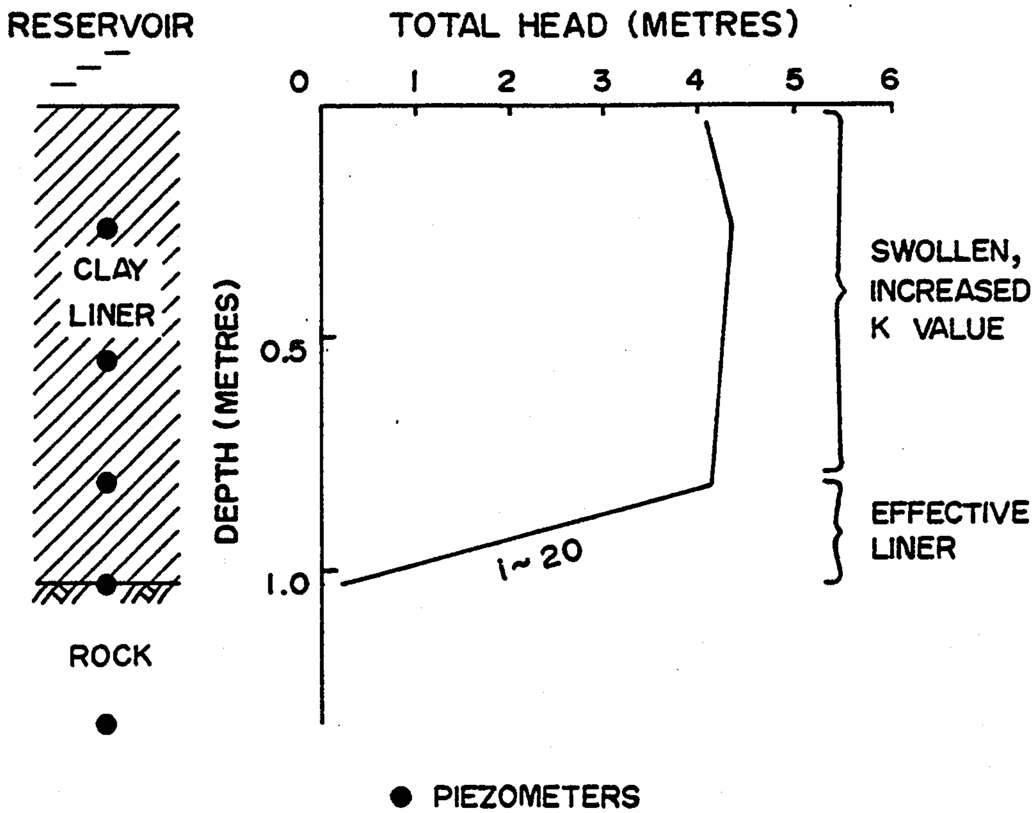


FIG. A4.8 EFFECT OF SWELLING ON SEEPAGE GRADIENTS IN PLASTIC MONTMORILLONITIC LINER (AISENSTEIN ET, AL. 1961) below. Besides recommending proper filter layers between clay and coarse granular materials, the recommended solution to the swelling problem was ballasting of high swelling clays with about 0.3 m of ballast material.

The montmorillonitic clay tills in Alberta generally have a low swelling potential because of the low activity (typically 0.7 to 0.9), although leaching of salts from the pore fluid by liquids low in total dissolved solids could alter this condition . Swelling potential may be estimated from the chart by Seed et al (1962), or the equation:

(A4.1). . . $S = 3.6 \times 10^{-5} (A)^{2.44} (C)^{3.44}$

where S is the percent swell for laterally confined samples under a 7 kPa surcharge, A is the activity, and C is the percent clay content (2 m). Effective stresses due to the effective weight of the liner material and seepage forces will generally average about 7 kPa in the upper half of typical compacted clay liners, under heads of about 1.5 m wastewater.

A4.3.3 Erosion

Erosion of liner material by wave action, etc., reduces the effective thickness of the liner and therefore can increase seepage rates. Provided that adequate scour and erosion protection is provided around inlets on the sideslopes, these erosion mechanisms should not be a serious problem.

Piping erosion of fine-grained soils is a more serious problem that could lead to the failure of the liner (e.g. Aisenstein et al. 1961), or more catastrophically, collapse of a dyke. Piping occurs when preferential seepage along fissures or beside pipes and structures washes away fines, or when high seepage gradients wash fines into coarse granular material. These conditions can be avoided by sufficient compaction of materials beside pipes and structures; by employing seepage cutoffs when necessary; and by ensuring that all adjacent materials along potential seepage paths meet standard filter criteria (e.g. see Cedergren 1976). Indicators of potential piping erosion are visible seeps in the downstream slopes of the dykes and cloudy discharge waters from underdrain and toe drain systemns. Dispersive clays are more susceptible to piping than flocculated clays and extra precaution should be taken in their use.

Other forms of potential erosion include scour by flood waters or storm runoff. The exterior slopes of dykes should be protected with adequate riprap for anticipated flows in areas of potential scour.

A4.3.4 Vegetation and Wildlife

Long-rooted grasses and other vegetation may create root holes in compacted clay liners and cause drying of the soil. Short-rooted grasses should be used for erosion control of side slopes, and should be regularly cut. Weed growth is a particular problem with sewage lagoons and should be controlled. Burrowing animals can also create seepage paths through dykes and should be controlled.

APPENDIX 5
DESIGN AND CONSTRUCTION OF BENTONITE ADMIX LINERS

This Appendix provides detailed information on the properties of bentonite (Section A5.1 and A5.2) design and construction procedures (Section A5.3) and potential failure mechanisms that should be considered in design (Section A5.4).

A5.1 ORIGIN AND COMPOSITION OF BENTONITE

Bentonite may be defined as any clay composed primarily of smectite clay minerals (e.g. montmorillonite) and whose properties are dominated by the smectite clay mineral (Grim and Guven 1978). Many bentonites are derived from devitrification and chemical alteration of volcanic ash or tuff. Bentonites in North America are found in Late Cretaceous and Tertiary sedimentary deposits in beds ranging from less than a centimetre to a few metres in thickness. Deposits are mined commercially in the Black Hills region of Wyoming, Montana, and South Dakota; in Texas and Mississippi; Arizona and Nevada; California; Alaska; and in the Prairie provinces of Canada.

The properties of a bentonite are determined by the smectite mineral, the cleanness of the clay (sand, silt, and other constituents are considered to be contaminants), and the exchangeable cations occupying the interlayer adsorbed water zone of the clay minerals. Most bentonites have calcium as the predominant exchangeable cation. Sodium bentonites are found in the Black Hills region of the United States and in Alberta deposits. A study of Alberta bentonite deposits is reported by Scafe (1975). Deposits outcrop or are at shallow depths in the areas shown on Fig. A5.1. Good quality (clean) sodium bentonite is mined at Rosalind, and some lower quality (silty) bentonite is stockpiled near Onoway. The other deposits in Alberta are generally of poor quality because of high silt and sand contents and higher calcium contents (Scafe 1975).

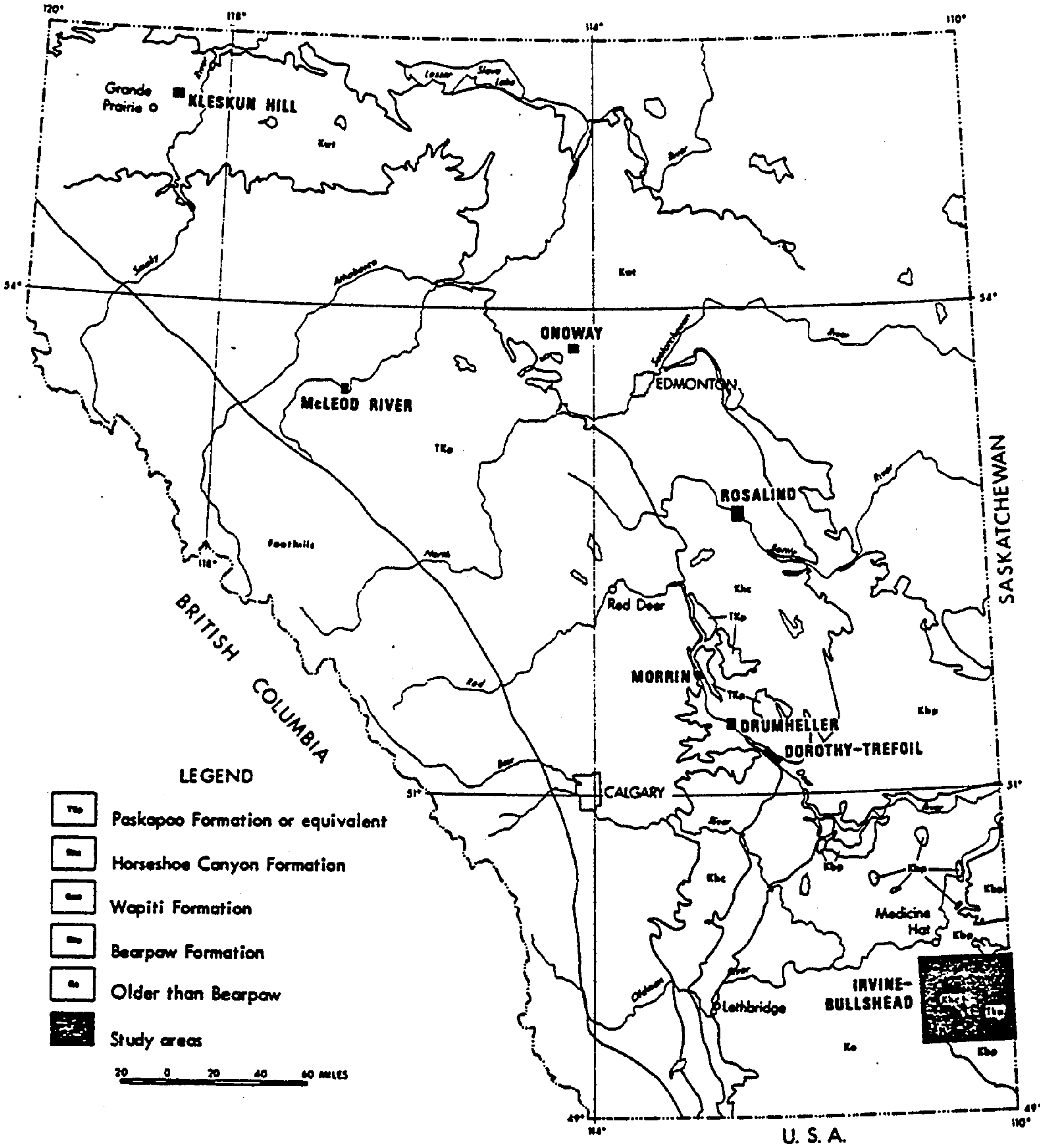


FIG. A5.1 LOCATIONS OF AREAS STUDIED BY SCAFE (FROM SCAFE 1975)

A5.2 PROPERTIES AFFECTING BENTONITE LINER PROPERTIES

A5.2.1 Swell Potential and Hydraulic Conductivity

Bentonite admix liners are formed by mixing dry, granular or pulverized bentonite with sandy or silty soil. The introduction of water causes the bentonite particles to hydrate and swell, filling some or most of the void space in the soil and thus impeding seepage of water through the liner. Important properties of the bentonite are thus swell potential and permeability. These properties are controlled by the smectite content and the exchangeable cation. Wyoming bentonites are typically 90% smectite, while deposits at Rosalind, Alberta range from about 76 to 88% clay (montmorillonite). Sodium bentonites are much more expansive and dispersive than calcium bentonites, resulting in higher swell potentials and lower permeabilities than calcium bentonite (e.g. see effects of percent Na versus Ca on the hydraulic conductivity of montmorillonite in Fig. A4.1). This behaviour is reflected by plasticity values shown in Table A5.1, and hydraulic conductivity values shown in Table A5.2. The test values reported in Table A5.2 from Grim and Guven (1978) originate from an obscure reference published in 1938 and therefore the accuracy of the measurements cannot be assessed. Nevertheless, the lower permeability of sodium bentonite is evident.

A5.2.2. Bentonite - Sand Mix Ratio

The percent bentonite in the bentonite and sand admix (by dry weight) is a critical factor in the performance of the admix liner. Seepage will occur through continuous channels in the sand if there is insufficient bentonite to fill most of the voids after swelling. The bentonite will still reduce the rate of seepage by reducing the overall cross-sectional area of flow, but the quantity of seepage flowing through continuous voids or seepage channels will still be relatively high. For example, if the application of bentonite reduces the area of voids available for flow to 5% of the original area, the hydraulic conductivity of a sand with an original K value of 10^{-4} m/s will only be reduced to 5 x 10^{-6} m/s, which is still quite high for a liner. However, once all continuous voids are plugged by bentonite the seepage rate will be controlled by the hydraulic conductivity of the bentonite, which typically is in the order of 10^{-10} to 10^{-12} m/s. Occluded voids in the bentonite sand mixture will increase the liner hydraulic conductivity somewhat by reducing the overall thickness of bentonite, but this has only a small effect on the liner K value.

The uniformity of the bentonite-sand mix is as important as the quantity of bentonite. The mixing process must distribute the bentonite through the sand so that all parts of the liner have the minimum amount of bentonite required to prevent continuous voids.

The percent bentonite required to form an effective liner will depend on the type of bentonite (i.e., percent montmorillonite and cation type), the gradation of the native sand or silt, and the porosity of the compacted admix. A mix that is adequate in a dense material may not be adequate if the sand has a looser structure.

In general, published information and product literature suggest that sodium bentonite percentages of about 10 percent (by dry weight) are adequate to reduce the K value of the admix to 10^{-9} m/s or less (see Table A5.3); however, the required mix ratio will vary with the soil type. It should be noted that for a given bentonite product, silt admixes will generally have lower K values than sand admixes. Because the tests were carried out using different native soils, the data in Table A5.3 should not be used to compare the different bentonite products.

A5.2.3 Chemical Weathering of Bentonite

The high swelling, dispersive properties of sodium bentonites can be altered by the introduction of contaminated fluids. For example, a fluid with a high total dissolved salts content may cause cation exchange, with calcium and other divalent or trivalent cations preferentially replacing sodium. This results in suppression of the double diffuse layer thickness, lower repulsive forces between clay particle surfaces, and a more flocculated, permeable structure. Acids and bases may cause breakdown of the clay mineral structure leading to channelling of the liner. Organic fluids may also damage untreated smectite clays (Anderson and Brown 1981, Brown and Anderson 1982).

Some producers treat bentonite products with polymers to improve the bond between the bentonite and sand particles and to increase the resistance of the clay to cation exchange and chemical breakdown of clay structure. For example, the American Colloid Company produces various treated bentonite (Volclay) for relatively fresh water, for solutions with high salt contents (total dissolved salts), for municipal solid waste landfill leachate, and for hazardous organic and inorganic chemicals. The compatibility of any contaminated fluid with a bentonite product should be tested by permeability tests using the fluid to be contained, over a relatively long period (preferably several months under constant head). It should be noted that acids may initially cause a decrease in permeability in clays because small particles from broken-down clay minerals block seepage paths. Eventually, however, the acid dissolution of the clay will result in channelling or piping through the liner (Brown and Anderson 1982).

Tables indicating the compatibility of bentonite with various liquids are presented in Appendix 14.

A5.3 DESIGN AND CONSTRUCTION OF BENTONITE ADMIX LINERS

A5.3.1 General

The only type of liner using a bentonite additive recommended herein is the admix design where a processed bentonite powder or granular material is mixed into a layer of soil. In some areas it may be possible to borrow directly from a natural bentonite deposit (for locations, see

Scafe 1975). Construction of liners using native bentonite may be difficult because of the highly plastic nature of the clay.

A schematic cross-section of a bentonite admix lined pond is shown in Fig. A5.2.

The native soil in which the bentonite is mixed should be a silty or sandy material. Cohesive soils will not mix with the bentonite adequately unless pulverized so that a uniform mixture is achieved, and gravels will require large quantities of bentonite. In addition, the bentonite will likely wash out of coarse sand or gravel, thus dispersing the liner "layer".

A5.3.2 Percent Bentonite Required in Mix

The required percentage of bentonite should be determined by performing hydraulic conductivity tests on different mix ratios of the proposed bentonite product(s) and the native soil. These tests may be provided as a customer service by the bentonite supplier; nevertheless, the mix ratio should be confirmed by an independent laboratory. If the supplier and independent laboratory report different required ratios, the higher ratio should be used in design.

It is recommended that all percentages be computed on a dry weight basis to allow for easier comparison.

The hydraulic conductivity tests should be performed in falling head permeameters on samples compacted to about 95 to 100% of the maximum dry density (ASTM D698), or as required to achieve the desired permeability. The samples should be allowed to hydrate for 1 to 2 days prior to application of large hydraulic heads, and the tests should be run for several days or until a linear relationship between time and the logarithm of hydraulic head is well established. Tap water will be more representative of the total dissolved solids concentration in the lagoon than distilled water.

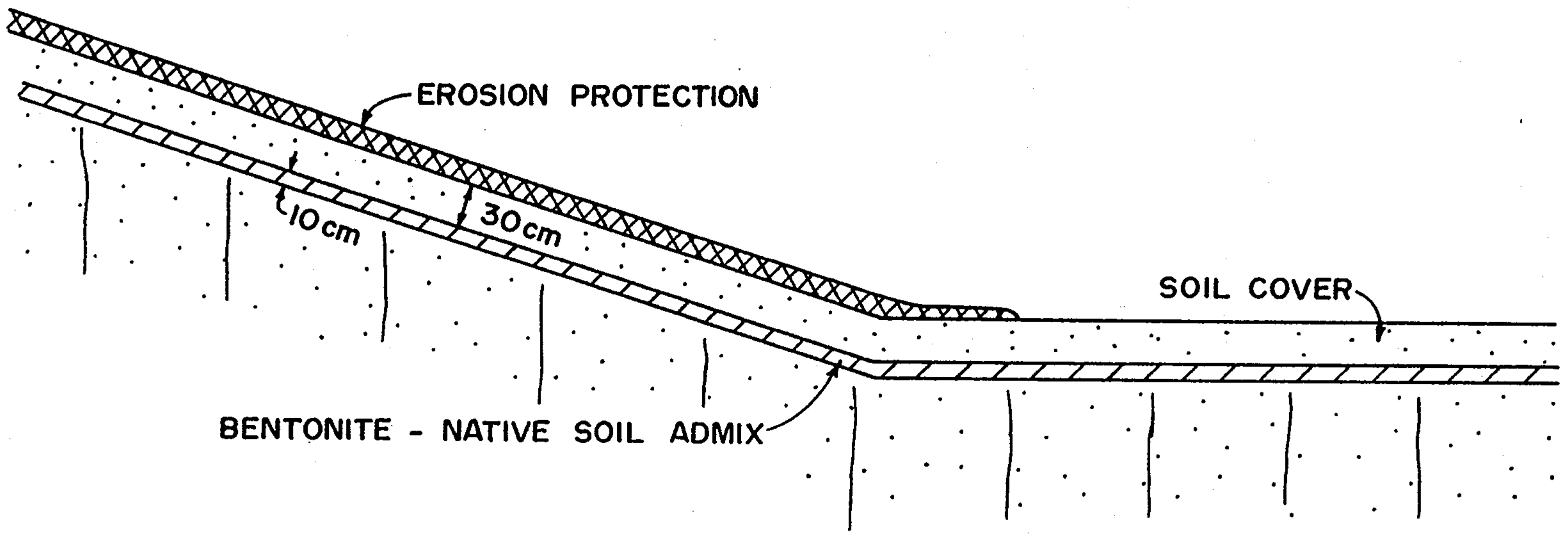


FIG. A5.2 SCHEMATIC CROSS-SECTION OF BENTONITE ADMIX LINER

The hydraulic conductivity of the bentonite and soil admix must meet the seepage control standards outlined in Section 4.1.2. For a 10 cm thick admix liner, a K value of about 2×10^{-10} m/s or less is required.

A5.3.3 Determination of Application Rate

If the admix liner is to be constructed by first spreading the bentonite on the pond bottom and then tilling or discing the bentonite into the native soil, the required percent bentonite in the admix will have to be converted to an application rate (weight of bentonite per unit area).

It is recommended that the percent bentonite and thus application rate required be increased by 25%. This will help allow for incomplete coverage, non-uniform mixing, and loss of bentonite due to wind.

The dry weight application rate may be calculated by the following equation:

(A5.1) Application Rate (Dry Wt.)

$$= 1.25 \times \frac{P}{100 - P} \times \gamma_d \text{ (compacted liner)} \times L$$

where P is the required percent bentonite in the mix, γ_d (compacted liner) is the dry density of the admix after compaction to the desired density, and L is the compacted thickness of the liner. All units should be compatible. The 1.25 factor increases the application rate by 25%, as discussed above.

The application rate on a bulk weight basis is calculated by:

(A5.2) Application Rate (Bulk Wt.)

$$= \text{Application Rate (Dry Wt.)} \times (1 + w)$$

where w is the moisture content of the bentonite at the time of application (typically about 0.09).

EXAMPLE A5.1

The results of hydraulic conductivity tests on different mixes of bentonite and native sand indicate that a mix with 7 percent bentonite (dry weight) is required to produce a K of 2×10^{-10} cm/s or less. The tests were carried out on samples compacted to 95% of the maximum dry density, or a dry density of about 1700 kg/m^3. The design thickness of the liner is 0.10 m. The moisture content of the bentonite powder is 8%.

$$\text{Application Rate (Dry Wt.)} = 1.25 \times \frac{7}{100 - 7} \times 1700\ \text{kg/m}^3 \times 0.1\text{m}$$
$$= 16.0\ \text{kg/m}^2$$

$$\text{Application Rate (Bulk Wt.)} = 16.0 \times (1 + .08)$$
$$= 17.3\ \text{kg/m}^3$$

A5.3.4 <u>Liner Thickness and Depth of Tilling</u>

The compacted bentonite admix liner should be at least 7.5 cm in thickness. A 10 cm thickness is recommended since the thinner the liner is, the more vulnerable it will be to damage and seepage erosion. A typical cross-section of a bentonite admix liner is shown on Fig. A5.2.

The depth of tilling will be a function of the in situ density of the native soil prior to tilling, and the required density after compaction, i.e.

$$\text{(A5.3)} \quad \text{depth of tilling} = \frac{L \times \gamma_d \text{ (compacted liner)}}{\gamma_d \text{ (in situ soil)}}$$

A5.3.5 Subgrade Preparation

The pond invert should be smooth and uniform prior to the application of the bentonite. All vegetation, organic soils, and debris should be removed, and pockets of unsuitable soils for admixing (e.g., gravel or clayey pockets) should be replaced with suitable sandy or silty soil. The liner layer should be free of gravel or lumps greater than about 2 cm in diameter.

A5.3.6 Application and Compaction

Before the application of the bentonite, the soil should be brought to a moisture content between the optimum for compaction of the admix and 4 percent above optimum.

The bentonite should be spread and mixed in two coats, first on the pond slopes (first up, then down) and then on the base of the pond (applying the two coats at right angles). A 1:4 (bentonite:soil) mix should be placed for 10 cm around any structures protruding from the surface of the liner. The applied bentonite should be disced into the soil, taking care to control the depth and uniformity of mixing. Pre-mixing of the sand and bentonite in a portable batch plant may result in more uniform mixtures.

After the spreading and mixing is completed the liner should be compacted to at least 95 percent of the maximum dry density (ASTM D698), or as required to achieve the desired K value for the mixture.

Compaction should be conducted using smooth drum or pneumatic rubber-tire equipment. Sheepsfoot rollers should not be used. Bentonite should be spread, mixed and compacted within the same working day. Uncompacted layers should not be left exposed overnight. The effects of wind damage to the site should be considered and measures taken to minimize such damage.

A5.3.7 Soil Cover

All parts of the liner should be covered with a minimum of 10 cm of soil having a maximum particle size of 3 cm. As much as 30 to 60 cm of soil may be required on exposed side slope portions of the liner to prevent drying of the bentonite in dry climates. A minimum of 10 cm of gravel should be placed over the cover soil around pond inlets, areas of potential current erosion and on sideslopes. Larger riprap over the gravel may be required to give erosion protection against waves on the side slopes. The admix layer must not be damaged by the placement of soil cover and erosion protection materials.

A5.3.8 Construction Control and Testing

A5.3.8.1 Application Rate Check

The actual rate of bentonite application during spreading operations may be checked by passing the spreader over a premeasured tarp. The tarp is then weighed to determine the quantity of bentonite applied per unit area. The bentonite is then placed back on the test area. This check should be done by the engineer for every hectare of pond area.

A5.3.8.2 Tillage Depth

Frequent checks of the depth of tillage should be made by the contractor and engineer. A 2.5 cm increase in a 10 cm liner results in a 25% reduction in the bentonite mix ratio, which may increase permeability significantly.

A4.3.8.3 Compaction Control

Good compaction is essential for the creation of an adequate liner. Density and moisture content tests should be performed by the engineer on at least 10 locations per hectare in order to ensure proper compaction of the liner. The compacted admix should appear densely packed without visible voids or open structure.

A5.3.8.4 Permeability Tests

Representative samples of the compacted liner, either undisturbed or reconstituted in the laboratory to field density, should be tested in the laboratory to determine the permeability since coverage and mixing of bentonite in the field may not be as thorough as in samples prepared in the laboratory. This should be done at least once per hectare.

The American Colloid Company recommends the barrel test to measure seepage rates through completed bentonite admix liners. In this test, a 45 gallon drum with both ends removed is forced through the bentonite seal. The contact between the inside drum wall and the top of the liner is well sealed with a thick bead of bentonite slurry. The drum is then filled with fresh water for 2 days to allow the bentonite in the liner to hydrate. A second drum with an intact bottom is placed nearby, and both drums are filled to the same specified height. Water loss is recorded in each drum every day for at least a week, topping off the drums with additional water at the end of each day to maintain relatively constant heads. The seepage rate is then equal to the rate of water loss from the drum penetrating the liner minus the rate of evaporation from the second drum. If it is assumed that one-dimensional, steady state seepage is occurring through the liner, and that pore water pressures below the liner are atmospheric then the hydraulic conductivity of the liner may be calculated from the following equation, (A5.3) $K = vL/(H + L)$

where v is the seepage rate, L is the liner thickness, and H is the depth of water maintained over the liner. Greater than atmospheric pore pressures below the liner will reduce the hydraulic gradient through the liner, and equation (A5.3) will indicate a K value that is too low.

The barrel test is fairly crude and is subject to experimental errors that are greater in magnitude than the recommended seepage rate for admix liners of 2×10^{-10} m/s. However, the test may provide a quick check to ensure that very large seepage rates (larger than 1×10^{-9} m/sec) will not occur.

A5.3.9 Remedial Measures During Construction

If construction control tests or permeability tests indicate any inadequacies in the liner, the application rate should be increased in suspect areas. Re-blending and re-compaction should be done until the liner meets specified standards. High permeabilities may be due to insufficient or incorrect application rates (e.g. due to changes in soil type with location), too thin a liner thickness, or tilling too deep for the application rate.

In some cases a lagoon located in a natural clay deposit may have only a few areas of permeable soil (e.g., sand lenses) that require lining. Bentonite may be suitable to seal the permeable zones (i.e., by admixing with sandy and silty soils), but only if the permeable zones are easily and well identified, and if the bentonite "patches" extend well beyond the limits of the permeable zone and are well bonded with the native clayey soil. However, it is preferable to construct a uniform, complete liner over the entire lagoon area whenever there are doubts about the ability of the native soils to adequately control seepage.

A5.4 POTENTIAL FAILURE MECHANISMS

The failure mechanisms that may affect compacted clay liners may also affect bentonite admixes (see Section A4.3). In general, bentonite admixes are more susceptible than compacted clay to erosion and damage, and therefore must be protected by a soil cover (and gravel or riprap where required). Bentonite is less susceptible to drying than most other clays, particularly if covered by 30 to 60 cm of soil on the sideslopes. If drying does occur, the swelling properties of the bentonite allow considerable self-healing of cracks, particularly if bentonites are treated to prevent cation exchange and the like. However, migration of sand into fissures may prevent complete self-healing. Because they are thin, admix liners are more susceptible than compacted clays to rupture due to differential settlements, frost heave of the subgrade, and creep. A minimum depth of wastewater should be maintained in the pond at all times to prevent drying of the bentonite on the pond bottom and to protect the admix liner against frost heave (of the liner or subgrade).

TABLE A5.1

ATTERBERG LIMIT VALUES AND ACTIVITIES

(After Grim & Guven 1978)

	Plastic Limit (W_p,%)	Liquid Limit (W_L,%)	Activity (A)
Na-Bentonite	89-97	344-700	3.1-7.1
Ca-Bentonite	63-65	155-177	1.2-1.3

(Range of values for 3 bentonites from Mississippi, Arizona & Wyoming).

TABLE A5.2

PERMEABILITY OF BENTONITE-SAND MIXES (Grim & Guven 1978)

MATERIAL	MIX RATIO	HYDRAULIC CONDUCTIVITY K (m/s) at 65 kg/cm^2
Quartz sand: calcium bentonite	9:1	7.2×10^{-9} m/s
	7:3	3.5×10^{-10} m/s
	1:1	9.2×10^{-11} m/s
	0:1	3.3×10^{-11} m/s
Quartz sand: sodium bentonite	9:1	2.7×10^{-11} m/s
	7:3	5.0×10^{-12} m/s
	1:1	not measureable
	0:1	not measureable

TABLE A5.3

REPORTED VALUES OF HYDRAULIC CONDUCTIVITY FOR BENTONITE-SAND MIXES

MATERIALS	MIX RATIO BY WEIGHT	HYDRAULIC CONDUCTIVITY	REFERENCE
silt and bentonite (cations not described	2% 4%	4×10^{-9}m/s 10^{-10}m/s	Lacroix 1960
beach sand and Volclay SG-40 treated Na-bentonite (85% mod. Proctor)	6% 8% 10%	1×10^{-8}m/s 1×10^{-9}m/s 1×10^{-10}m/s	American Colloid Co. literature (1979)
fine silty sand to sandy silt and Envirogel Na-bentonite	5-6%	1×10^{-11}m/s to 3×10^{-10}m/s	Wyo-Ben, Inc. (private communication, 1982)
clean fine sand and Envirogel Na-bentonite	8%	3×10^{-10}m/s	" "
20-40 frac sand and Dresser Mineral Autobond and Arrowhead Na-bentonite. (95% Std. Proctor)	10%	1×10^{-10}m/s	tests by Komex Consultants, 1983.

APPENDIX 6
DESIGN AND CONSTRUCTION OF ASPHALT LINERS

This Appendix provides detailed information on the properties of asphalt as related to its performance as a liner (Section A6.1), and specifically on the design and construction of hydraulic asphalt concrete (Section A6.2), soil asphalt (Section A6.3), spray-on membranes (Section A6.4), and prefabricated asphalt panels (Section A6.5).

A6.1 ASPHALT PROPERTIES

A6.1.1 Definition of Asphalt

Asphalt is dark brown to black semisolid cementious material consisting of bitumens which gradually liquefy when heated. Asphalt may occur in natural deposits, or may be obtained from the residue from the refining of petroleum. This residue consists mainly of hydrocarbons and their derivatives which remain after the evaporation of the volatile components of petroleum.

A.6.1.2 Types of Asphalt

There are two types of asphalt as defined by their colloidal chemistry. The sol type, in which micelles move freely with respect to each other, exhibits high ductility, high susceptibility to temperature changes and a high rate of age hardening. The gel type, in which the micelles form a structural arrangement, has low ductility, low susceptibility to temperature, and low rate of age hardening. The gel type should be used for pond lining.

Asphalts used for lining purposes may be divided into asphalt cements and liquid asphalts. Asphalt cements are fluxed or unfluxed asphalts, specially prepared as to quality and consistency for direct use in the manufacture of bituminous pavement. Asphalt cements may also be air-blown in the presence of a catalyst. Liquid asphalt may be cutback asphalt or emulsified asphalt. Cutback asphalts consist of asphalt dissolved in solvents, while emulsified asphalts consist of asphalt particles dispersed in water. Upon evaporation, both cutback and emulsified asphalts leave a film or layer of asphalt. Cutback and emulsified asphalts are generally applied by spraying.

A6.1.3 Chemical Compatibility of Asphalt and Wastewater

The chemical compatibilities of asphalt-based liners and other lining materials are compared in the Tables presented in Appendix 14. Most of the reported test results on small samples cannot be extrapolated to field performance because tensile cracks would result in significantly higher migration rates. In general, asphalt based materials will likely degrade with time when exposed to organic and oil fluids (Matrecon 1980).

Asphalt materials are generally resistant to corrosion by acids, bases, inorganic salts at low (<30%) concentration, and hydrogen sulphide and sulphur dioxide gases. The list of individual chemicals which are successfully retained by asphalt liners is long, but most will fall under the above categories.

Asphalt liners are not suitable for the containment of organic solvents and chemicals (especially hydrocarbons), petroleum - derived wastes, fats, oil, aromatic solvents and hydrogen halide vapours.

In recent years various substances have been blended with asphalt in liner materials. Since these mixtures may not offer the same chemical resistances as 'pure' asphalt, tests should be performed to establish the compatibility of the liner material with the waste to be contained.

No coal tar, phenols, or similar materials should be included in asphalt mixtures for the purpose of lining water - containment facilities since these substances tend to contaminate water (Kays 1977). Asphalts from different sources may not be chemically compatible with each other. This is of concern in the manufacture of asphalt panels. Stewart (1978) recommends that only asphalts derived from petroleum residue be used for liners.

A6.1.4 Types of Asphalt-Based Lining Materials

Asphalt may be used in several types of lining materials. Hydraulic asphalt concrete is a mixture of asphalt cement and mineral aggregates, similar to asphalt concrete used in road-paving. Soil asphalt is a mixture of low plasticity soil and liquid asphalt. Membranes may be formed by spraying liquid asphalt, in various forms, directly onto soil or on synthetic fabrics. Asphalt panels are pre-fabricated sheets of asphalt, mineral fillers, and fibres. These uses of asphalt in liners are described more fully in the following Sections.

A6.2 HYDRAULIC ASPHALT CONCRETE (HAC)

A6.2.1 Properties of HAC

A6.2.1.1 General

Hydraulic asphalt concrete (HAC), also called 'bituminous pavement', is a carefully controlled mixture of asphalt cement and graded aggregate that is placed and compacted at elevated temperatures. When properly mixed and placed, HAC can form a stable, durable, erosion-resistant lining (Stewart 1978). HAC is similar to the asphalt concrete used for road paving, except it generally has a higher content of asphalt cement and mineral filler in order to produce an essentially voidless mix after compaction (Matrecon 1980).

A6.2.1.2 Behaviour of HAC as a Liner Material

Properly installed HAC serves as a vandal-proof, economical, and reparable liner. HAC is usually resistant to wave action, plant growth and temperature variations (Kays 1977). It is not adversely affected by water, resists slip and creep (at operating temperatures) and is flexible enough to withstand slight deformations and to avoid rupture under low level seismic activity. However, it is recommended that HAC be used in conjunction with spray-on seals.

Problems with HAC liners may result from non-homogeneity of the mix and difficulties in obtaining sufficient compaction, especially on sideslopes. Exposure to sunlight may result in age hardening and cracking of the liner. The rate of aging is slowed by earth covers and spray-on seals. HAC will crack at cold temperatures (possibly about -15°C to -20°C for AC-20 or AC-40 asphalt (E. Thompson 1983)). Therefore, earth covers are recommended on exposed sideslopes. Frost heaving is a major cause of HAC cracking and should be controlled by use of free-draining base materials and maintaining a depth of unfrozen liquid in the pond during freezing weather. The liner may be subject to ice damage, due mainly to brittleness of the asphalt material at low temperatures. Smooth versus open textured HAC and earth covers will limit ice damage. Due to its dark colour, an HAC liner will absorb heat, incubating weeds beneath the liner which may grow and rupture the asphalt. Thus, the subgrade soil should be treated with a sterilant, and earth cover vegetation should be short-rooted.

HAC liners should require little maintenance if properly constructed. However, several minor sources of potential damage should be watched closely. A build-up of algae and/or clay deposits at the water line may pull the bituminous seal from the liner surface. Scouring of the liner surface has been observed in HAC-lined canals. Timely treatment of the surface with an asphalt sealant will minimize these effects.

Cracks may occur in the liner, especially at pavement joints. Problems arising from these cracks may be prevented with multi-layer pavement construction with the staggering of joints. If necessary, conventional road asphalt report techniques may be applied to repair cracked liners.

A6.2.2 Design and Construction of HAC Liners

A6.2.2.1 Materials

HAC liners should be designed using 60-70 penetration grade asphalt (Asphalt Institute 1974), with 7 to 9% asphalt content and a well graded aggregate. Gel type asphalt should be used. The HAC should conform to ASTM D946.

The design should specify compaction to less than 4% voids. As indicated on Fig. A6.1, this is approximately equivalent to a Marshall density of 97% or greater and should result in hydraulic conductivity values that are sufficiently low to meet the K vs D_L requirement of Fig. 4.1 for 5 to 10 cm liners.

A6.2.2.2 Thickness

The recommended thickness is 10 cm comprising two 5 cm lifts with staggered joints. In small ponds on well-prepared, non-frost susceptible subgrades, a single 5 cm lift may be adequate. A schematic section of HAC lined pond is shown on Fig. A6.2.

A6.2.2.3 Seal

The HAC should be sealed with a 5 to 8 mm thick catalytically blown asphalt membrane. The spray-on asphalt should meet the standards of ASTM D2521.

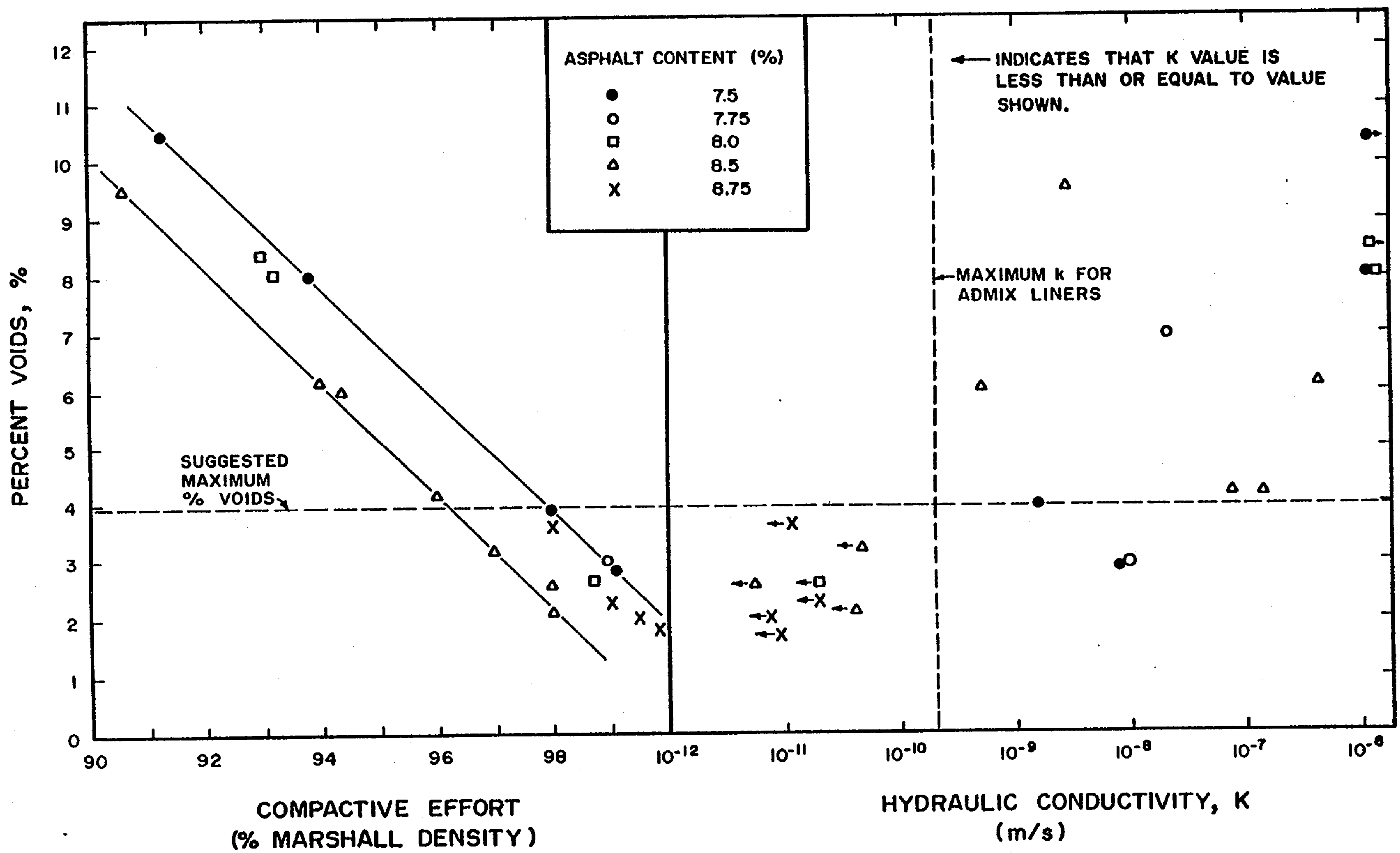


FIGURE A6.1 RELATIONSHIPS BETWEEN VOIDS, PERMEABILITY AND COMPACTION OF ASPHALT (DATA FROM HINKLE, 1976)

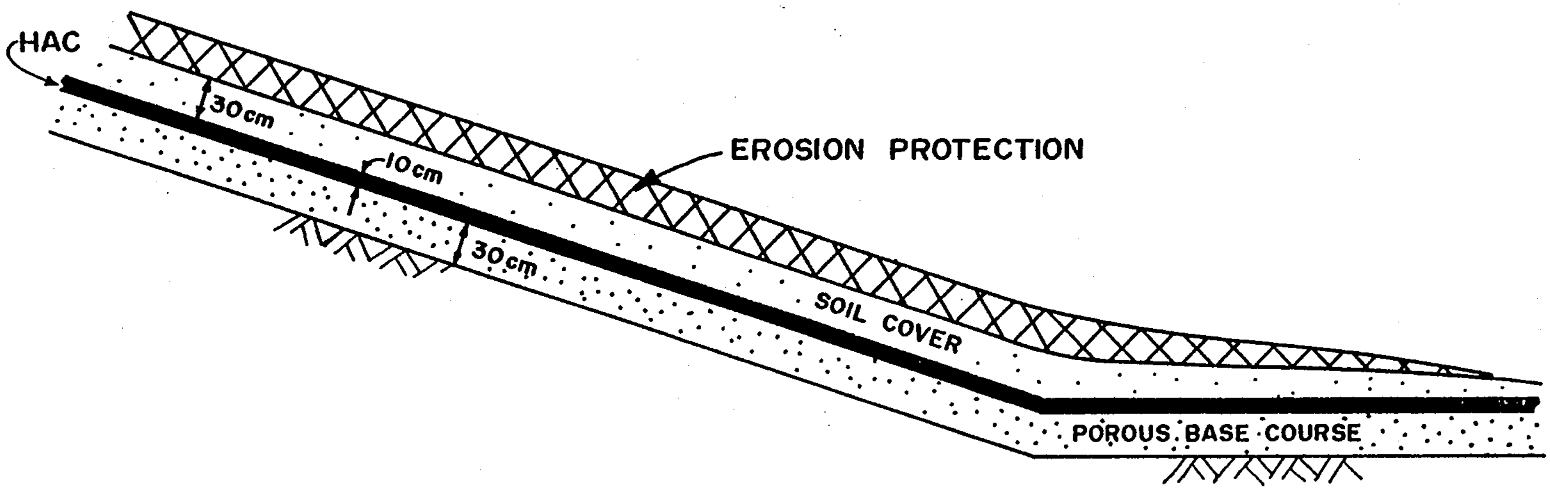


FIG. A6.2 SCHEMATIC CROSS-SECTION OF HYDRAULIC ASPHALT CONCRETE (HAC) LINER

A6.2.2.4 Base Course

The HAC should be laid on a relatively free-draining, compact layer of sand and gravel, crushed rock, or porous asphalt about 23 to 30 cm thick. Granular base courses should be drained by a network of PVC drainage pipes to a gravity outfall or sump. The porous asphalt should also have drainage provisions. Underdrainage reduces the potential for frost heave damage and hydrostatic uplift.

Granular underdrains should be compacted sufficiently to provide a good working base for the paving equipment. A coating of hot liquid emulsified asphalt meeting ASTM D2397 may be applied to the surface of the underdrain layer and allowed to cure for 48 hours. All components of the underdrain should meet applicable filter criteria requirements.

A6.2.2.5 Soil Cover

Although the spray-on asphalt seal limits air ingress and aging of the HAC, the HAC is still susceptible to cold cracking at temperatures below about -15 to -20°C. Thus, a 30 cm earth cover is recommended on the sideslopes as thermal insulation and as protection against mechanical damage. Non-frost susceptible subgrades are generally considered to be sands and gravels with less than 5% by weight passing the No. 200 sieve.

A6.2.2.6 Subgrade Preparation

The subgrade must be firm enough to prevent differential settlement and to provide support for construction equipment. All debris, vegetation, or other organic materials should be removed, and the site should be graded to the proper elevation. The subgrade should be smoothed with rollers and proof-rolled to at least 95 per cent of maximum dry density (ASTM D698). The surface should be treated with a sterilant to inhibit plant growth.

A6.2.2.7 HAC Placement

The hydraulic asphalt concrete may be placed with conventional highway paving equipment in lifts of 4 to 8 cm in thickness. On slopes, pavement should progress from bottom to top. Paving should take place in 3 to 5 m wide 'spreads' with no holes, depressions or grooves. The edges of these spreads should be sloped for 15 - 30 cm to allow sufficient bonding surface with the adjacent spread. Rolling should start as soon as the hot-mix material can be compacted without displacement, and should continue until throughly compacted and all roller marks have disappeared. In areas too small for the roller, compaction may be accomplished with hand tampers or vibrating-plate compactors. Asphalt around valves, inlets and outlets in the pond floor on slopes should be placed while the mix is hot. Each lift of pavement should be compacted to 97 per cent of the Marshall density, or so that the void content of the compacted lining is less than 4 per cent (ASTM D3203) (Asphalt Institute, 1974). The joints should be staggered if more than one 'lift' of pavement is placed (Matrecon, 1980).

A6.3 SOIL ASPHALT

A6.3.1 General

Soil asphalt is a general term refering to a mixture of soil of low plasticity with various forms of liquid asphalt (Stewart 1978). These liquid asphalts include cutback and emulsified asphalts. Many authors recommend that soil asphalts be avoided for liner purposes on their own. However, they may be used in conjunction with other liner materials, particularly for the construction of an asphalt-stabilized sand working table to support the installation equipment for other liner systems. A suggested procedure for the construction of such a working table is given by the Asphalt Institute (1974).

Cutback asphalt is a liquid solution of asphalt in volatile petroleum solvents. Specifications for cutback asphalts of the slow, medium and rapid-curing are given by ASTM standards D2026, D2027, and D2028. The selection of a cutback asphalt for various purposes may be made with reference to ASTM Standard Practice D2399. After the application of the solution to a soil surface, the solvent evaporates, leaving asphalt mixed with the crust of the soil. Soil asphalts made with cutback asphalts generally do not have low permeability, and the manufacture of a homogeneous liner is difficult.

Emulsified asphalt is a mixture of asphalt and water in which the asphalt is held in suspension by an emulsifying agent (small amounts of chemicals or clays). Emulsified asphalts should meet the requirements of ASTM Specification D977 (or D2397 for cationic emulsified asphalt). The selection of an emulsified asphalt for various uses may be made using ASTM D3628. Emulsified asphalt may be charged (anionic or cationic) or nonionic. Soil asphalts containing emulsified asphalts require a water-proofing seal to be effective as a liner.

A6.3.2 Design and Construction of Soil Asphalt Liners

A6.3.2.1 Materials

A soil asphalt liner will consist of a liquid asphalt (e.g., emulsified or cutback) mixed with a 15 cm thickness of native non-cohesive soil. The soil asphalt stabilized ground should then be sealed with 5 to 8 mm of catalytically-blown asphalt cement meeting the standards of ASTM D2521. A schematic section of a sealed soil asphalt lined pond is shown on Fig.A6.3.

It is recommended that an emulsified asphalt meeting the standards of ASTM D2397 be used for the soil asphalt layer. An application rate of about 100 to 125 L/m^3 is recommended. Trial mixes should be prepared to determine the moisture content required in the sand to

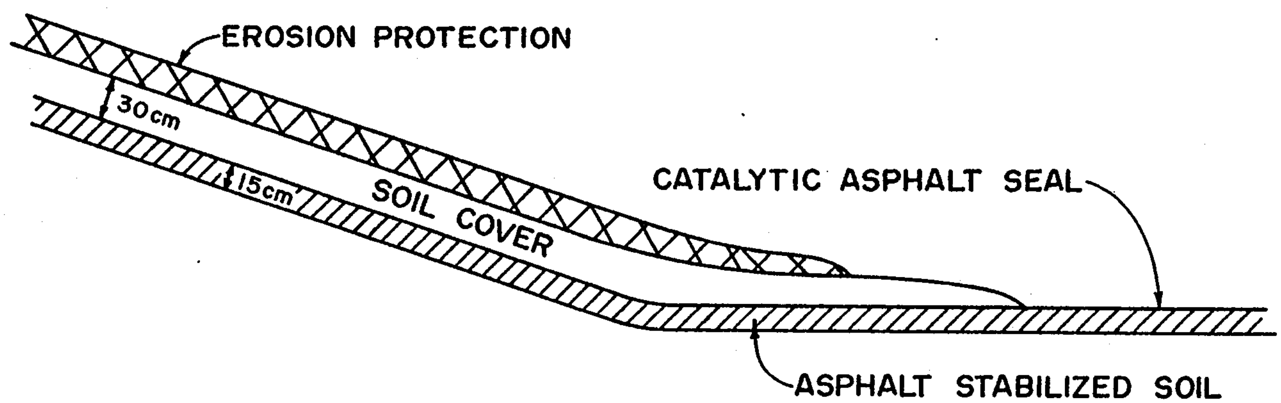


FIG. A6.3 SCHEMATIC CROSS-SECTION OF
SEALED SOIL ASPHALT LINER

ensure coating of grains by the asphalt. Water may be added to the soil or the emulsion prior to mixing, if required.

A6.3.2.2 Subgrade

Underdrain construction is not possible if the soil asphalt is mixed *in situ* rather than in a batching plant. In this case, the bottom of the pond should be protected with wastewater at all times during the late fall and winter months. The dyke material below the sideslopes should be relatively pervious and non-frost susceptible.

A6.3.2.3 Soil Cover

A 30 cm earth cover is recommended on the sideslopes to reduce the chances of cold cracking and mechanical damage. The earth should be covered with suitable erosion protection.

A6.3.2.4 Soil Asphalt

Detailed construction procedures for asphalt-stabilized soil are presented by the Asphalt Institute (1974). In brief, the subgrade should be graded to the required pond invert lines, removing all organics and debris and gravel greater than 2 cm in maximum dimension, and replacing soft or otherwise objectionable zones with suitable soil for asphalt stabilizing. The air temperature should be at least 10°C.

Mixing of the asphalt and sand may be carried out by trial plants, rotary or mechanical mixing, or by motor graders. The mix should aerate sufficiently to allow satisfactory compaction prior to spreading and compaction. Compaction should be carried out by pneumatic rubber-tire rollers or other pavement compaction equipment.

A6.4 SPRAYED-ON ASPHALTIC MEMBRANES

A6.4.1 Definition

Sprayed asphalt membranes are a continuous layer of asphalt, usually without fillers or reinforcement, which are formed in the field by spraying liquid directly on prepared surfaces (subgrades). They are usually 5 - 8 mm in thickness, but may be thinner when used in conjunction with HAC or soil asphalt.

A.6.4.2 Material Properties

Asphalts used in sprayed membrane must have a very low susceptibility to temperature, a high degree of toughness and durability, a high softening point and sufficient plasticity to avoid rupture due to earth movements.

A6.4.3 Types of Sprayed-on Membranes

The most successful type of sprayed-on asphaltic membrane is catalytic air blown asphalt. Asphalts used for catalytic air-blown purposes should meet the requirements of ASTM Specification D2521. In this process, a liquid asphalt containing a catalyst (usually phosphorous pentoxide or ferric chloride) is air-blown at 260°C at a pressure of 345 kPa. It is then applied at a rate of 4.6 L/m^2. This produces a 6 mm thick liner with two passes of the spray. The addition of 3 to 5 per cent rubber or other polymers may improve the properties of air-blown asphalt, resulting in lower permeability, higher elasticity and toughness, decreased brittleness, and more resistance to ageing.

Emulsified asphalt may also be used as a sprayed-on membrane. Although it does not generally produce an adequate liner when sprayed directly on a subgrade, emulsified asphalts may be sprayed on geotextiles to produce liners. Woven glass and nylon fabrics should be avoided for this purpose because of difficulties with salt solutions. Asphalts must have the proper 'strike-through' when sprayed on fabrics, i.e. enough coating to provide good adhesion but not so much as to penetrate the fabric and reduce its tear resistance.

A urethane-modified asphalt may be spray-applied to form a liner on a prepared surface. It is generally sprayed at about 0.28 gallons per square yard, producing a thickness of 50 mil. This material has good ultravoilet stability and low temperature ductility but is not recommended for storing hydrocarbons or organic solvents (Matrecon 1980).

Although sprayed membranes are not generally effective as liners on their own, they may be applied as a subgrade surface treatment prior to paving with asphalt concrete, or sprayed on the surface of other liners to produce a <u>bituminous seal</u>. This seal is usually a 2 mm thick layer of emulsified asphalt sprayed on the surface for additional water proofing.

A6.4.4 Performance of Sprayed Asphalt Membranes

Sprayed-on asphalt membranes are not generally adequate liners. They are prone to sun-ageing and damage. It is difficult to control liner thickness and to prevent pinholes (Matrecon 1980). Sprayed liners tend to heat quickly and creep on sideslopes.

A6.5 PREFABRICATED ASPHALT PANELS

A6.5.1 Constituents and Manufacturing

Asphalt panels are manufactured from air-blown asphalt and mineral fillers and fibres. The materials are mixed in an open mixer at 120 to 150°C to produce a mixture with 65 per cent asphalt by weight (Kays 1977). The specific gravity of the material is 1.1 to 1.2, although higher specific gravities may be produced by altering the mineral content. The above mixture is extruded through a roller head die, during which time both sides of the panel are lined with an asphalt-impregnated geotextile to add tensile strength to the panel. Tests by Matrecon (1980) showed that the fabric helps the liner retain strength, even though the asphalt may swell and lose strength when in contact with some waste liquids. After extrusion, both surfaces are coated with liquid asphalt and the product is submerged in a water bath for cooling. Once this process is complete, the material is cut into panels of 3 to 12 mm thickness, 0.9 to 1.2 m wide and 3 to 6 m long (Kays 1977).

Prefabricated asphalt panels without a soil cover should meet the specifications of ASTM D2643. The liner should be sufficiently flexible to conform to the contour of the prepared subgrade and tough enough to withstand the anticipated service environment.

A6.5.2 Installation of Panels

ASTM D3745 outlines a recommended practice for the installation of prefabricated asphalt liner materials complying with ASTM D2643.

A6.5.3 Standard Requirements for Prefabricated Asphalt Liner Adhesives

In order to join butted prefabricated asphalt panels, joint sealants (either hot-mopped asphalt or cold-applied asphalt mastic) and batten or cover strips for the joints are required. The hot-mopped joint sealant should satisfy ASTM D312. Cold applied joint sealants should meet the requirements of ASTM Specification D2822. All sealants should be approved by the manufacturer to ensure the compatability of the asphalts. The batten or cover strips should satisfy Specification D2643.

Heat welding techniques may be used to join overlapped panels. However, this requires specialized labour and may be impractical for small liner installations.

APPENDIX 7
DESIGN AND INSTALLATION OF FLEXIBLE POLYMERIC MEMBRANE LINERS

A7.1. GENERAL

Plastic and synthetic rubber membranes have become increasingly popular as liners for surface impoundments over the last 30 years. Membranes include thermoplastics, elastomers (rubbers), and combinations of the two. The basic compound is calendered or extruded into sheets, usually fairly narrow and long, which are then factory-seamed to make panels as large as 30 m x 60 m. The panels are seamed in the field to line an impoundment. The membranes are very thin, typically 20 to 40 mils[1], and occasionally up to 100 mils thick (HDPE). To be effective, a liner must have very low permeability to water transmission and must be strong enough to resist damage during installation and the service life of the liner.

A7.2. TYPES OF POLYMERIC LINERS

There are many types of polymeric liner materials and often many variations of one type of polymer, depending on additives, processing procedures, etc. Thermoplastic polymers are generally soft and flexible. Strength and deformation properties vary with temperature, the sheets becoming softer with heat and stiffer with cold. Examples of thermoplastic materials are polyvinyl chloride (PVC) and polyethylene (PE).

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1 A mil is equal to 0.001 inches, and is the most common unit for liner thickness (1 mm is equal to 39 mils).

Elastomers have rubber-like properties, being elastic and relatively unaffected by temperature. The elastic nature of elastomers is created by molecular cross-linking of polymer molecules, a process called vulcanization or curing. This is brought about by applying heat and pressure to the material, together with a vulcanizing agent (usually sulpher) to cause the cross-linking. Examples are butyl rubber, EPDM (ethylene propylene diene monomer), Nordel, and elasticized polyolefins (e.g. 3110).

Many polymers may be produced in vulcanized (elastomeric) or unvulcanized (thermoplastic) form. Vulcanized liners may be stronger and more chemically resistant, but thermoplastic versions of compounds like chlorinated polyethylene (CPE) and Hypalon are more common. Thermoplastics are easier to seam and repair in the field. Table A7.1 shows several types of polymeric liners and their availability as vulcanized or thermoplastic material. The characteristics of some of the more common polymer liner materials are summarized below.

TABLE A7.1 POLYMERIC MATERIALS USED IN LINERS (after Matrecon 1980)

Polymer	Available as Thermoplastic	Available as Vulcanized	Available as Reinforced	Available as Unsupported
Butyl rubber	-	X	X	X
Chlorinated polyethylene	X	X	X	X
Hypalon	X	X	X	NR
Ethylene propylene rubber	X	X	X	X
Neoprene	-	X	X	X
High density polyethylene	X	-	-	X
Polyvinyl chloride	X	-	X	X

NOTE: X indicates available.
NR indicates not recommended.

Polyethylene (PE)

Polyethylene is made from ethylene monomer resin and generally does not contain other additives besides 2-3% carbon black to improve resistance of the polymer to degradation under ultraviolet (UV) radiation from the sun.

PE is a natural thermoplastic, but is a relatively stiff material because plasticizers cannot be added to make the material softer and more flexible. PE comes in two forms, low density polyethylene (LDPE) and high density polyethylene (HDPE) (i.e. high molecular weight). Increased density results in increased strength and stiffness.

Low density PE is used for vapour barriers, plastic bags, etc., generally in thicknesses of 2 to 10 mils. Thicker sheets of LDPE are stiff and difficult to handle. It is relatively weak, easily damaged and abraded, and degrades under outdoor exposure very rapidly.

LDPE must be covered by earth or fluid. The excellent resistance of PE to water transmission as well as its inertness in many chemical solutions also results in seaming problems. Solvents will not attack the PE material to form solvent bonds and the melting point range is narrow, making heat welding of thin liners tricky. LDPE is usually seamed with sticky tape in the field and the seam is thus a highly vulnerable part of the liner. The authors of this manual do not recommend LDPE as a liner material for impoundments.

HDPE, on the other hand, can be an excellent liner material, and is made in thicknesses of about 40 to 100 mils. HDPE is tough and resistant to damage. It has a much superior resistance to UV radiation if it includes 2% carbon black, and it is resistant to a wide variety of chemicals. Seaming is done by heat welding which requires good control of temperature, no moisture, and has minimum temperature restrictions. HDPE is very thermoplastic and considerable shrink and swell can occur from day to night. If seaming is carried out in warm weather, then sufficient slack must be left in the sheets to avoid high stresses in the liner and lifting of the liner from the bedding surface at the toe of the slopes during cold periods. On the other hand, too much slack may result in problems with wrinkles during seaming. In some cases seaming at night may be warranted.

Caution should be exercised in the use of thin HDPE liners because they have not been widely used, are more difficult to seam, and are easier to puncture than thick HDPE liners. There may be a tendency to think of HDPE as being indestructible, particularly on the basis of a 100 mil sample, whereas 40 mil material must be handled very carefully (like any other flexible membrane).

Polyvinyl Chloride (PVC)

Polyvinyl chloride, one of the oldest and most commonly used polymeric liner materials, has been in use since the 1950's. It is made of a vinyl chloride monomer compounded with plasticizer to make it softer and more flexible. Carbon black is used to stabilize the polymer against UV attack. Although it is a relatively strong, flexible material and ozone resistant, PVC needs to be covered with earth or fluid to protect it from heat and volatilization of the plasticizer. PVC bonds well electronically in the factory and bonds easily in the field with solvents. PVC has a low resistance to hydrocarbons, oils, and solvents (unless it is compounded with nitrile or other oil-resistant plasticizers). It is recommended that PVC liners be at least 20 mils thick to reduce the number of punctures during placement of soil covers.

Chlorosulfonated Polyethylene (Hypalon)

Hypalon is a polyethylene polymer in solution with chlorine and sulfur dioxide. It is generally a thermoplastic but can be vulcanized. Very ozone and weather resistant, it can be left exposed. Hypalon should be reinforced because it is a relatively weak material and susceptible to shrinkage. Thicknesses range from 30 to 60 mils, generally comprising 2 plies of plastic and scrim, and sometimes 3 plies with 2 scrim layers. Hypalon resists many acids and alkalines, as well as attacks by mold, mildew and fungus, but has a poor resistance to oils. It is solvent bonded in the field, but may cure slowly with time (cross-link) thus making repairs more difficult. Hypalon has a specific gravity of nearly 1.5, and therefore will not float in brines.

A7.3. PROPERTIES AFFECTING LINER PERFORMANCE

A7.3.1 Permeability

The ability of a liner material to impede the passage of the impounded fluid is obviously of prime concern. Most polymeric membranes are described as being "impermeable" because of their continuous, voidless structure. The amount of seepage occurring below an intact piece of membrane in a standard permeability test is generally not measureable. Nevertheless, a finite quantity of fluid is observed to escape from membrane-lined impoundments (Kays 1977.)

Possible modes of seepage through polymeric liners include: seepage through pinholes (manufacturing defects), punctures and tears (mechanical damage during transportation, storage, installation, or service life), separated or defective seams, and transmission of fluids through the intact material under chemical (osmosis) or vapour pressure differentials. It is difficult to account for seepage through discontinuities such as holes or improper seams, except in a qualitative fashion. The stronger and thicker the membrane, the less likely it is that holes will occur. LDPE is relatively weak and abrades easily, while HDPE is quite tough. Nevertheless, HDPE can still be easily damaged by improper handling. Flexible membranes such as PVC, CPE, and

Hypalon have an intermediate strength and toughness. Material strength is discussed in more detail in the next Section.

Seepage through seams can occur due to discontinuities in either the solvent or welding along the length of the seam, "fish mouths" due to wrinkles, poor seams that are later separated under stress, and seams that separate because of reaction with the impounded fluid. Welded seams are generally superior to solvent bonded seams, which in turn are superior to sticky tapes and mastic.

The placement of earth covers over membrane liners can cause puncturing of the liners. The number of punctures will depend on the liner material, the coarseness of the subgrade and cover soil, the thickness of cover (first lift), the method of placement, etc. Gunkel (1981) reported the number of punctures caused by placing sand over 20 mil PVC, 30 mil CPE, 36 mil Hypalon, and elasticized polyolefin (3110), with variable subgrade conditions (sand, gravelly sand, coarse gravel, sandy silt over coarse gravel, and 5 cm crushed gravel). A D-7 bulldozer and rubber-tired loaders with contact stresses of 62 kPa and 221 kPa, respectively, were used to place the cover over the liners, and then trafficked back and forth over the cover soil for 30 passes. The number of punctures were recorded in 0.5 m^2 sections of liner removed from the test sections.

The results of the tests are summarized on Figs. A7.1 and A7.2. In all cases where liners were underlain by gravelly subgrade materials, punctures resulted. Some punctures also resulted when sand or silt were used as subgrade materials. For material M-1 (elasticized polyolefin) the number of punctures were generally (but not always) lower for the tracked vehicles than the rubber-tired loader. Increasing the sand cover thickness from 15 cm to 45 cm also helped, and there was some beneficial effect from placing a light nonwoven fabric between the liner and gravelly subgrades. The same comments apply to the PVC membrane

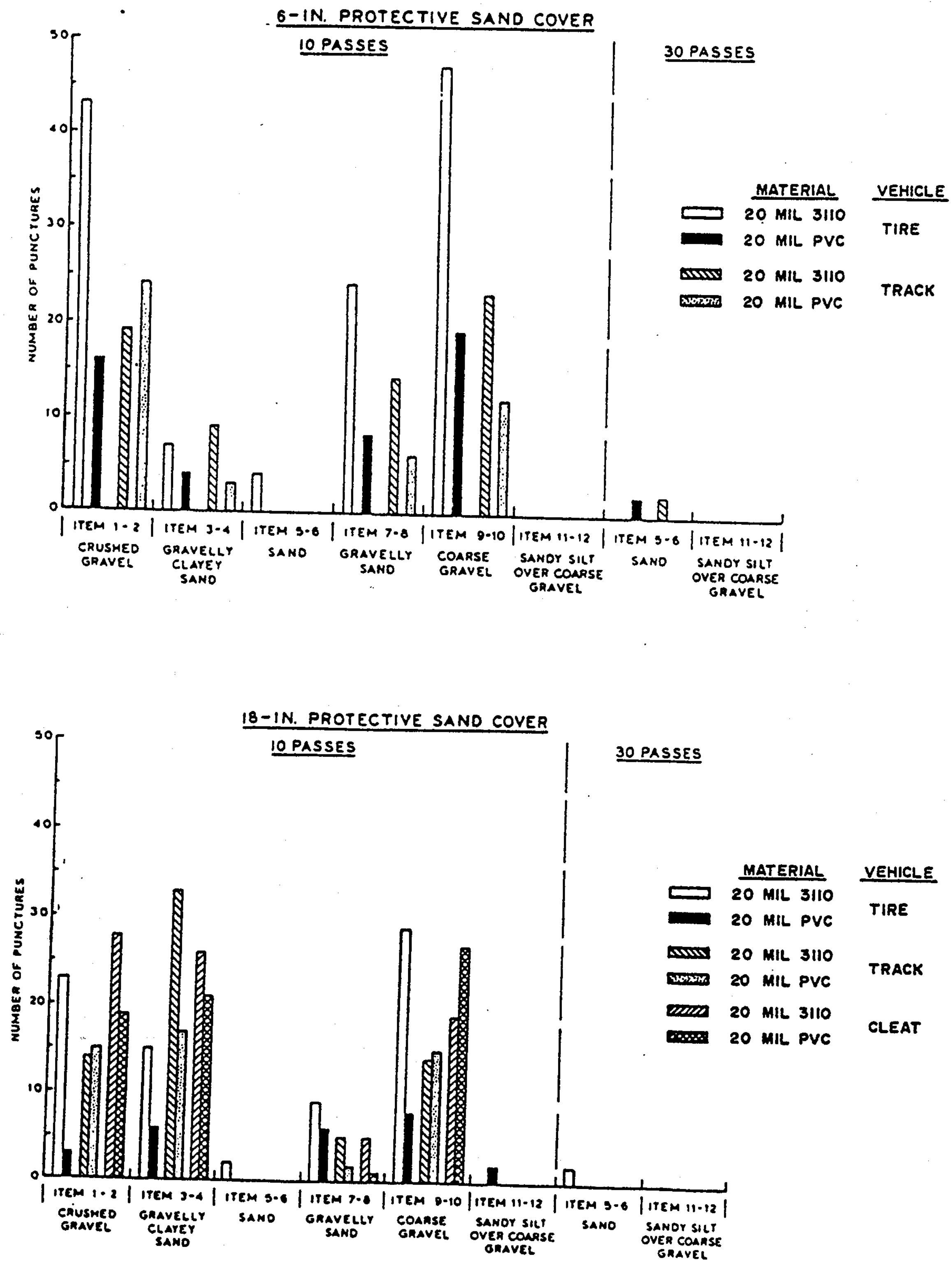
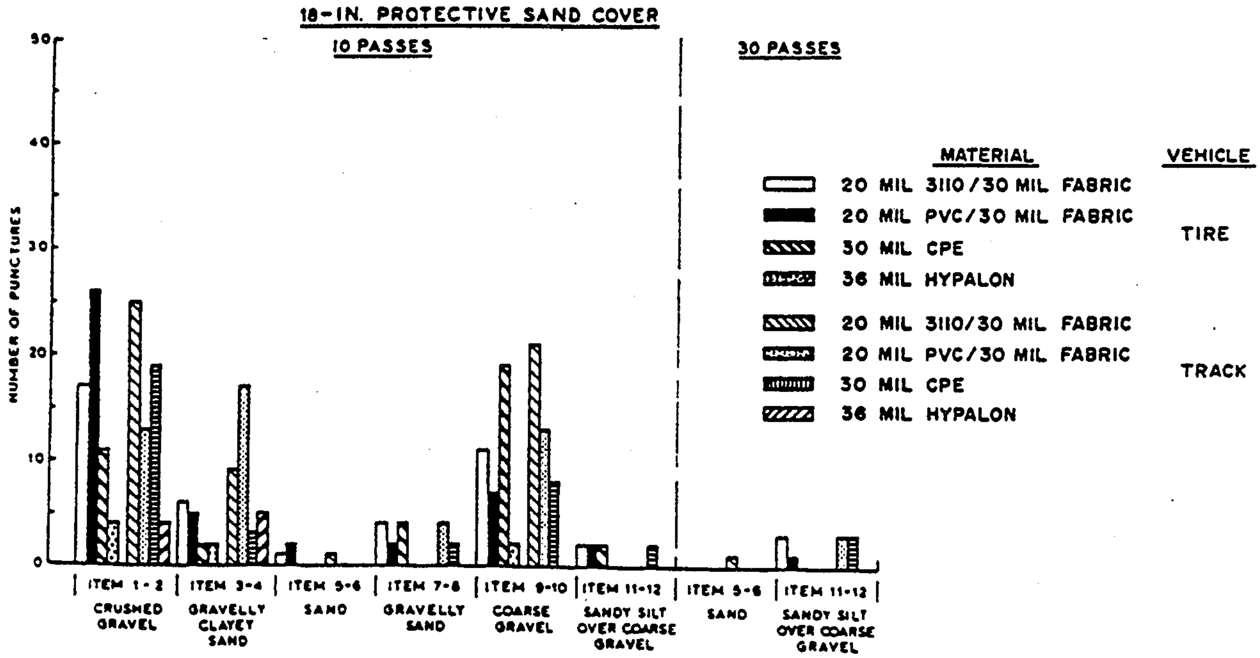


FIG. A7.1
LINER PUNCTURE DUE TO COVER PLACEMENT (GUNKLE 1981)



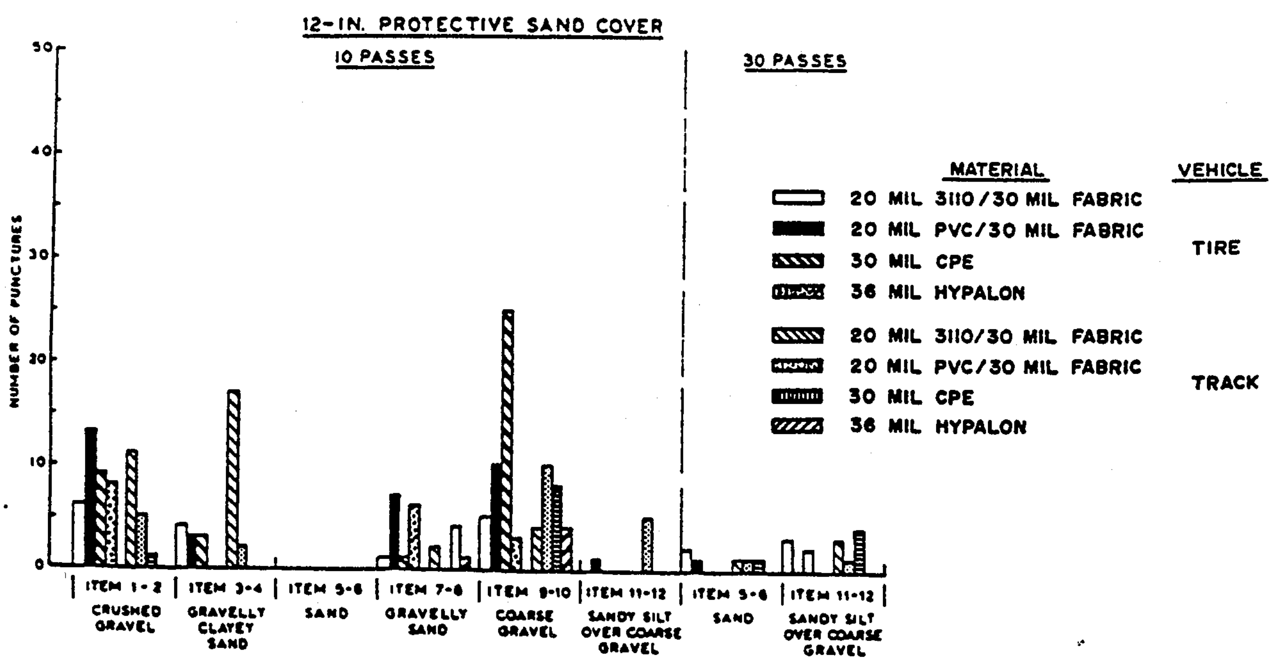


FIG. A7.2
LINER PUNCTURE DUE TO COVER PLACEMENT (CONT'D)

(M-2), although there was less benefit from the fabric. The CPE and Hypalon membranes (M-3 and M-4) did not experience significantly fewer punctures when the cover thickness was increased from 30 cm to 45 cm. In summary, it appears that some punctures may be inevitable when cover soil is placed over typical 20 to 36 mil membranes by tracked or rubber-tired vehicles with typical lift thicknesses of 15 cm to 45 cm. A minimum cover lift of 30 cm is recommended herein. There is a significant increase in the number of punctures if the subgrade material (or, presumably, the cover material) contains gravel.

Some polymeric membrane suppliers report the "perm" rating of a liner, which is the rate at which water vapour passes through the material under a vapour pressure differential at a constant temperature (ASTM E96 BW). These are frequently reported in units of grams/24 hours/m^2 or gals/acre-day, or sometimes as a permeability value, K, in cm/sec. The latter interpretation is not correct since the perm rating describes the transmission of a gas, not a fluid, under vapour pressure rather than hydraulic head differentials. The perm test conditions have no direct relation to conditions adjacent to a liner in service, although the BW test procedure involves wetting the top side of the test specimen.

Based on observed pond performances, Kays (1977) suggests minimum seepage rates to be expected for various liner materials under a 6 m head after 1 year of service. For an exposed, 45 mil synthetic membrane, Kays suggests that 3×10^{-10} m/s is the minimum rate to be expected. Assuming that seepage is primarily due to hydraulic pressures and discontinuities in the liner and is proportional to liner thickness, this suggests an "apparent" K value of about 5.5×10^{-14} m/s. It should be noted that the apparent low permeability of a synthetic membrane is partially offset by the thinness. Thus, while K may be 4 or 5 orders of magnitude lower than for a clay liner, the thickness is also 4 or 5 orders of magnitude less, resulting in seepage rates (assuming hydraulic flow principles govern) that may not be much lower than for a good clay liner. Nevertheless, the integrity of a synthetic membrane is usually easier to ascertain than the properties of a heterogeneous compacted clay liner.

The most important aspect of membrane liner permeability may be compatibility with the impounded fluid. If the fluid causes the liner material or seams to degrade, then it will likely fail in its function to control seepage. Liner/fluid compatability is discussed in more detail below and in Appendix 14.

In conclusion, it may be assumed that most commercial polymeric liner materials will sufficiently impede seepage of the impounded fluid if the liner is well-installed. The liner should be made from high quality, virgin material, supplied by a reputable, experienced manufacturer. The liner material should be compatible with the contained fluid, and should be strong enough to withstand installation procedures, bedding soil conditions, and the service environment without significant damage or degradation to the liner material or seams.

A7.3.2 Stress-Strain Behaviour

A7.3.2.1 General

A number of laboratory tests have been developed to measure stress-strain parameters for membranes. The purpose of this Section is to provide background information on the common types of tests to help the design engineer compare specifications for different materials. In most cases, the results are very specific to the load-time conditions of the test and may not be directly correlated with other tests or field performance. Plastics are sensitive to temperature and strain-rate conditions and rubbers may be sensitive to temperature conditions. Most membranes exhibit some degree of anisotropy. The tests are useful as indices for qualitative comparison with other materials, empirical correlations with field performance, and for quality control. The use of parameters in engineering design, however, would require modification of tests to model field loading conditions, boundary conditions, temperature, strain rate, etc. The repeatability of many tests is not great and numbers quoted are usually averages or means with undisclosed standard deviation. The test types commonly performed are tensile strength, tear strength, burst strength, bonded seam strength, and impact resistance at low temperature. The test standard used depends on the type of material. These are summarized in Table A7.2. There may be more than one type of test for a particular parameter and material, and within that test there may be two or more procedures. To compare results, the same test must be performed under exactly the same conditions. Preferred tests are shown first on Table A7.2. There may be a tendency for some manufacturers and suppliers to show the results of tests that give the best values. Some test procedures may be inappropriate or are discontinued standards. The individual test types are discussed below.

A7.3.2.2 Tensile Strength

There are a variety of tensile tests depending on material type and also on procedure used for some of the tests. As seen in Fig. A7.3, the tests involve a wide variety of specimen shapes, dimensions, clamp widths, strain rates, etc. The three major types of tensile tests are: strip tests with 25 mm wide strips stretched between 25 mm jaws; grab tests with 100 mm wide specimens in 25 mm jaws; and dumbbell specimens. In the strip and dumbbell tests the applied force is divided by the initial width of the specimen at the narrowest point and thus the tensile strength is reported as a force per unit width. The thickness of the membrane should also be reported. If the tensile strength properties are linearly proportional to thickness, then the results may be reported as applied force divided by the initial membrane cross-sectional area, i.e. kPa or psi units. The elongation of the specimen is usually measured as the distance between the clamps or jaws

TABLE A7.2
PHYSICAL STRENGTH PROPERTIES OF MEMBRANE LINERS

	ASTM STANDARD				
PARAMETER	PLASTIC(1mm)	PLASTIC(1mm) (14mm)	RUBBER	COATED FABRIC	JUST FABRIC
Tensile Strength	D882	D638	D412	D751	D1682
Tear Strength	D1004 D1922	D1004 D1922	D624	D751	D2261 D1424
Burst Strength				D751	
Hydrostatic Resistance				D751	
Bonded Seam Strength	D3083	D3083	D3083	D751	D1683
(Low Temperature) (Impact)	impact D1790 10mils or less	impact D746 (B)	impact D746(elastomer) (B) D2137(rubber)	impact D2137	
Dimensional Stability 212°F,15 min 7 days	D1204 212°F, 15min	D1204 212°F,15min	D1204 212°F, 7 days	D1204	

- tests developed for packaging, clothes, car seats, etc.

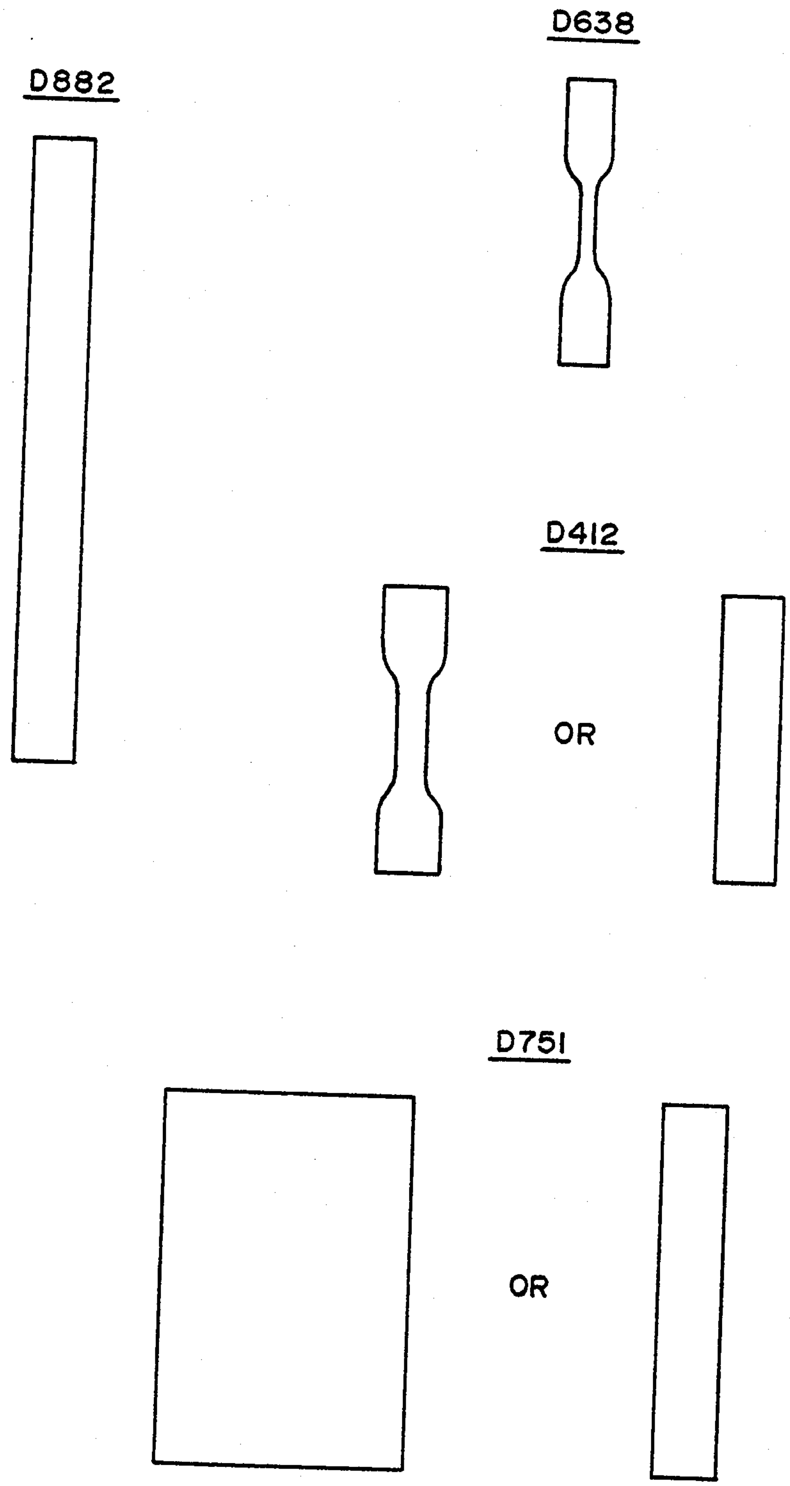


FIG. A7.3 TENSILE TEST SPECIMEN SHAPES ACCORDING TO VARIOUS ASTM STANDARDS of the testing apparatus. Once the plastic begins to yield, (past the linear portion of the stress-strain curve) the strip usually "necks in", i.e. the width of the strip decreases as the strip stretches. At this point the strain is not uniform throughout the strip, and the elongation is an average value. Sometimes the elongation is only measured over a smaller "gauge length" marked near the middle of the strip where most of the elongation occurs. This gives a larger value of elongation. Because of necking in, the cross-sectional area of the strips decrease after yield, and thus the <u>true stress</u> is actually greater than that determined using the initial width of the strip. To avoid the effects of necking in, the grab tensile tests were developed with the wider specimen in the narrow grips. This, however, makes it impossible to determine the relevant cross-sectional area or width resisting the strain, and thus results should only be reported as an applied force.

Because loading conditions are likely to be plane strain, at least for simple models, several tests have been developed that are essentially wide strip tensile tests where the width of the sample and clamps are the same and large compared to the distance separating the clamps. Thus, the necking in effect at the edges has a small effect on the overall specimen. Standards have not been set for this type of test.

Parameters usually reported for tensile strength tests are tensile strength, elongation at break, and sometimes tensile stress and elongation at yield (Fig.A7.4). In addition, secant modulus at a specified strain (usually 100% for unsupported membranes) or the elastic modulus may be reported. The elastic modulus is defined as the tangent modulus of the initial linear portion of the stress-strain curve (ASTM D882) as shown on Fig. A7.5.

In summary, there is a wide and confusing variety of tests for tensile strength. The individual tests are not generally correlatable with each other, and may not have a high reproducability. No correlation with tensile properties in the field should be construed. Thus, the texts

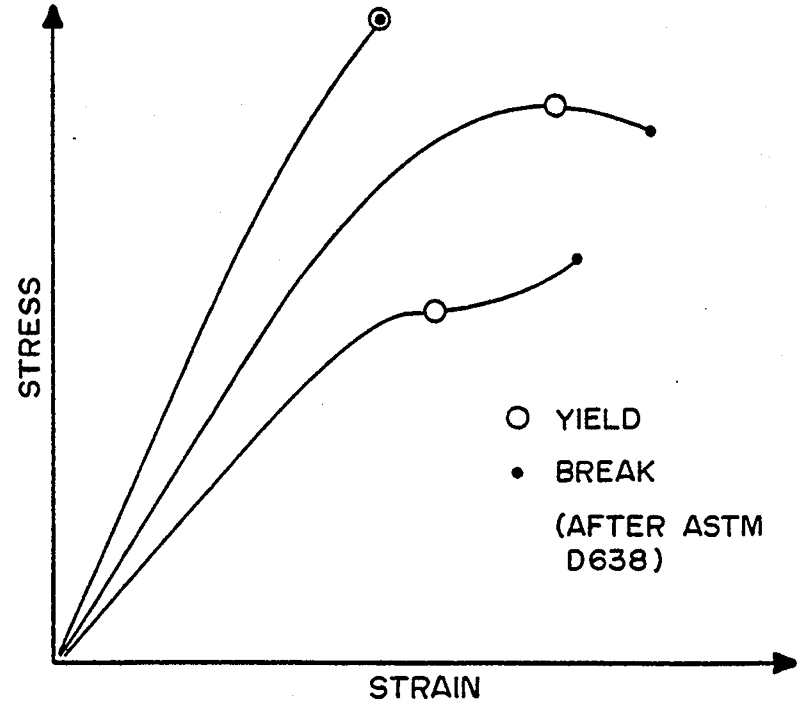


FIG. A7.4 TYPICAL STRESS-STRAIN BEHAVIOURS FOR PLASTIC SHEETS IN TENSION

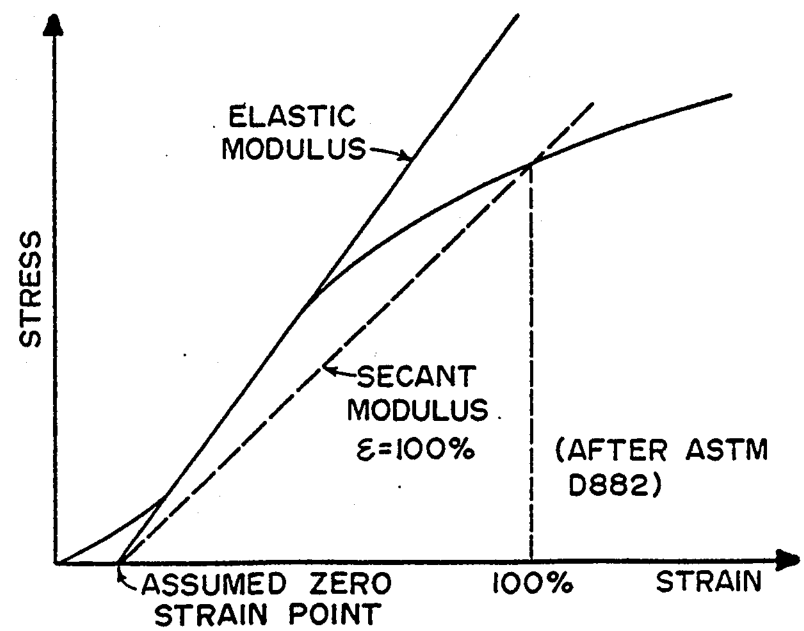


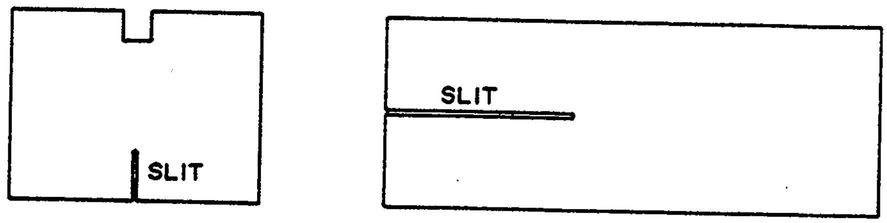
FIG. A7.5 DETERMINATION OF ELASTIC AND SECANT MODULI are index values only, useful for comparing materials when tested and measured in identical manners, for quality control, and qualitative evaluation of field performance based on experience.

A7.3.2.3 Tear Strength

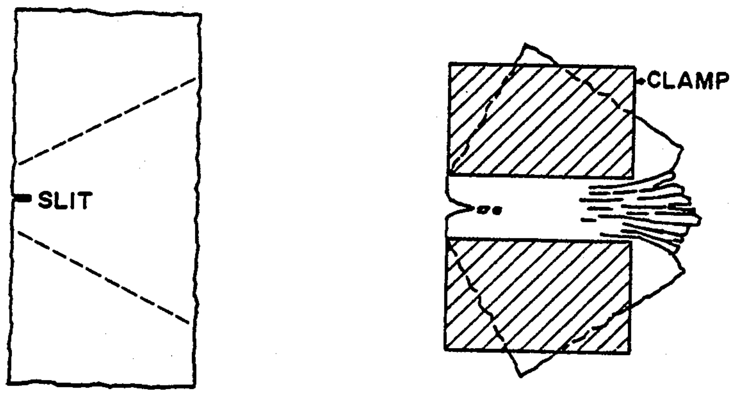
Tear strength is likely more important than tensile strength for liner performance (Kays 1977). The tensile strengths and moduli of most membranes are sufficiently high to make tensile failure unlikely (although it is recommended that weak materials such as CPE and Hypalon be reinforced on sideslopes steeper than about 4:1). Punctures, nicks, and other stress concentrators are more likely to initiate a rupture in a liner under stress, i.e. by tearing. There are as many, if not more, tear tests than tensile tests (Fig. A7.6). These generally involve the use of a nick or cut to create a stress concentration, and then the specimen is stretched as in tensile tests. The results of tear tests may be reported as a force, or a force divided by the thickness of the membrane (although according to ASTM D1004, the results for thin plastic sheets cannot be normalized with respect to thickness). As with tensile tests, both the machine direction (MD) and traverse direction (TD) of the membrane should be tested.

It should be noted that even within one test standard, e.g. D624, the results of tests on the 3 different specimen shapes cannot be compared. In general, the Graves test is preferred over the Elmendorf for unsupported membranes because of the greater reproducability of results for the former. In trapezoid tear tests or tongue tear tests there is often a variation from the ASTM standards with respect to slit length and sample dimensions. Therefore the results of trapezoid tests by different labs may not be directly comparable.

D751

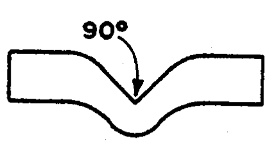


TRAPEZOID TEST



D624 (GRAVES TEST)

D1004 AND D624
(GRAVES TEST)



D1922
(ELMENDORF TEST)

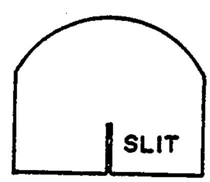


FIG. A7.6 TEAR TEST SPECIMEN SHAPES
ACCORDING TO VARIOUS ASTM STANDARDS
(NOTE: FIGURE IS FOR ILLUSTRATION PURPOSES ONLY)

A7.3.2.4 Burst Strength

The application of pressure, normal to the plane of a membrane with clamped edges, introduces axisymmetric tensile stress in the membrane; this is known as a burst test.

Burst tests are normally only specified for fabric reinforced membranes because their relative rigidity or resistance to extension makes them more liable to encounter situations where failure could occur by bursting under pressure; for example, a reinforced membrane bridging a void or depression in the soil. Unsupported membranes would tend to follow the profile of the depression with relatively low induced stresses because extension is likely to be less than the 200 or 400% required to cause failure (assuming that the results of strip tensile elongation tests are applicable). Pressure is either applied by fluid within a rubber diaphragm, by turn screw, or by a rising column of water.

The usefulness of the burst test as an index value or an engineering parameter is not at present clear.

A7.3.2.5 Low Temperature Impact Strength

Membranes, and thermoplastics in particular, are more vulnerable to damage at cold temperatures when their behaviour is brittle.

Several low temperature impact test standards are shown on Table A7.3. It should be noted that D1790 originally applied to films less than or equal to 10 mils, which would exclude all common membranes; however, it is the test recommended by the National Sanitation Foundation (NSF) and the one which manufactuers have tended to use for plastics. It should be noted that for D1790 the temperature reported is the temperature at which 50% of the specimens would probably fail 95% of the time. The test involves dropping a free-fall hammer onto a loop of the membrane at

TABLE A7.3 TEMPERATURE TEST DATA FOR POLYMERIC LINERS

MATERIAL	COLD TEMPERATURE TESTS Impact test D2137	Mandrel test D2136	Impact test D746	Impact test D1790	MAX.RECOMMENDED SERVICE TEMP.	REFERENCE
20-30mil PVC					71	B.F.Goodrich
				-29		Gardewick
				-29		Can.General Tower
				-26 to -29		Synflex
30 mil DRS(R)	-65	-60	-54			Gardewick
					93	Dow
		-54	-54			Synflex
		-65				
30 mil CPE(R)				-32		Synflex
		-40		-29	70	Dow
30-36 mil Hypalon(R)	-34		-43		70,107	Dupont
	-43					Burke
	-43		-43			Synflex
					66	Kays(1977)
70 - 80 - 100 mil HDPE			-76 (80 mil)		80	Schlegel
			-40			Gundle
Butyl					93	Kays(1977)
					82	Exxon
Nordel			-68		100	Dupont

NOTE: The above data were extracted from manufacturers' and suppliers' product literature. Values were not reported unless the test standard (except for maximum temperature) were reported. In general, no correlation exists or should be attempted between test types. Considerable variation for similar sheeting could result for the same test procedure, depending on a number of factors.

decreasingly cold temperatures. The weight, drop, and fold conditions are arbitrary.

The specifications are not necessarily the lowest temperature at which a material can be used (i.e. if actual load-deformation conditions are not as severe then lower working temperatures may be applicable, and vice versa).

D746 involves a cantilever of plastic that sticks out from a clamp, and is then struck by a striking edge. The A method is the temperature at which 50% fail; the B method recommended by the NSF is the minimum temperature at which none of 5 tests fail. This will clearly be a higher temperature. Nevertheless, the temperature determined by either method is arbitrary. The same test method should be used when comparing materials. Tests carried out by the American Society for Testing and Materials indicate that the results of D746 have a precision within about 8°C with a 35% probability if performed by different labs or individuals following identical procedures as specified in the standards.

D2137 is the low temperature impact test for coated fabrics, but is essentially the same test as D746 for plastics and elastomers. Frequently the cold bend "mandrel" test is run instead of (or together with) the low temperature impact test. The mandrel test (D2136) involves bending the membrane around an 1/8 mandrel at increasingly colder temperatures. According to Kays (1977), the mandrel test gives "pass" temperatures that are some 20 to 50°F lower than the impact test, thus the tendency is to report the mandrel test result. The impact test is considered by Kays to be more meaningful, although neither test is an indicator of the actual minimum service temperature the membrane can withstand.

The specified minimum cold temperature test values for different materials and manufacturers are shown on Table A7.3.

A7.3.2.6 Seam Strength

The strength of factory and field seams is clearly a vital property in liner performance. The quality of seams ranges from mastic and sticky tape to extrusion welding of the material, although any type of seam can be poor if done improperly. The strength of both factory and field seams should be tested and specified since the seaming methods and environmental conditions are likely to be different. D757, the standard for strength testing of coated fabrics, includes a procedure for a grab tensile test of a seamed strip. Kays (1977) recommends that the seam be tested in both shear and peel modes (Fig. A7.7).

The NSF recommends that seam strength be at least 80% of the material strength.

A7.3.2.7 Elevated Temperature Tests

All of the ASTM standard tests discussed above are run at either Standard Laboratory Atmosphere temperatures of 23 ± 2°C, or at cold temperatures. Polymeric materials, and particularly thermoplastics, lose strength and increase in flexibility with increasing temperatures. Yield strength and elongation versus temperature are shown for Schlegel high density polyethylene in Fig. A7.8.

Consideration should be given to carrying out elevated temperature tests in hot environments or when the impounded fluid is heated. Because of the black colour of most membranes (because of carbon black additives), liners absorb a great deal of heat when exposed to sunlight. Areas of southern Alberta could be a concern in this regard. Typically quoted maximum service temperatures for various lining materials are shown on Table A7.3.

It is apparent from Table A7.3 that cold temperature index values vary significantly according to test type. In general, rubbers and synthetic

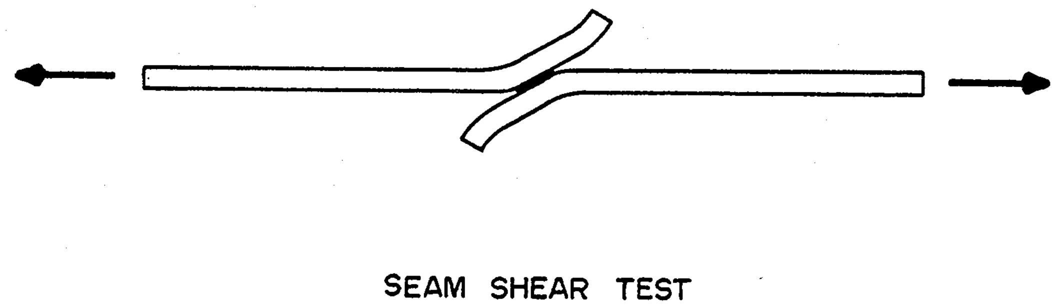


FIG. A7.7 TENSILE TESTING OF SEAMS

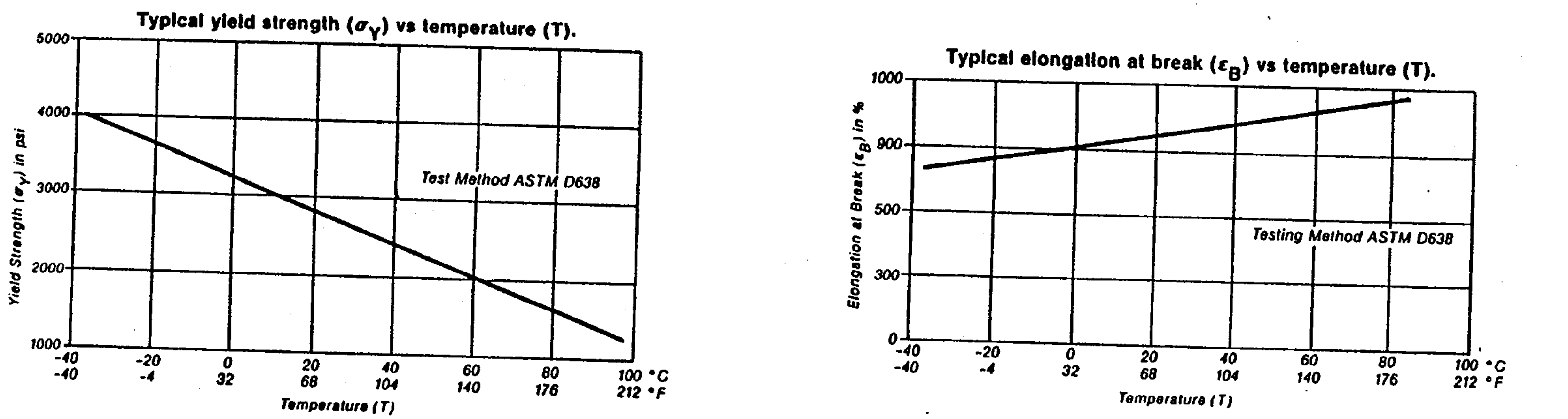


FIG. A7 . 8 EFFECTS OF TEMPERATURE ON HDPE (FROM SCHLEGEL)

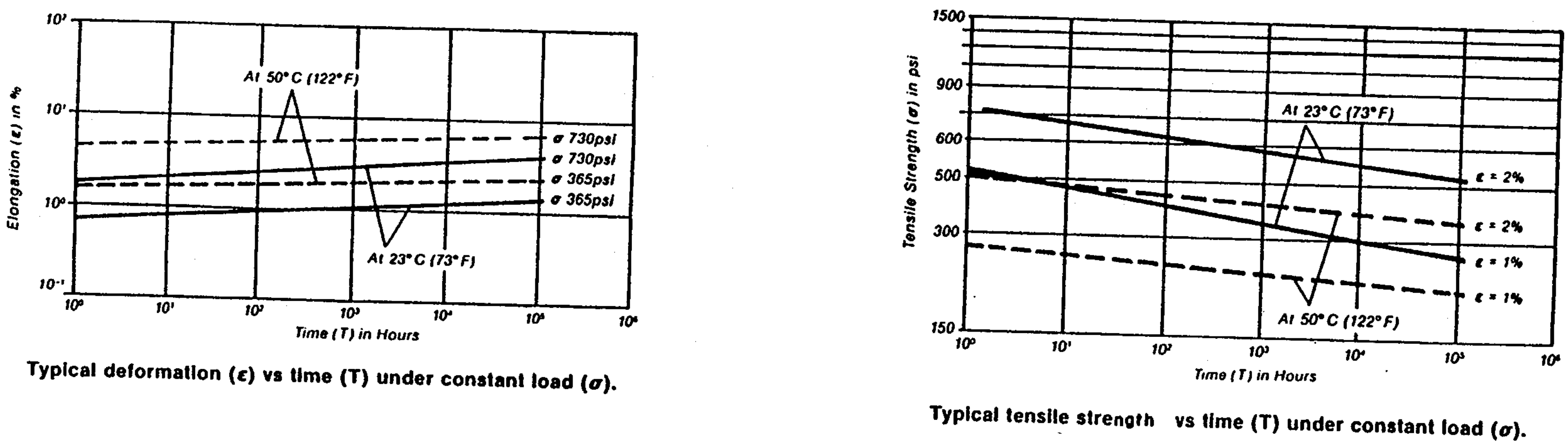


FIG. A7 .9 CREEP OF HDPE (FROM SCHLEGEL) rubbers have superior high and low temperature characteristics. It should be noted that extreme temperatures, although within the manufacturer's specified range, will accelerate ageing processes in the liner material. ASTM D759 and D794 are test standards for carrying out physical property tests at high and low temperatures for short term and long term exposure, respectively. Even short exposure of heat to plastics may result in loss of plasticizer (where applicable) and resultant stiffening and shrinkage of the membrane.

The dimensional stability of plastic membranes is tested by ASTM D1204, which measures the change in linear dimensions of the sheet at elevated temperature for a specified period of time. The NSF recommends minimum values for this test in draft specifications for each type of polymeric membrane.

A7.3.2.8 Creep

As a rule, creep tests are not carried out for membrane liners although Schlegel reports the results of creep test for their HDPE product (Fig. A7.9). Tensile creep tests are standardized in ASTM D2990, and essentially involve measuring the creep under load with time for ASTM D638 procedures. Significant creep can occur in unsupported thermoplastic materials when exposed to the sun, even on gentle slopes.

A7.3.2.9 Coefficient of Friction

Impoundment side slopes of 3.5:1 or flatter are generally specified when soil cover is to be placed on liners in order to reduce the possibility of material sliding or creeping down the liner surface. For specific applications, the coefficient of friction, , between the liner and a soil may be determined by ASTM D1894. The coefficient of friction may vary over the surface of some membranes with additives, and may vary with age, load-rate, and the condition of the surface. If the results are to be used in engineering analysis a number of tests should be done under conditions that model the field situation as closely as possible. Schlegel reports a value for their HDPE sheeting of about 0.35. The U.S. Bureau of Reclamation (1981) reports that Soviet investigators recommend using a value of 0.30 in designs using LDPE. They also report a greater tendency for sliding on LDPE than on PVC.

A7.3.3 Physical Weathering

A7.3.3.1 Ozone

Ozone attacks natural rubbers causing cracking particularly at folds. This was a problem with butyl rubber, but now manufacturers claim that butyl can be compounded to offer excellent ozone resistance (Exxon literature). PE, PVC, Hypalon, and CPE are not affected by ozone. Newer synthetic rubber compounds (elastomers) such as EPDM are compounded to be resistant to ozone. Ozone resistance may be tested by ASTM D1149.

A7.3.3.2 Ultraviolet Radiation

The LDPE and PVC polymers are degraded by exposure to UV radiation (sunlight). Two to three percent carbon black improves the resistance of LDPE and PVC, but both still break down after a few years of exposure. PVC is less susceptible to UV effects, but the effect of the carbon black additive is to increase heat absorption (due to the dark colour) and thus increases volatization of the plasticizer, which causes the membrane to stiffen and shrink after a few years exposure.

Thus, both LDPE and PVC must be protected by an earth cover (usually 30 to 45 cm, depending on construction procedures and the need to protect the liner from mechanical damage during soil cover placement). CPE, Hypalon, and HDPE have superior resistance to UV and heat, and soil covers are not required for this purpose.

The resistance of PVC to loss of plasticizer may be tested by ASTM D1203; standard levels are recommended by the NSF. Although only an index test, the results are useful when compared with field performance. PVC membranes have been installed for many years, and there is good confidence regarding the longevity of typical PVC materials in suitable environments.

A7.3.4 Chemical Weathering

A most important aspect of liner performance is compatability with the impounded fluid. The fluid may cause degradation of the liner by:

a) swelling of the liner
b) extraction of plasticizer
c) breakdown of polymers

All materials swell somewhat, even when in contact with water. Cross-linking or vulcanization reduces the tendency to swell, e.g. vulcanized CPE and Hypalon[1], butyl rubber, EPDM, and neoprene. A high degree of crystallinity also reduces swell potential, e.g. HDPE. Thus HDPE will not dissolve in gasoline although the chemical composition is similar (Haxo 1981).

The shrink/swell behaviour of a liner when immersed in the fluid is a good indication of the suitability of the liner. Stabilized and low levels of shrink or swell indicate compatability as liner material (Haxo 1981).

The best indicator of compatability is long term service in similar facilities. Most data is in the form of relatively short term lab tests provided by manufacturers. The tables in Appendix 14 provide test data for various membrane liner materials in a variety of waste fluids.

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1 The thermoplastic versions of CPE and Hypalon are more common.

Most common polymeric liners should be compatible with municipal wastewater. If the wastewater is outside typical ranges, then the compatibility of the liner material and wastewater should be investigated.

Seams are vulnerable to chemical attack. Tests reported by Matrecon for landfill leachate indicate that solvent and adhesive seams generally lose strength. Extrusion weld seams (HDPE) are the safest, since the seam is composed of the parent liner material alone. Other materials can also be heat-welded under proper conditions, resulting in a seam superior to solvents or adhesives (mastics and tapes).

A7.4 DESIGN AND INSTALLATION OF FLEXIBLE POLYMERIC MEMBRANE LINERS

A7.4.1 General

The design of flexible polymeric membrane liners should include the following considerations:

- liner bedding requirements
- underdrainage and gas venting
- need for reinforced materials on sideslopes
- anchoring methods
- seals at structures, e.g., transfer pipe outlets
- requirements for earth cover
- need for access or cleaning out of sludge

A typical schematic section of a liner requiring an earth cover is shown on Fig. A7.10.

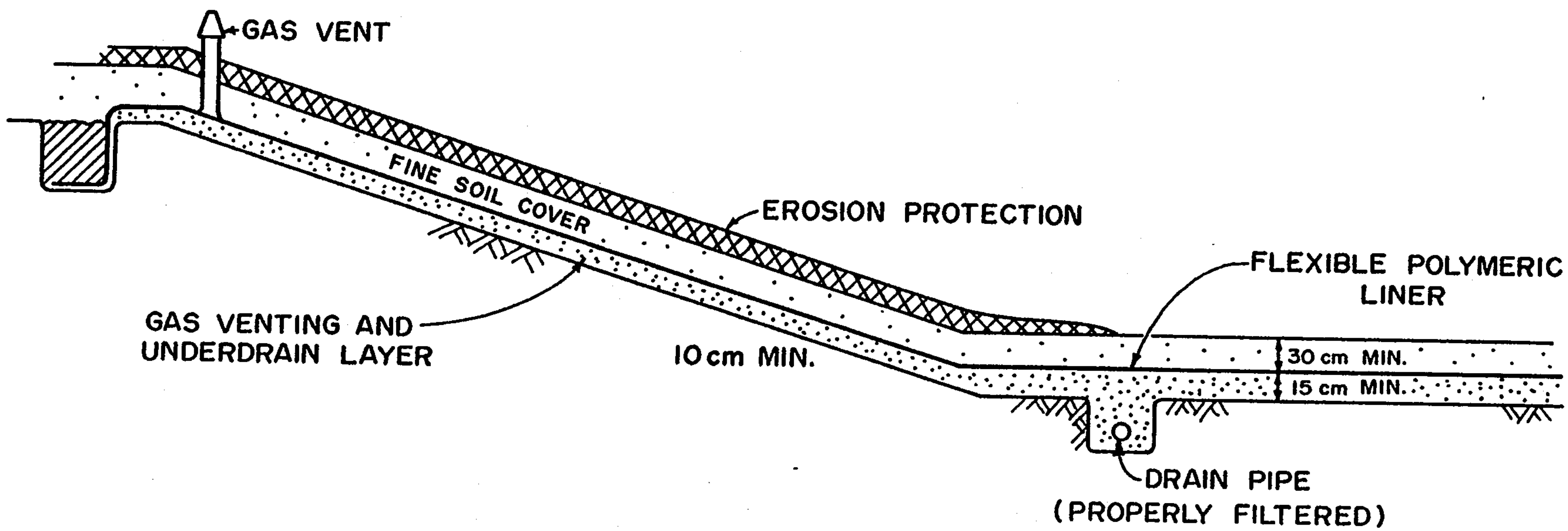


FIG. A7.10 SCHEMATIC CROSS-SECTION OF FLEXIBLE POLYMERIC MEMBRANE INSTALLATION

A7.4.2 Subgrade and Bedding Surfaces

The subgrade surface should be stable and compacted to at least 90% of the maximum dry density (ASTM D698). The underlying foundation soils should be firm enough to support the net increase in surcharge due to the pond fluid. The subgrade surface should be smooth and uniform and parallel to the final pond grade.

Trenches for underdrain collection pipes should be excavated at the time of subgrade preparation. The trench inverts should be relatively smooth and uniform, and at a grade sufficient to promote drainage to the collection sump.

The surface on which the liner is placed is called the liner bedding surface. This may be the subgrade surface in sandy soils, or the underdrain surface. The bedding surface should be free of rocks, roots, debris, stake holes, cracks, and any rapid changes in elevation. Recommendations regarding maximum gravel size vary. Matrecon (1980) indicates a maximum rounded particle size of 19 mm, while Schlegel Lining Technology Inc. requires a maximum rounded particle size of about 7 mm, and a maximum particle size of 3 mm in crushed rock material. Fine sand is frequently recommended as a bedding material. In general, avoidance of gravel sizes and particularly angular particles will facilitate the installation procedure and reduce the potential for punctures.

In some cases heavy nonwoven geotextiles may be used to protect the liner from gravel sizes in the subgrade or underdrain layer. Nonwoven geotextiles will also provide gas venting and underdrainage, with in-plane hydraulic conductivity values in the order of 10^{-2} to 10^{-4} m/s. The transmissivity of the geotextile is low, however, because of the thinness of the compressed fabric. If the geotextile is placed on fine-grained soils, silt and clay particles may reduce the permeability of the material.

The bedding layer should be reasonably compact to support the liner during seaming. A narrow strip of geotextile placed below each seam may provide increased support for heavy welding tools.

Weed growth can cause serious damage to a polymeric liner. If dormant seeds are present in the subgrade soils, weeds could grow and penetrate the lining material. Since most lining materials are black (or dark) in colour, they absorb heat, providing incubation for underlying seeds. Once weed growth begins and the liner is penetrated, the holes through which the weeds grow become enlarged, allowing more sunlight and moisture to reach the seeds below the liner (Kays 1977). In liner systems where an earth cover is used, the problem of weed growth is compounded. The roots of weeds growing on the earth cover may penetrate the liner from above. The best method to combat weed growth is to treat the subgrade with soil sterilants (weed-killing chemicals) just before liner installation. Effective maintenance after liner installation is also important.

A7.4.3 Gas Venting and Underdrainage

All flexible membrane liners should be bedded on a relatively permeable layer, such as clean sand or open-graded asphalt, to provide a venting medium for gas accumulations. Gas may originate from the groundwater or from decomposing organics (native or in the wastewater).

The gas venting medium should extend up the sideslopes and vents should be provided in the liner near the crest at about 15 m intervals (Kays 1977). An earth cover of 30 cm over the liner will help ballast the membrane against gas uplift, which is a buoyancy phenomenon (i.e, the pond liquid does not serve as a ballast once gas pressures equal the pressure of the pond contents, but instead is a contributing mechanism in gas uplift). Because flat bottoms are required in most stabilization ponds, the invert cannot be graded to promote gas migration. Thus, extra care should be taken to provide a permeable venting medium below the liner.

The gas venting medium may also serve as an underdrain and remove groundwater that seeps under the liner; this prevents hydrostatic uplift at low pond levels. Wastewater seepage can be recycled back into the pond.

The underdrain thickness, spacing of drainage pipes, etc., depend on site conditions and the purposes of the underdrain; e.g., control of precipitation in the pond prior to lining, control of seepage through the liner, control of hydrostatic pressures due to groundwater seepage. Cedergren (1976) provides design charts for drainage layers. The underdrain material and drainage pipes must have adequate filter protection to prevent migration of fines and loss of subgrade support, and clogging of sand and pipes. The underdrain system should terminate in a sump which is dewatered as required e.g., by a submersible or suction pump operated by a float control.

A7.4.4 Installation of Polymeric Liners

A7.4.4.1 Storage at Site

Polymeric liners should be stored out of direct sunlight and hot areas prior to installation, as heat may cause "blocking" of the liner material, i.e. the liner may become cemented together while in a rolled form. Blocking can cause tearing of the liner as it is unrolled. Affected liner types include all members of the EPDM family, Hypalon, PVC, neoprene, butyl rubbers and LDPE. In general, all polymeric liners should be stored away from sunlight and heat.

Vandalism is another source of potential damage to polymeric liners. Since single rolls of a polymer liner may cost nearly $100,000, a security system to protect the liners is often warranted.

A7.4.4.2 Handling of the Liner Materials

Proper equipment should be contracted to transport liner rolls between the storage area and installation site. Heavy equipment is necessary in most cases since liners in rolled form or accordian-folded packages may weigh as much as 4,500 kg.

If folded liners are not shipped and delivered on wooden pallets, pallets should be provided. Pallets make liner movement much easier if a fork-lift is being used. Rolled liners may have slings or straps near the centre of the roll for easier lifting.

Machines that can be used in other phases of liner installation should be considered. For example, front-end loaders may be more useful overall than fork-lifts or cranes.

A7.4.4.3 Work Crew

The working crew and equipment required for liner installation must be secured before the operation begins. The crew size depends on the required speed of the installation as well as the difficulty of placement. Arrangements must be made with the liner supplier and/or contractor regarding the numbers of unskilled labourers that are to be supplied. The number of labourers may range from 6 to 30, depending on the size of the operation. The supplier should have an experienced foreman on site to oversee the liner installation. The customer and supplier/installer should have worked out all of the arrangements regarding crew, equipment, and services to be supplied by each party prior to the commencement of construction.

A7.4.4.4 Laying of the Liner

Attention should be given to the direction in which the liner must be unrolled or unfolded. These directions are usually clearly printed on the liner container. Moving or relocating a sheet of unrolled liner can be very difficult and time consuming due to a sheet's large size and weight. Following the shippers' directions will also ensure that the correct side of the liner is exposed for seaming. It is not advisable to roll out more sheets at once than can be seemed in a normal work period.

The liner is usually unfolded lengthwise, often with the use of a fork lift, in the direction indicated on the construction drawings. It is then unfolded in the width direction and "spotted" in its proper position. The liner sheet should be pulled to a relatively smooth state with enough slack left to compensate for possible shrinkage due to temperature change. The foreman should supervise the spotting of the liner carefully and determine when it is in the correct position.

Once the first two sheets are properly spotted, sandbags are placed along the edges to be seamed, and seaming may begin.

A7.4.4.5 Weather Conditions

(a) Wind

Wind conditions can cause major problems during the installation of a liner. If wind gets under a sheet or liner material and a "sail" effect occurs, much damage can be done to the surrounding facilities and workmen, and to the liner itself. For this reason, work should be halted when wind speeds exceed 32 km/h (Kays 1977).

Sand-bagging and edge folding are only partially effective in preventing wind damage. An earth cover on part of a liner sheet will help combat the wind. Light winds may stir up dust which may impair the quality of the seams.

(b) Temperature

Temperature also has an effect on liner installation. With regard to seaming, most adhesive systems work best if the temperature of the liner itself is greater than 15°C (Matrecon 1980). If ambient temperatures are lower than 15°C, heat guns may be needed.

The most desirable temperature range for liner installation is 15°C to 25°C (Nilos Canada Ltd., personal communication). Problems could arise in all stages of installation if temperatures are above or below this range.

(c) Precipitation

Seaming during periods of precipitation should be avoided although seaming can continue if protective structures are provided.

A7.4.4.6 Seaming

(a) General

Some general procedures apply to most seaming methods and are summarized below. Seaming is probably the most important aspect of liner placement, and should commence after the first two liner sheets have been spotted and held down with sandbags. Seaming should begin at the centre of these two sections and run in both directions to the end of each section. This allows two field crews to operate simultaneously.

The only seaming method used should be that recommended by the manufacturer of the liner. If no method can be recommended for a particular liner material, it should not be used.

All surfaces which are to be bonded must be clean and dry. Failure to observe this could result in a weak seam, small paths in which liquid can escape, or complete seam failure. Some form of solvent is usually used to clean surfaces.

The edges of liners which are to be seamed should not be stretched because the tension that is created may cause buckling or separation of the seam.

The field crew must be careful not to allow any wrinkles to develop in the seams. If any wrinkles, tears, or other forms of defects occur during the seaming operation, these must be repaired immediately using the proper methods. Any patches used must have rounded edges.

(b) Contact Cement and Bodied Solvents

The application of contact cement adhesives and bodied solvents in seaming liners is fairly similar, and the procedures for both are discussed below. A contact cement is an adhesive material that acts as a bonding agent between the liner sheets. A bodied solvent, however, is a solvent with dissolved polymer, usually the same polymer as in the membranes. The solvent evaporates, leaving a bond created by the polymer molecules. Thus, a bodied solvent seam is usually superior to a contact cement seam.

i) Overlap Of Liner Sheets

After the sheeting is properly positioned, the edges of the two sheets are overlapped. The width of the overlapping section will vary, although the width suggested by the manufacturer should always be followed. An example of overlap widths are a 15 cm overlap for Hypalon with at least a 7.5 cm bonded overlap for supported material and a 10 cm minimum for unsupported material (DuPont, personal communication). A loose edge on the underside of the sheet is acceptable, but there should be no loose edge on the top side of the seam.

ii) Cleaning of Surfaces

The surfaces to be seamed must be cleaned of dust, dirt, oil, etc. In the cases of Hypalon and EPDM liner materials, this is usually done with trichloroethylene (Matrecon 1980). This solvent also helps to remove a surface cure which forms on these types of liners.

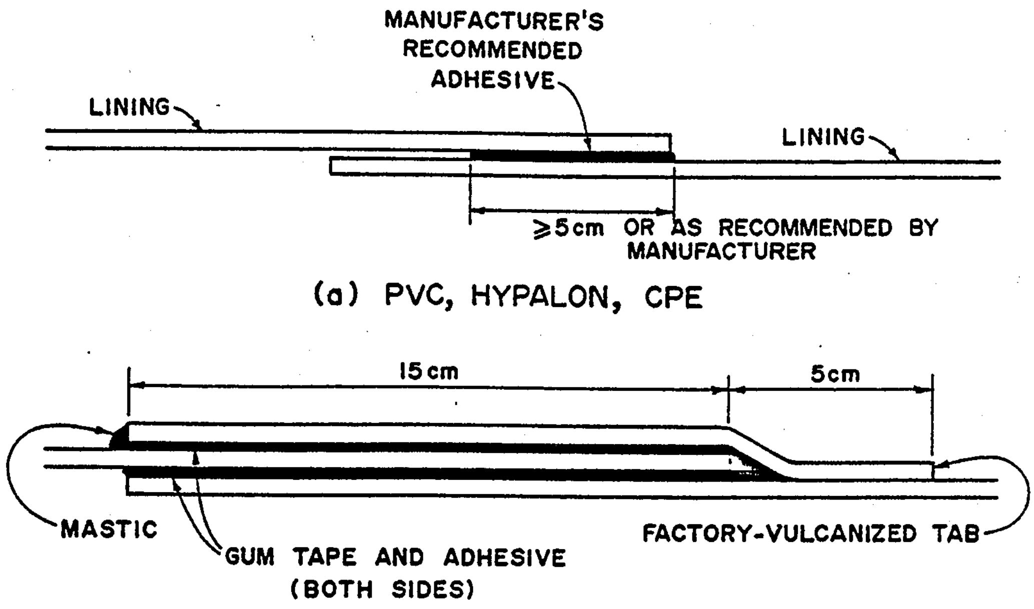
iii) Application of Adhesive

An adhesive may be applied by brush, spray, or extrusion through a nozzle. If the temperature of the sheet is not above 15°C, hot air guns should be used to heat the sheet before the adhesive is applied. The two surfaces are lapped together immediately after application.

The adhesive method of seaming is widely used because of its compatibility with a wide variety of liner materials. The types of liner materials which may be seamed by the solvent or adhesive methods include LDPE, PVC, Hypalon, CPE, EPDM, Neoprene and Butyl. Of these, PVC, Hypalon, and CPE are usually seamed by bodied solvents (Staff Industries Inc. (1980); DuPont, personal communication) with a simple overlapping seam (see Figure A7.11a).

A gum-tape-adhesive has been used for seaming butyl rubber, EPDM, Neoprene, and LDPE. Adhesive is applied to the surface of one of the sheets and gum tape is applied to the adhesive. The matching piece of liner is treated with adhesive and joined to the tape. Vulcanizing cement is usually used as the adhesive. EPDM, neoprene, and buyl rubber usually employ a tongue and groove-type seam (see Figure A7.11b). It is believed that neoprene and EPDM are still seamed with this method (Matrecon 1980), whereas butyl rubber is now commonly heat-sealed.

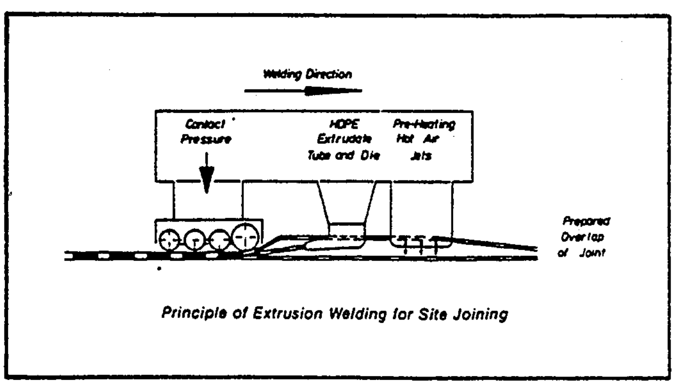
Joints seamed with adhesives must be rolled using a small hand-held hard-faced roller. The rolling should be perpendicular to the edge of the sheet. A wooden board provides a rigid base for the rolling operation.



(a) PVC, HYPALON, CPE

(b) EPDM, BUTYL RUBBER, NEOPRENE

FIG. A7.11 TYPICAL FIELD JOINT DETAILS (ELASTOMERIC AND PLASTIC MEMBRANES) (AFTER KAYS, 1977)



Principle of Extrusion Welding for Site Joining

FIG. A7.12 EXTRUSION WELDING OF HDPE (FROM SCHLEGEL LINING TECHNOLOGY LITERATURE)

The time delay between adhesive application and lapping is one difference between contact adhesives and bodied solvents. Contact adhesives require a certain tackiness before the adhesive system is applied.

(c) Heat-Sealing

The heat sealing process is not well documented in the literature. In this process the liner is placed and overlapped according to the manufacturer's specifications. A form of electronic welder is then used to melt the sheets together.

Materials which can be joined by heat-sealing include butyl rubber, which employs a hot vulcanizing method (Butyl Products, Ltd., personal communication), and Elasticized Polyolefin 3110, using a hand-held welding machine (Kirby 1979). HDPE is also known to be seamed in this manner by at least one installer (Columbia Reservoirs, personal communication). Many other liner materials may be seamed with heat welding. However, bodied solvents or contact adhesives are more widely used.

(d) Extrusion Welding

Extrusion welding is unique to HDPE liners at present (Schlegel, personal communication). A specialized welder heats the liner, injects a molten ribbon of HDPE between the overlaps, and then compresses the seal (Fig. A7.12). Before welding, the sheets are preheated with hot air jets. The temperature of the air is varied according to ambient temperature conditions. The HDPE extrusion is forced through the two sheet ends at a temperature of 235°C. Finally, pressure is applied to the weld seam by means of pressure rollers attached to the roller on a suspension system.

The average welding speed is about 1.2 m/min. The sheet overlap may be from 20 to 40 cm, but only a 5 cm wide weld seam is necessary for a satisfactory joint. Three models of this type of welder have been developed for different areas and degrees of difficulty of welding (Schlegel, personal communication).

A slightly different extrusion welding method is used by Gundle (Synflex, personal communication). The welder has two agitating tips which heat the liner material itself and mix the extrudent and liner material. The extrudent is applied at 270°C and is the same material as the liner. The overlap width is at least 10 cm.

A7.4.4.7 Testing of Seams

(a) Visual Inspection

The seaming operation should be observed by an experienced field supervisor, and any necessary changes made while seaming is still taking place. All seams should be visually inspected.

(b) Testing of Large Seam Areas

i) Ultrasonic Testing:

Several methods are used for the testing of large seam areas. One of these is ultrasonic testing in which a testing head is run over the seam. The response of the reflected waves gives an indication of the quality of the seam. The ultrasonic method is essentially a measurement of thickness. If the seam is homogeneous, the transmitted wave will return undisturbed. If a non-homogeniety exists, the wave will be reflected at the interface and the fault will appear on a monitor screen (Schlegel). This method is easily performed and is quite sensitive to imperfections in the seam.

ii) Suction Test

Another method of testing the continuity of seams is the suction test. The seam is put under suction by a clear plastic suction cup which is attached to a vacuum pump. A foaming agent is used to indicate the exact location of leaks (Schegel). This method is not well documented in the literature, and is believed to be much slower than the ultrasonic method. The extent of its use is not known.

iii) Pressure Techniques

Double welded seams may be tested by pressurizing the channel between the welds (FIG. A7.13). In the procedure developed by Sarnafil Inc., the channel is pressurized to 207 kPa for 15 minutes. If the

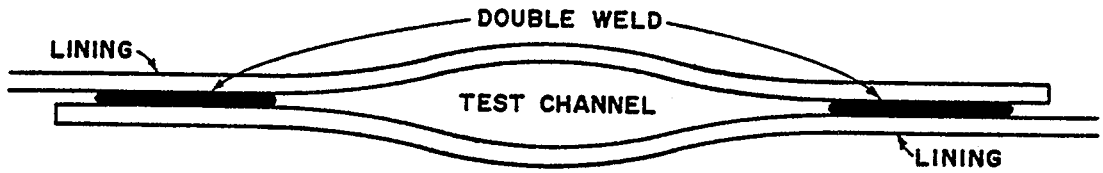


FIG. A7.13 DOUBLE WELD (FROM SARNAFIL®, INC., LITERATURE) pressure drops more than 28 kPa during this period then the seam is not accepted (Sarnafil Inc., personal communication).

iv) Spark Test

A spark test method is also used to test the integrity of seams. A 30 kv electric current is applied to the seam. Any leakage to ground as caused by voids, sinkholes, etc., can be detected (Sarnafil Inc., personal communication). This method is also not well documented, and specific details on its operation are not available.

v) Air Lance Test

In air the lance air is directed through a 4.8 mm nozzle at pressure of 345 kPa to detect tunnels or fishmouths in a seam. The nozzle should be held no more than 15 cm from the seam edge (Matrecon 1980).

(c) Destructive Tests on Small Seam Areas

Destructive tests can also be used to test field seams, where a small sections of a seam are cut out from the main liner for use as a test sample. The two most common tests are the high speed tensile test and the peel tensile test. The location from where the sample was taken is noted, and the hole is patched by the appropriate method.

A7.4.5 Anchoring of Polymeric Liners

Polymeric liners may be anchored by two different procedures. The first, and most common, of these is the trench and backfill method. Figure A7.14a shows a typical trench and backfill design. The trench is usually constructed using a "ditch witch" , backhoe or by using a bulldozer blade tilted on an angle. As the soil is excavated it is spread evenly on the opposite side of the trench from the liner in order to make unrolling and spotting of the liner easier. Since the liner is temporarily secured by sandbags during seaming, the trench should not be backfilled until the seaming is completed for a particular panel. The

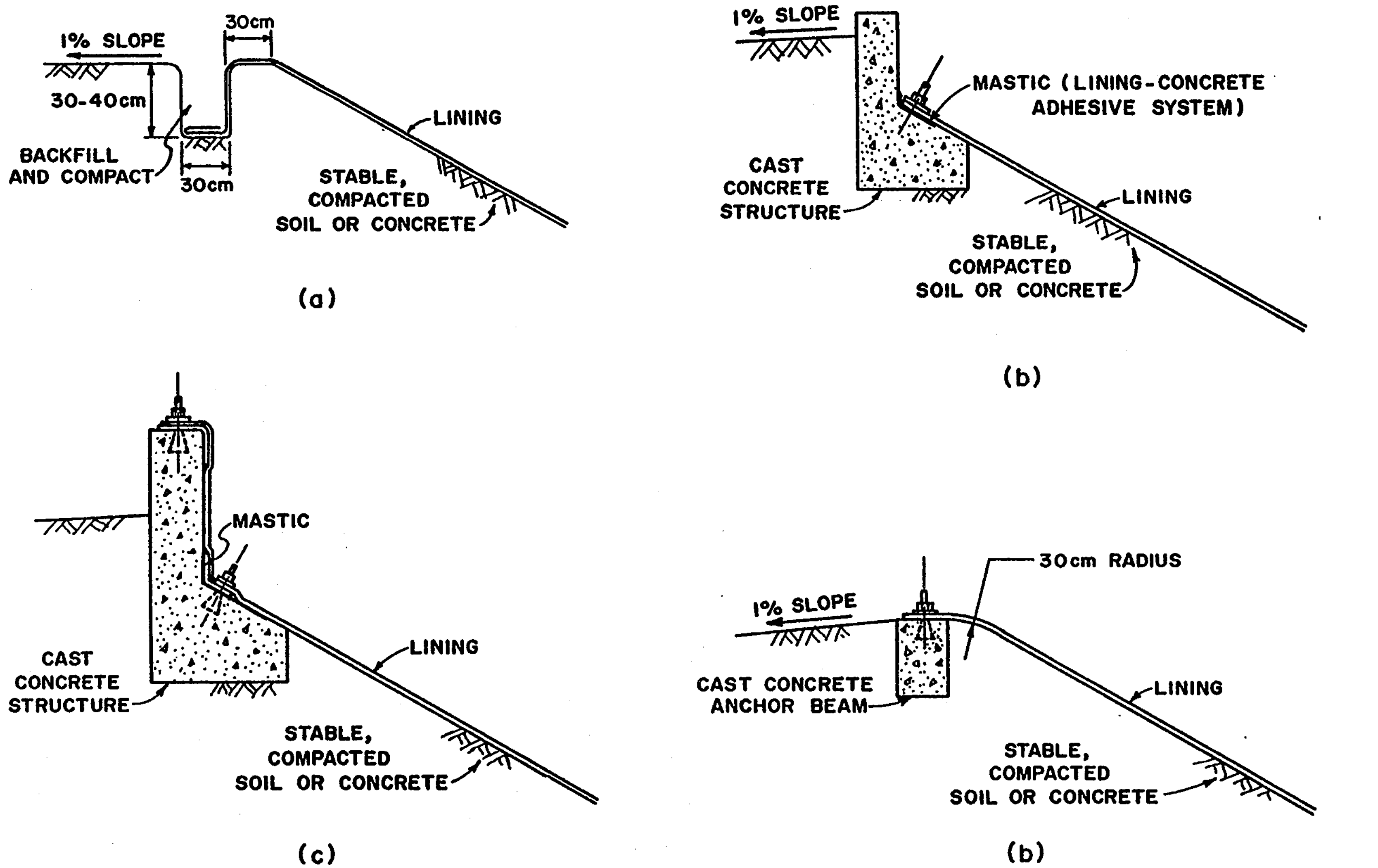


FIG. A7.14 CONCEPTUAL DRAWINGS OF LINING ANCHOR SCHEMES (AFTER KAYS, 1977) end of the liner is usually doubled, as shown in Figure A7.14a, with no type of reinforcement being required.

The second type of anchoring technique is to mechanically secure the liner to a concrete structure along the top of the berm. There are many different methods and designs for this concrete structure, as shown in Figures A7.14b, c, d (Kays 1977). The most important design and construction considerations of the concrete-type anchoring system include the following:

i) The concrete area which comes into contact with the liner should be smooth and free of all curing compounds.

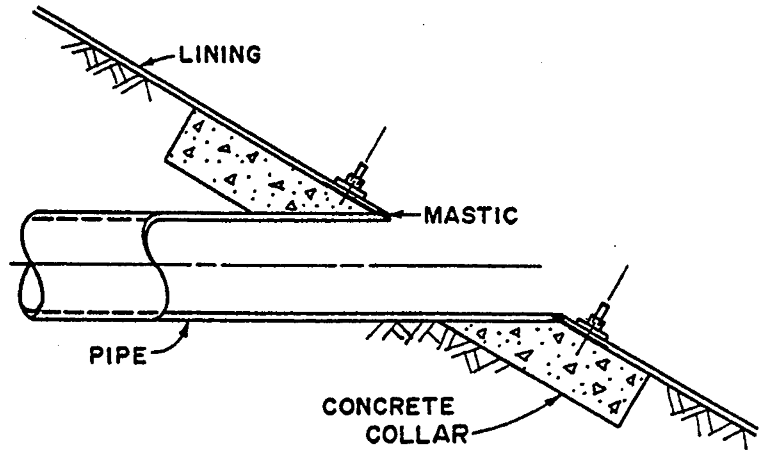
ii) Anchor bolts should be placed not more than 30 cm apart.

iii) Concrete adhesive and a chafer strip of liner material are usually incorporated into the design.

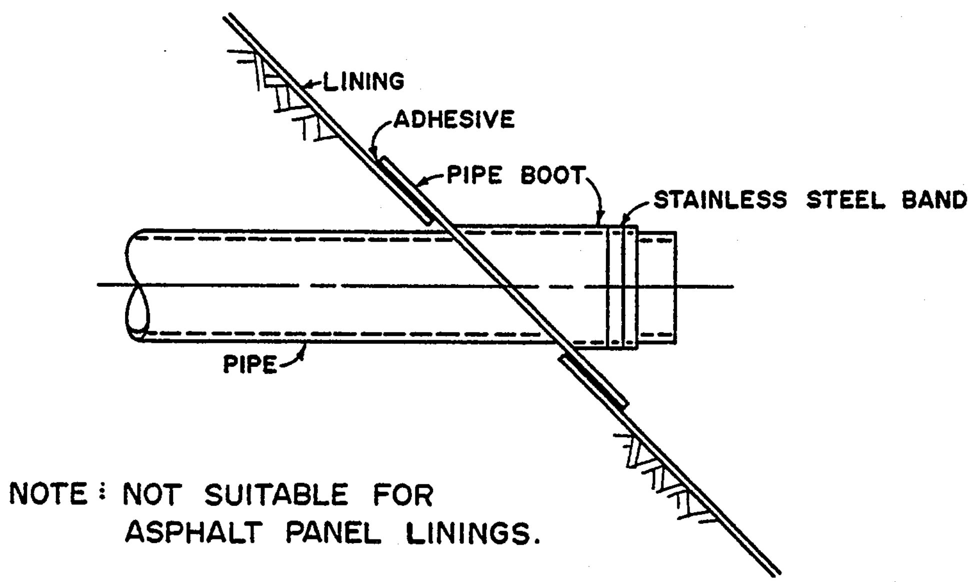
A7.4.6 Sealing Projections

There are two generally accepted methods used to seal projections through a membrane liner (e.g. inlet and outlet pipes). The first method involves the sealing of a projection in the plane of the lining, as shown in Figure A7.15a. This type of projection occurs when the pipe enters through a structure such as a concrete pad. Mechanical anchors are not usually used unless the projection occurs on a slope (Kays 1977). The appropriate adhesives, mastics, mechanical anchors, etc., should be recommended by the liner manufacturer. Quite often the manufacturer will supply detailed design drawings and may also have some special techniques to seal projections.

The second method of sealing projections involves the use of boot or shrouds, which are usually manufactured by the same company as the liner. These boots can also be manufactured in the field in emergency

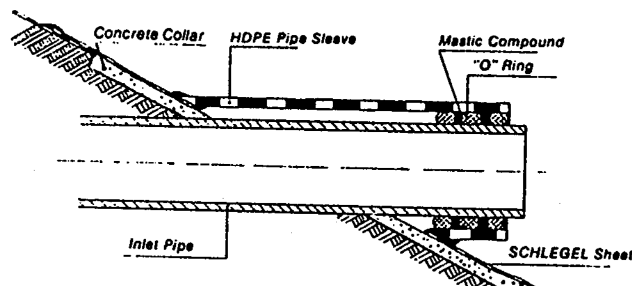


a) SEAL AT PIPES THROUGH SLOPES (AFTER KAYS, 1977)



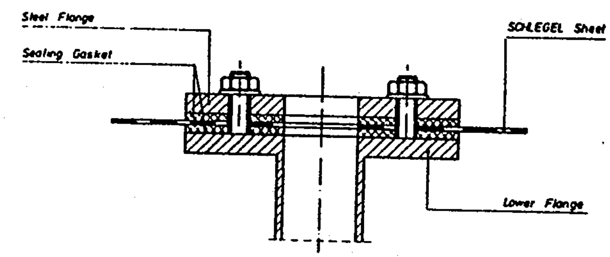
b) PIPE BOOT DETAIL (AFTER KAYS, 1977)

FIG. A7.15 PIPE SEALS



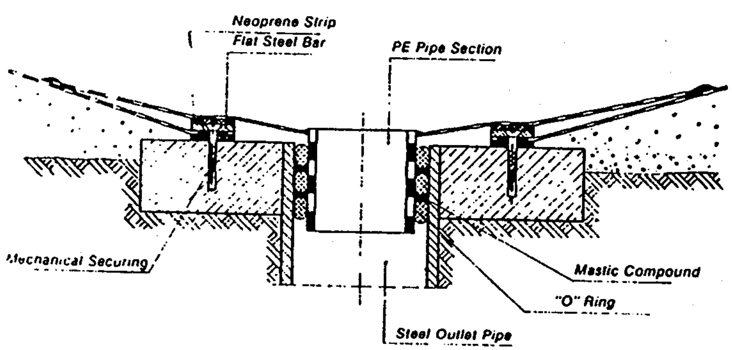
Connection of Concrete or Steel Inlet Pipe to SCHLEGEL Sheet

Prefabricated sleeves are individually manufactured for each pipe protrusion and sealed with compression "O" rings.



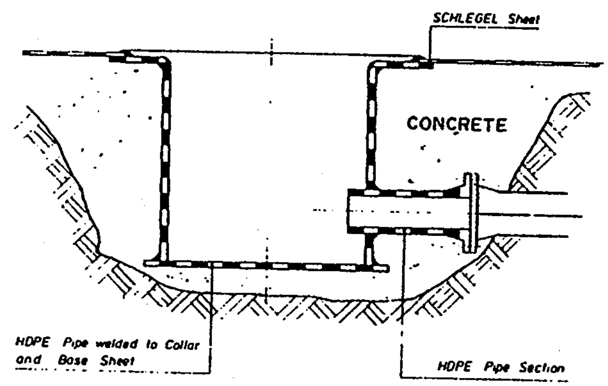
Flange Connection of SCHLEGEL Sheet to Pipe Section

Entrapment of SCHLEGEL® sheet between the flanges of a flush flange fitting with a compression gasket effects the seal.



Connection of SCHLEGEL® Sheet to Base Outlet Pipe

The size and number of "O" rings or foam sealants may vary for new or renovated flanges.



Connection of SCHLEGEL Sheet to SCHLEGEL Prefabricated HDPE Sump

Schlegel can provide HDPE sumps, prefabricated to detailed specifications, for applications where concrete sumps would be subject to chemical attack.

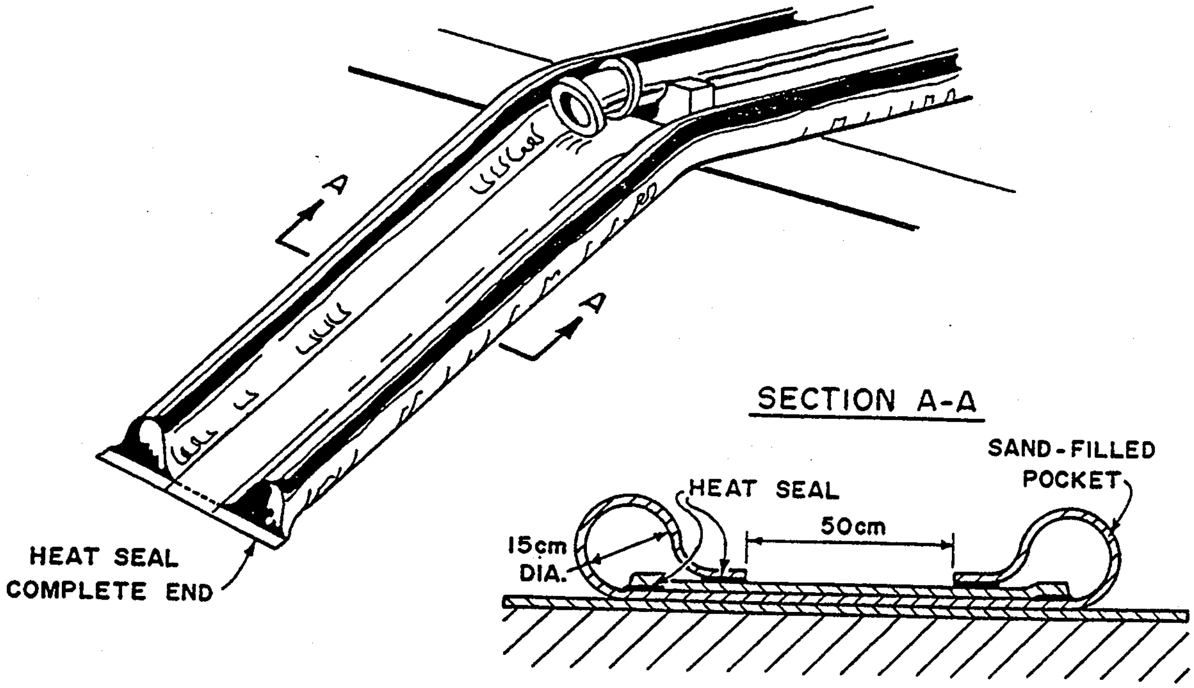
FIG. A7.16 ALTERNATIVE METHODS FOR SEALING HDPE LINERS TO PIPES (FROM SCHLEGEL LITERATURE) situations by experienced personnel. An example of such an installment is shown in Figure A7.15b. The shroud is usually made from unreinforced liner material so that it can be stretched over the pipe to ensure a tight fit. An adhesive is used to secure the pipe to the shroud, and an "O-ring", usually stainless steel, secures the end of the boot. Joining methods used by Schlegel for HDPE liners are shown in Figure A7.16. The liner supplier should recommend the proper sealing techniques, adhesives, and materials to be used in the sealing of any projection. Supplier/installer experience and knowledge is very important in the sealing of projections. Inlet and outlet pipes should be the "over the beam" type, if possible. This facilitates maintenance and repair, and also eliminates the need for sealing projections through the liner. A sluice-type trough is often used to protect the main part of the liner when pipes come over the berm. This type of trough, which is illustrated in Figure A7.17a is usually made from the liner material.

A7.4.7 Concrete Structures on Linings

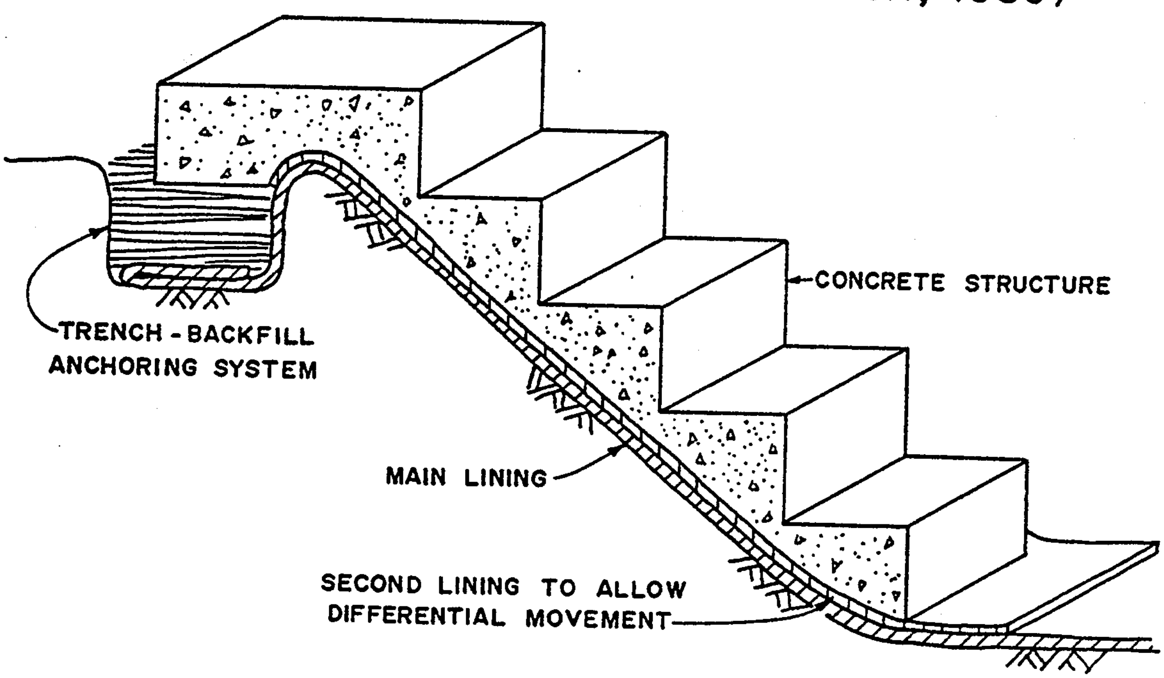
Concrete structures are often necessary in lined facilites for aerator pads, splash pads, and concrete ramps or steps. The common practice for concrete installations is to place another layer of liner material between the main layer and the concrete pad. This allows variable movement between different materials without damage to the liner. An example of a concrete structure placed on a liner is shown in Figure A7.17b.

A7.4.8 Placement of Soil Covers

Earth covers must be placed on standard PVC liners and any other materials subject to rapid weathering when exposed to air or sunlight. Earth covers are recommended on the sideslopes of flexible membranes less than 40 mils (1 mm) in thickness in order to protect against mechanical damage and weathering. The maximum side slope angle that will support an earth cover over a smooth polymeric liner is 3:1



a) SLUICE-TYPE TROUGH CONSTRUCTED OF LINER MATERIAL (AFTER MATRECON, 1980)



b) USE OF SECOND LINER TO SEPARATE MAIN LINING AND CONCRETE STRUCTURE

FIG. A7.17 LINER PROTECTION PADS

(Matrecon 1980). The earth cover should be at least 30 cm in thickness and should be placed in one lift to provide adequate protection of the liner from construction equipment. The grain size restrictions for cover material are usually less strict than for bedding material although angular gravel and other sharp objects must be avoided. Native soil should be used whenever possible to reduce material costs. Gravel and cobble sizes, riprap, etc., required for erosion protection may be placed on top of the earth cover.

The soil cover for side slopes should be spread in the up-slope direction. Trucking routes and soil placement directions should be specified by the engineer, after consideration is given to the liner subgrade and the location of pipe works. Restrictions on speed may also be necessary.

Placement operations should avoid repeated trafficking of any area of the soil cover, unless a temporary haul road is constructed using 0.6 m or more of soil cover. Tracked vehicles exert lower pressures on the soil and liner, but turning of tracked vehicles must be avoided.

APPENDIX 8
MISCELLANEOUS LINING SYSTEMS

A8.1 SOIL SEALANTS

A8.1.1 General

The permeability of soils which line waste impoundments may be reduced under certain conditions by the application of various chemicals that react to form a less permeable membrane. These sealants partially fill the soil voids thus reducing the effective porosity of the soil and the rate of seepage. Requirements of chemical sealing agents for soil sealant purposes are suggested by Stewart (1978) as follows:

a) The sealant should be non-toxic to humans, animals and plants.
b) The sealant should be capable of nonrestrictive application under a broad range of soil compositions in static or dynamic conditions.
c) The sealant should be resistant to damage by animals, erosion, mechanical equipment and cleaning processes, and hydraulic pressures of up to 138 KPa.
d) The seal should be durable and resistant to deterioration by climatic conditions such as freeze-thaw and wet-dry cycles, sunlight, soil micro-organisms, re-emulsification, chemical change, and reverse hydraulic flow.
e) The sealant should be capable of resealing existing liners.
f) The liner system should be cost-efficient.

A8.1.2 Types of Soil Sealants

There is no single chemical which will seal all soils effectively under all conditions. The main types of soil sealants are described below.

A8.1.2 Types of Soil Sealants

There is no single chemical which will seal all soils effectively under all conditions. The main types of soil sealants are described below.

a) **Monovalent cationic salts** (Matrecon 1980) are mainly sodium salts which chemically reduce the effective porosity of a clay soil by replacing exchangeable cations in the clay structure with cations of higher molecular weight and/or space requirements. This causes the clay structure to become more easily dispersed and the clay particles to expand into the voids, thus reducing the porosity and permeability of the soil. The most commonly used salts are sodium carbonates which when applied at 2 lb/sq.yd. (0.9 kN/m^2) can reduce seepage through some clay liners by up to 90 per cent (Matrecon 1980). Sodium silicates have been found to be compatible with sulphuric wastes if mixed with sulphuric acid when applied to the soil. Sodium pyrophosphates and polyphosphates have been used with some success in clay soil (Kays 1977).

b) **Polymeric soil sealants** are a relatively new development (Kays 1977). They are generally mixtures of swellable linear polymers and crosslinked polymers of similar molecular weight. Polymeric sealants, when mixed with low pH water-acid solutions, can penetrate a soil liner effectively, filling the soil voids and reducing its porosity. The mechanism employed by polymeric soil sealants is outlined below (Stewart 1978). A polymer-water-acid solution is spray-applied to the soil surface as a low-viscosity slurry. Dry blending and powdering techniques may also be described, as summarized in a later Section. The low-pH condition helps the solution to penetrate the surface of the soil. With exposure to water the polymers swell in the soil voids and "lock" in place. The linear polymer then attaches to adjoining clay particles to "complete the formation of a stable, impermeable membrane" (Kays 1978).

There are several types of polymeric sealants. Sprayable liquid vinyl polymers and polyacrylamides are commonly used. Latex soil sealants were found, in experiments by Uniroyal, to reduce the permeability of soil to water, but were subject to damage by microbiological attack, frost, and vegetation (Matrecon 1980). Gulf South Research Institute tested two polymers (Matrecon 1980). A styrene polymer with a high concentration of wetting agents was sprayed and allowed to enter into the soil to produce a fairly resilient film with good soil sealing capabilities. An off-grade polyvinylidene chloride film was reported to provide a good seal at high pressures.

c)**Bentonite-polymer mixtures** are also used as soil sealants (Stewart 1978). The bentonite replaces the cross-linked polymers in the mixture. This arrangement does not give as effective performance as polymer mixtures, but is much less costly and is good as a structural improvement to existing impoundments. The admix form of bentonite liners is discussed in Appendix 5.

A8.1.3 Methods of Application of Soil Sealants

Soil sealants are generally sprayed onto a soil surface as a low-viscosity, slightly acidic slurry formed by carefully blending dry powder and fresh water. These slurries have been successfully applied to the soil using water-hauling trucks equipped with centrifugal pumps, hoses, and adjustable fire nozzles.

Dry-blending is another term for admixing, e.g. bentonite admix liners.

Dusting with powdered polymers may be accomplished with any equipment suitable for distributing a powder.

A8.1.4 Limitations of Soil Sealants

The performance of soil sealants is affected by a number of factors, including wetting-drying and freeze-thaw cycles, reactions with pond wastes, leaching by waste fluids, and the application rate of the sealant (Stewart 1978).

The degree of soil compaction before sealant application (or during the application of dry-blended sealants) is an important factor. Seepage rates may be reduced by an order of magnitude if sufficient compaction is achieved.

Exposure to the same acidic conditions that allow the slurry form of polymeric sealants to be used will decrease the effectiveness of the seal once it is in place and exposed to higher pH conditions. Exposure to salts, especially monovalent cations, causes a reverse in polymer volume. Some salts that retard the hydrolysis of polymers (i.e. allow the powdered form to be used) decrease the efficiency of seals which have been established against fresh water. For these reasons, the choice of a soil sealant should be made only after consideration of its compatibility with both the soil and waste fluid.

A8.2 WATERBORNE DISPERSIONS

A8.2.1 General

Waterborne dispersions are substances which are dumped into an existing pond or reservoir (filled) and settle to the bottom. These sealants act by increasing the ionic attraction of soil particles to water, thus decreasing the void space in the soil. Waterborne dispersions are not generally effective as a primary lining system, but they may act as a secondary liner treatment or maintenance measure (Morrison et al 1971). They have the advantage of low cost and the ability to be placed without the removal of the water or waste liquid from the pond/reservoir.

A8.2.2 Examples of Waterborne Dispersion

Soil-Saver 13 (SS-13) is a mixture of oil-soluble resinous polymers in a diesel fuel carrier. SS-13 is mixed at 1 part per 1000 with water in a reservoir. The chemical migrates to the bottom of the pond and accomplishes most of its sealing action within 48 hours. Chevron produces a soil sealant named "Seelo W" which forms a thin wax- like membrane below the surface of compatible materials. This protects the membrane from animals and mechanical cleaning equipment (Kays 1977).

A8.2.3 Performance of Waterborne Dispersions

Most waterborne dispersions have the disadvantage of being toxic. All fish life in the pond is killed during application. After installation, however, it is non-toxic and safe for most life-forms including humans. The water should be treated before drinking.

The effectiveness of waterborne dispersions is heavily dependent on the chemical nature of the retained water. For example, waterborne treatments are unsuitable for brine ponds or where salt concentrations are greater than 400 ppm. Before application, the water should be chemically studied to determine the effectiveness of waterborne dispersions. Also, treatment should not be performed by anyone who is unfamiliar with the process (Kays 1977).

The formulation of the seals produced by the ponding method depends on the initial seepage rate of the soil, and the soil composition and particle size distribution (Kays 1977). Experiments by Morrison et al (1971) showed that waterborne dispersions did not effectively penetrate silty sands. In general, waterborne dispersions should be considered as a possible means of improving the seepage control properties of an existing liner when emptying of the pond is not practical. They are not recommended as primary liner systems.

The complete control of seepage by the above method is not guaranteed by the manufacturers of waterborne dispersions. The treatment does not hold up well to dewatering and cleaning operations which take place in large reservoirs.

A8.3 CHEMICAL ABSORPTIVE LINERS

Chemical absorptive liners function by removing pollutants from liquid waste as it passes through the liner mass. This is a relatively new concept and is still in the development stages (Matrecon 1980). There has been little laboratory success (up to 1980) with single absorbents, however combinations of various absorbents, used in sequence, have proven more successful. The type, sequence, and behaviour of absorbents depend on soil and waste conditions, particularly pH.

APPENDIX 9
PREDICTION OF GROUNDWATER TABLE MOUNDING

A9.1 GENERAL

Once a liner has been designed to meet the seepage control criterion in Section 4.1.2, the potential height of groundwater table mounding below the pond should be calculated. The first step is to determine whether seepage will occur as unsaturated or saturated flow below the liner, as follows, (Folkes 1982 and included references):

(A9.1)

$$\text{If } \frac{K_s}{K_L} > \frac{D_{ww} + L - h_d}{L} \qquad \text{Then unsaturated flow occurs.}$$

(A9.2)

$$\text{If } \frac{K_s}{K_L} \leqslant \frac{D_{ww} + L - h_d}{L} \qquad \text{Then saturated flow occurs.}$$

where K_s is the hydraulic conductivity of the subsoil, K_L is the hydraulic conductivity of the liner, D_{ww} is the depth of wastewater in the pond, and L is the liner thickness. The term h_d is the air-entry pressure (negative value) for the soil at saturation. The value of h_d may be assumed to equal 0 for coarse sands and gravels and about -1 m for silts and clays. For a compacted clay liner of 1 m thickness, a wastewater depth D_{ww} of 2 m, and a h_d of -0.5 m, the hydraulic conductivity of the soil below the liner must be greater than 3.5 times the hydraulic conductivity of the liner for unsaturated flow to occur.

A9.2 MOUNDING DUE TO UNSATURATED SEEPAGE

The following is a simplified procedure for calculating the approximate mound height due to unsaturated seepage from the pond (from Bouwer 1979 and included references).

The height of the centre of the mound below square or rectangular ponds at any time, t, may be estimated using equation A9.3 and Fig.A9.1:

$$h_c = \frac{vt}{f} F\left\{\frac{W}{(4KDt/f)^{1/2}}\right\} \qquad \text{(A9.3)}$$

where h_c is the rise of the mound centre (m),

- v is the seepage rate through the liner (m/s),
- t is the time since first arrival of the seepage front at the saturated zone (s),
- f is the fillable porosity of the unsaturated zone above the groundwater table, e.g. specific yield (dimensionless ratio),
- W is the pond width (m),
- K is the hydraulic conductivity (horizontal) of the saturated soil (m/s),
- D is the thickness of the saturated soil zone (original thickness D_o may be used, although the mean thickness is more correct, i.e. $D_o + h_c/2$, which requires an iterative procedure),
- $F\{\ \}$ is the function shown on Fig.A9.1.

Equation A9.3 is only valid for values of h_c less than $0.5D_o$.

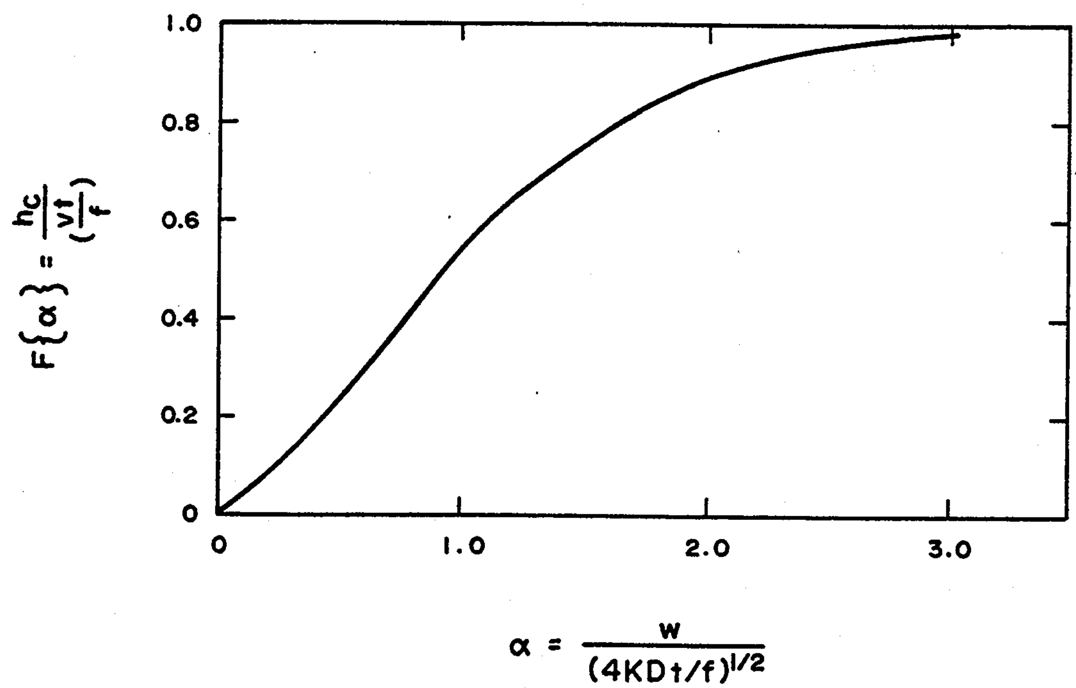


FIG. A9.1 GRAPH RELATING RISE OF CENTRE OF MOUND TO SYSTEM PARAMETERS FOR SQUARE BASIN (FROM BIANCHI AND MUCKEL, 1970)

The time, t, required for the seepage front to reach the saturated zone after pond filling may be calculated from equation A9.4:

(A9.4) $$t = \theta(D_w - h_f)/v$$

where D_w is the depth of the groundwater table below the bottom the pond liner (m),

h_f is the height of the saturated portion of the capillary fringe above the groundwater table,

θ is the volumetric moisture content of the unsaturated soil zone (dimensionless ratio).

Typical values of θ for soils subject to gravity drainage are shown in Table A9.1 from Todd (1955), where

(A9.5) $$\theta = n - Sy$$

The term n is the porosity of the soil, and Sy is the specific yield, which is the fraction of pore water that drains from a unit volume of soil under gravity drainage.

EXAMPLE A9.1

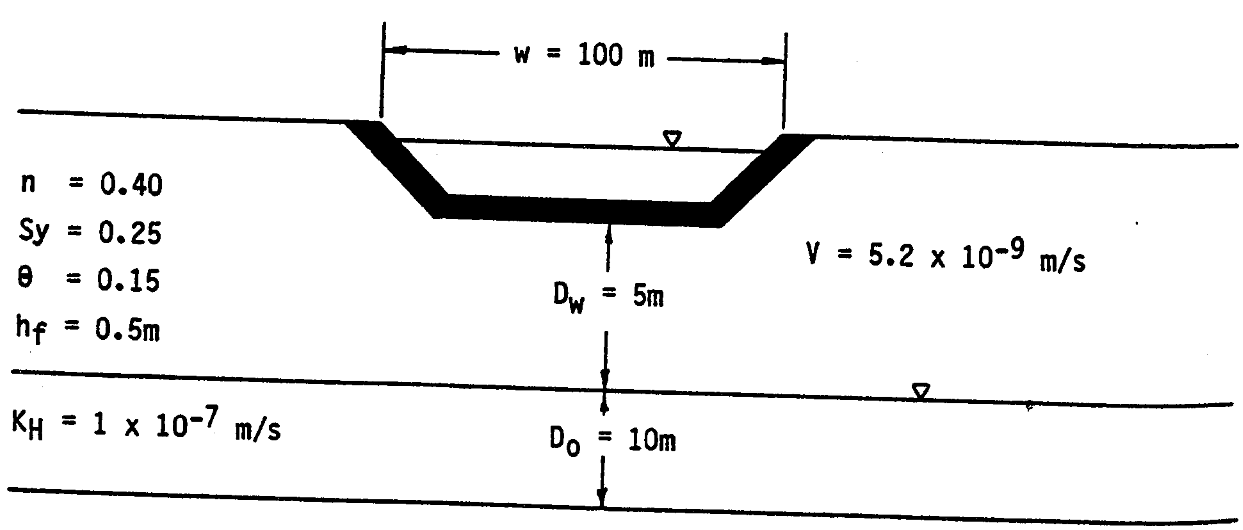
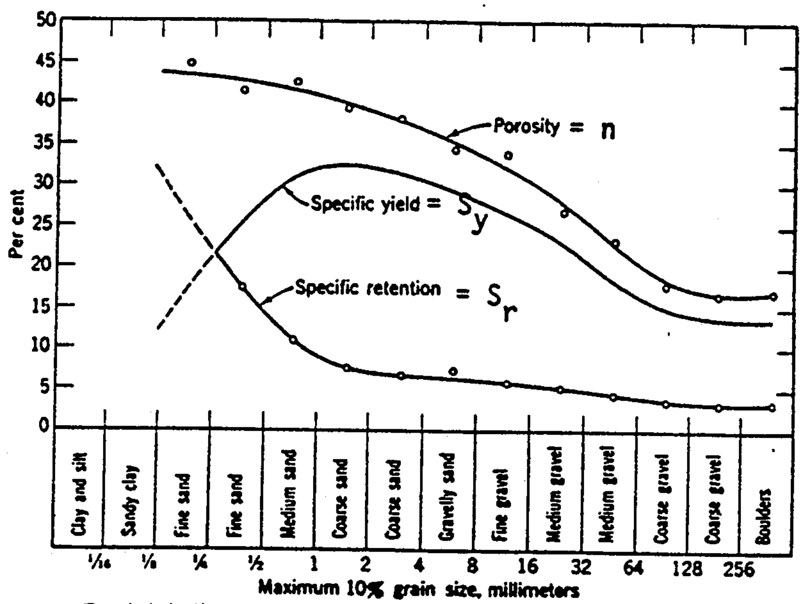


TABLE A9.1 TYPICAL VALUES OF SPECIFIC YIELD AND RETENTION (from Todd 1955)



NOTE: Volumetric moisture content, θ, of soils subject to free gravity drainage is calculated by:

$$\theta = S_r = n - S_y$$

i) time required for seepage front to reach saturation zone after filling pond:

$$t = (0.15)(5m - 0.5m)/5.2 \times 10^{-9}\ m/s$$
$$= 1.3 \times 10^{8} s$$
$$= 4.1 \text{ years}$$

ii) mound height at $t + 10$ years

$$h_c = \frac{(5.2 \times 10^{-9}\ m/s)(3.15 \times 10^{8} s)}{0.25}$$
$$\times F[100m/(4 \times 1 \times 10^{-7}\ m/s \times 10m \times 3.15 \times 10^{8} s/.25)^{1/2}]$$
$$= 6.6 \times F(1.41) \quad : F(1.41) = 0.73 \text{ from Fig. A9.1}$$
$$= 4.8\ m$$

Therefore the model predicts that the centre of the mound will approach the bottom of the liner about 14 years after filling the pond.

The above analyses ignore the effects of fluctuating wastewater depths, changes in material properties with time, and fluctuations in the natural groundwater table due to other causes. The results are approximate and should only be considered as a rough guide to mounding behaviour. More sophisticated modelling may be more accurate, but is generally limited by the accuracy of the measured parameters used for input data.

A9.3 MOUNDS DUE TO SATURATED SEEPAGE

In situations where saturated seepage occurs, the flow regime will eventually establish a groundwater recharge mound, as shown in Fig. 4.4 in Section 4.4.3. Because the soil within the zone of seepage (behind the seepage front) is already saturated, a mound exists as soon as the seepage front reaches the groundwater table. The time required for the seepage front to reach the saturation zone of the groundwater table in homogeneous material may be estimated by:

$$t = \frac{Sy}{K}\left[D_w - D_{ww} \ln\left\{\frac{D_w + D_{ww}}{D_w}\right\}\right] \tag{A9.6}$$

where Δt = time for seepage front to reach groundwater table or zone of saturation (s),

Sy = specific yield (dimensionless ratio),

K = vertical saturated hydraulic conductivity of the soil (m/s),

D_w = distance to the groundwater table or zone of saturation below the pond liner (m),

D_{ww} = depth of wastewater in the pond (m).

A more general form of equation (A9.6) is presented by McWhorter and Nelson (1979).

If saturated seepage is anticipated (from Equation A9.2) or if analyses indicate that unsaturated seepage will cause mounding to the base of the liner or higher within the design life of the pond, then an estimate should be made of the steady-state position of the groundwater table. High groundwater table levels that create surface seeps or boggy conditions may require control through use of drains or lower permeability liners (see Section 4.4.4).

The steady-state position of the groundwater table may be estimated by numerical (computer) models (e.g. Neuman and Whitherspoon 1970); electric analog models (Bouwer 1964); or by flow net construction. Computer seepage analyses were carried out by the authors for the pond shown on Fig.A9.2. The position of the groundwater table at various times after filling the pond was computed for different hydraulic conductivities in the soil. For this particular case, it was found that significant mounding did not occur within 25 years of filling the pond if the horizontal K value of the soil was more than 1000 times the K value of the liner. It was also found that varying the ratio of horizontal K to vertical K in the soil from 1 to 10 did not significantly effect the height of the mound.

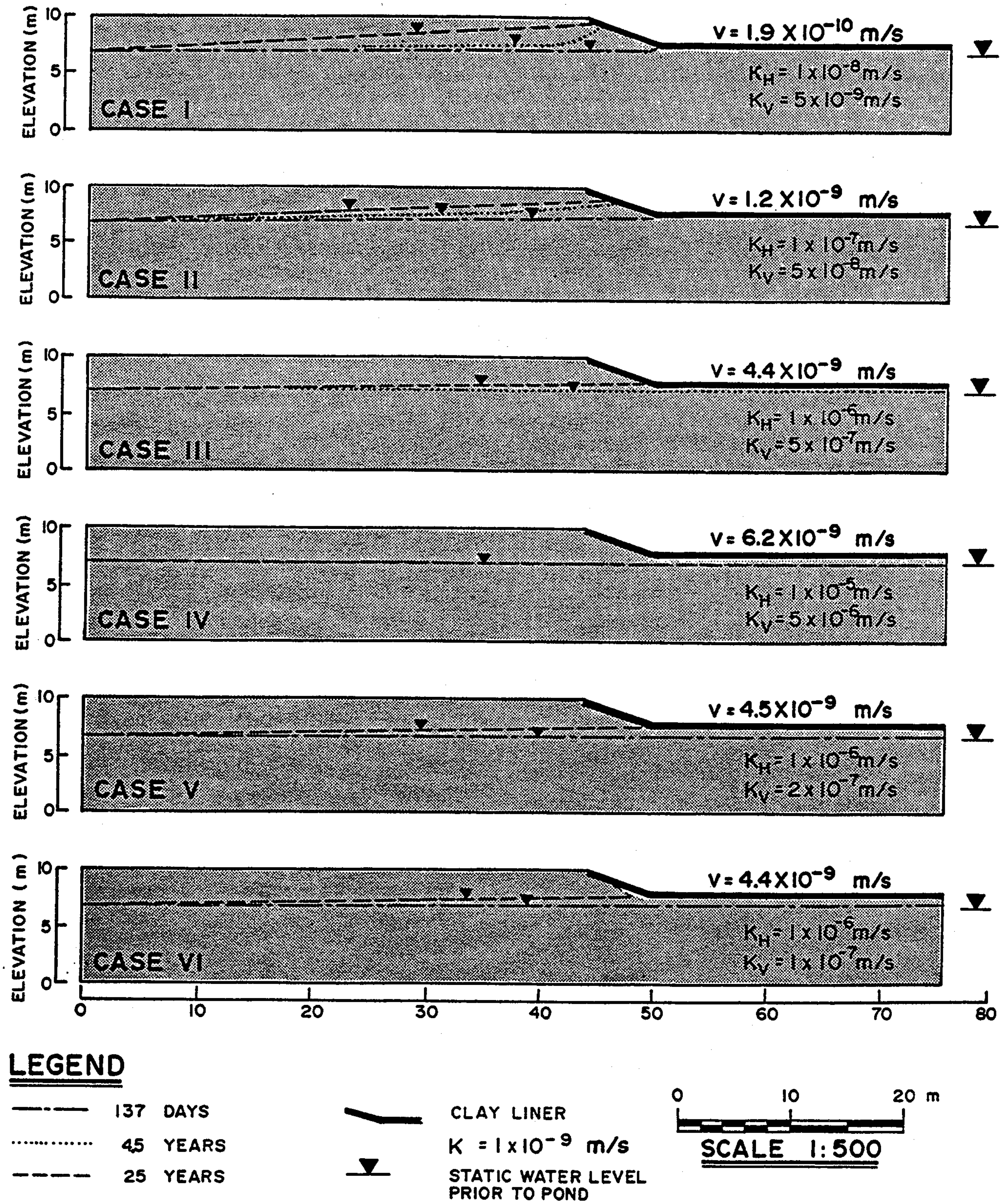


FIG. A9.2 COMPUTER ANALYSES OF SEEPAGE FROM LINED POND SHOWING POSITION OF GROUNDWATER TABLE WITH TIME

APPENDIX 10
EARTH VOLUME CHARTS

The following charts allow estimation of approximate cut and fill volumes for square and rectangular ponds ranging from 50 m x 50 m to 300 m x 300 m (inside crest dimension). The charts assume 3:1 sideslopes, 3 m wide crests, and a level ground surface. The variable dimensions are total pond depth H (includes freeboard) and the height h of the dykes above the stripped ground surface. Stripping volumes are not included.

An example of the chart use is shown on the first chart (Fig. A10.1) for the 50 m x 50 m pond size. In the example, an overall pond depth H of 3 m is required and a dyke height of 1.5 m is selected. Starting at an h value of 1.5 m on the horizontal axis and going vertically up to the H equal to 3 cut volume line, we find that the cut volume is about 2000 m^3, while going vertically down to the fill volume line we find that the fill volume is about 2400 m^3. The required borrow is thus 400 m^3, assuming that the native soil density and compacted backfill density are the same and that all of the cut material is suitable as fill. The borrow (or waste) volume may also be directly determined by going vertically from h equals 1.5 m to the H equals 3 m waste/borrow line[1]. In this case we must go downwards to intersect the line and again find that the borrow volume is about 400 m^3.

The charts may be useful for quick estimations of cut and fill requirements. However, more accurate volume calculations should be made during final design.

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1 The waste/borrow lines indicate waste volumes above the horizontal axis, and borrow volumes below the horizontal axis.

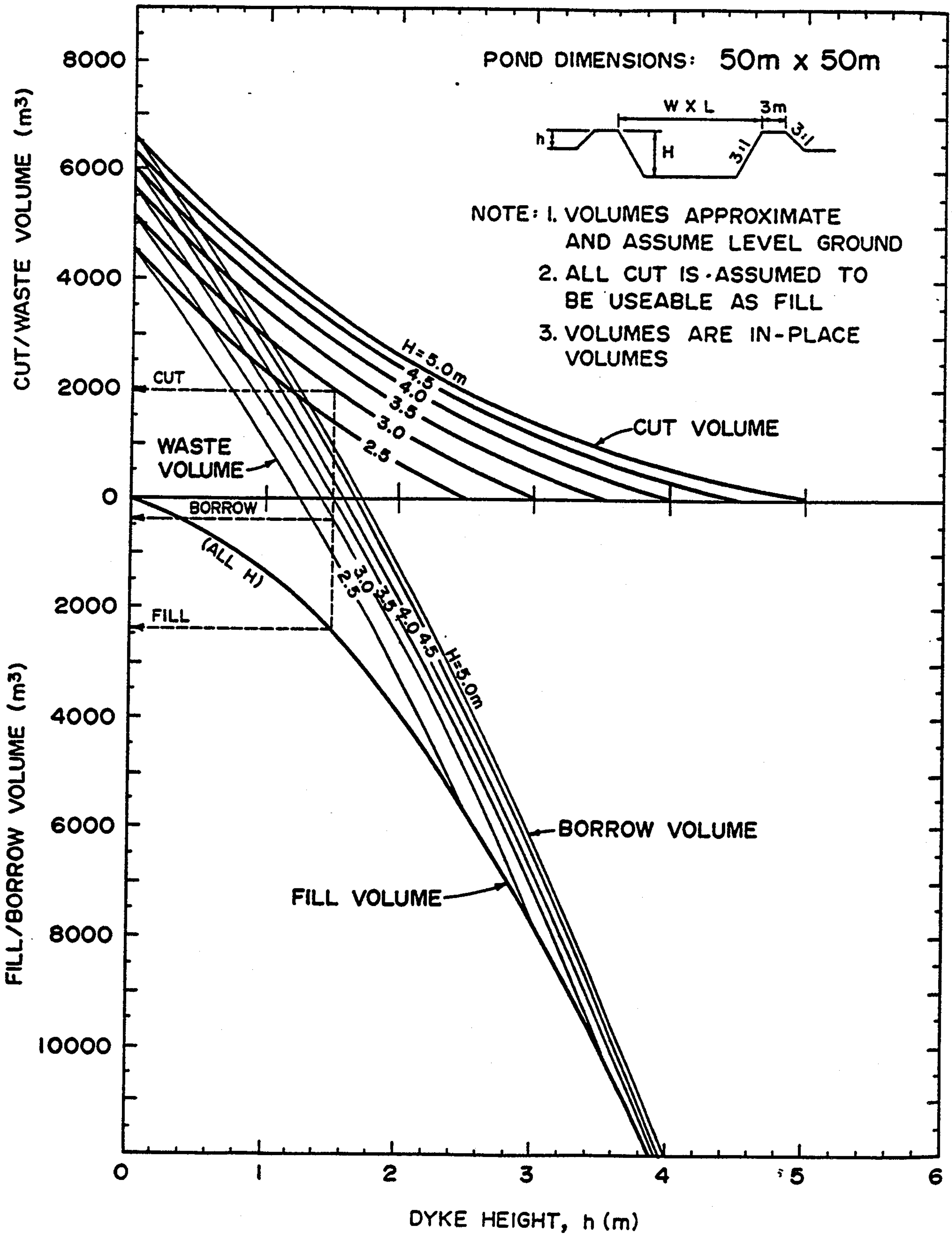

FIG. A10.1 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 50m X 50m POND

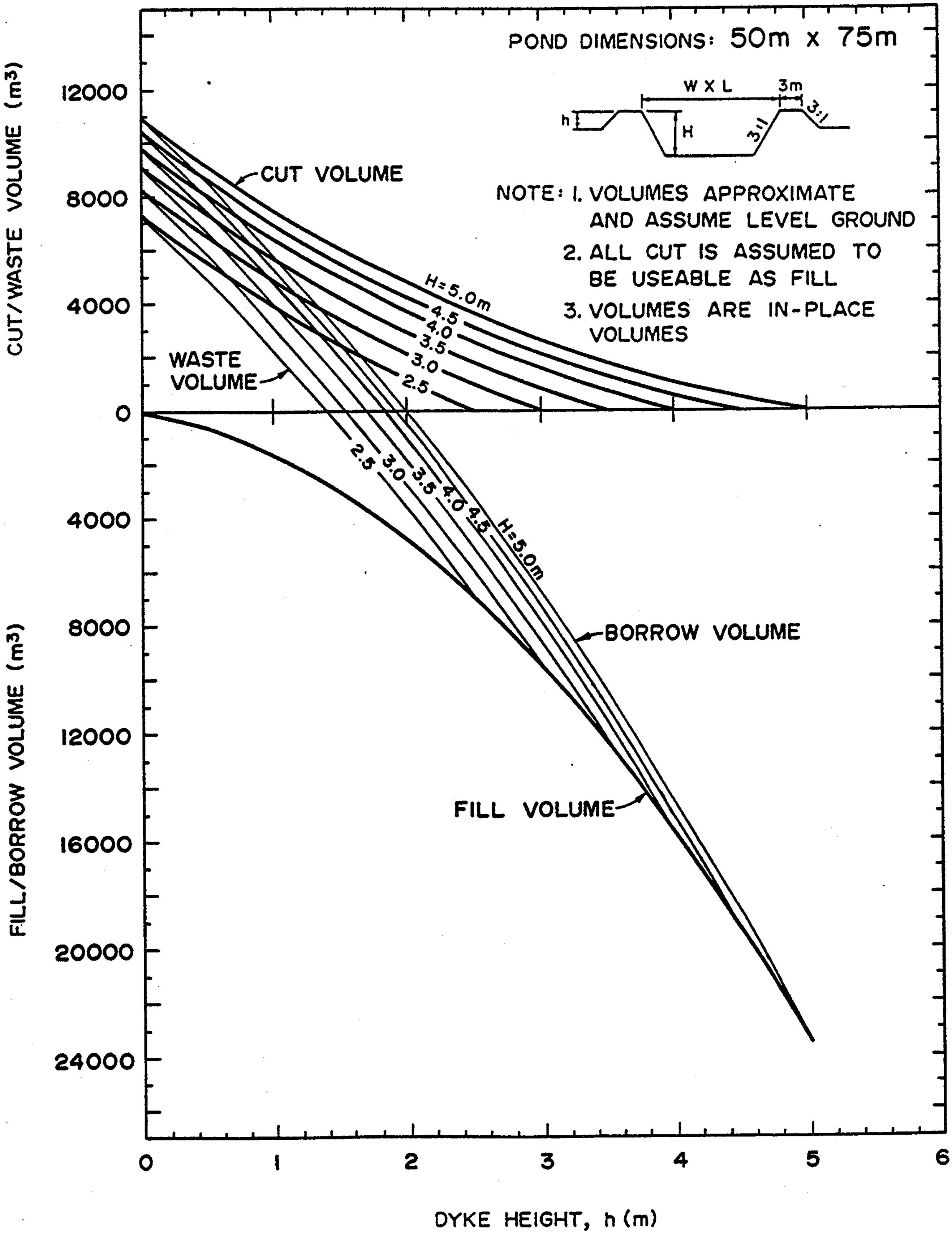

FIG. A10.2 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 50m X 75m POND

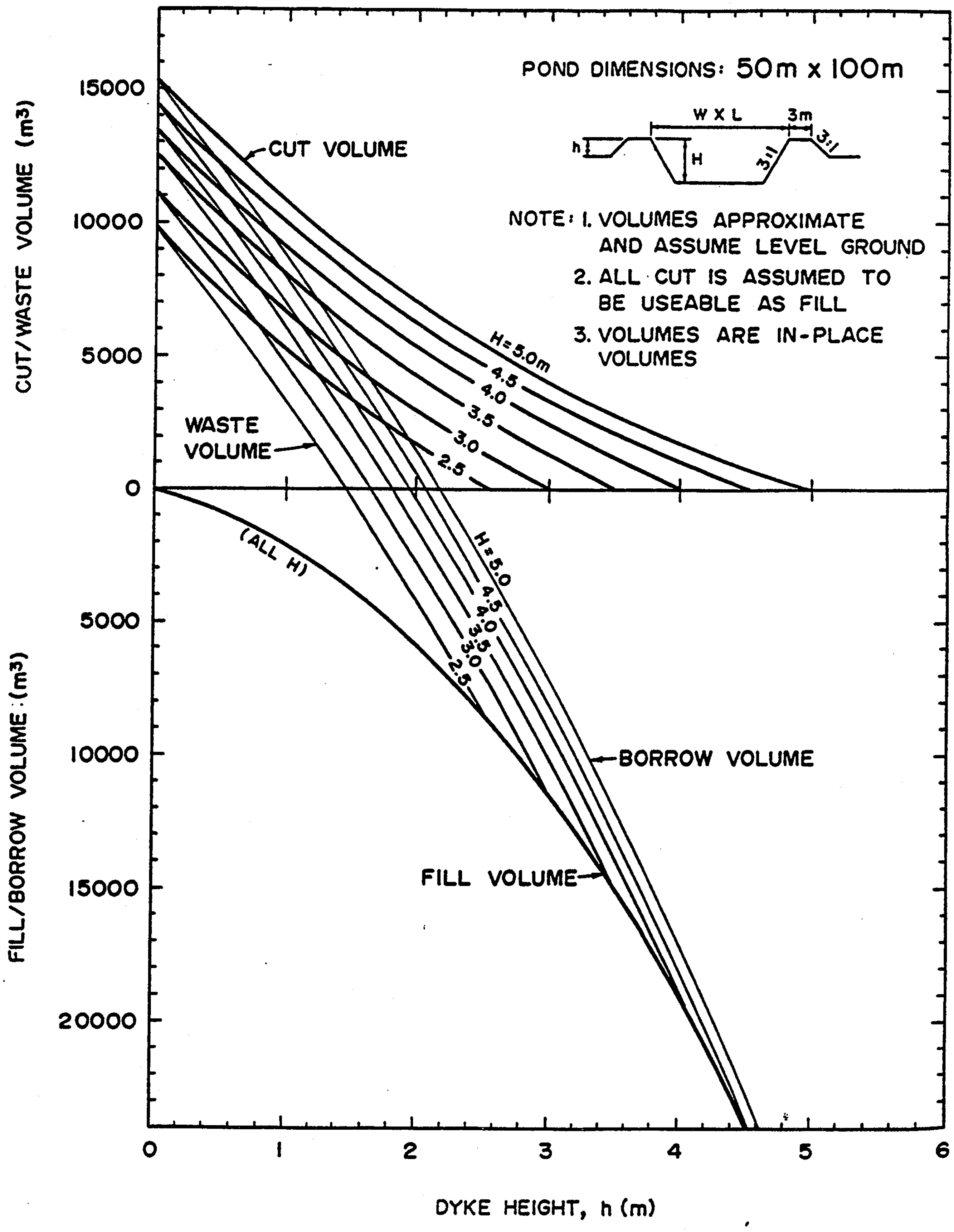

FIG. A10.3 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 50m X 100m POND

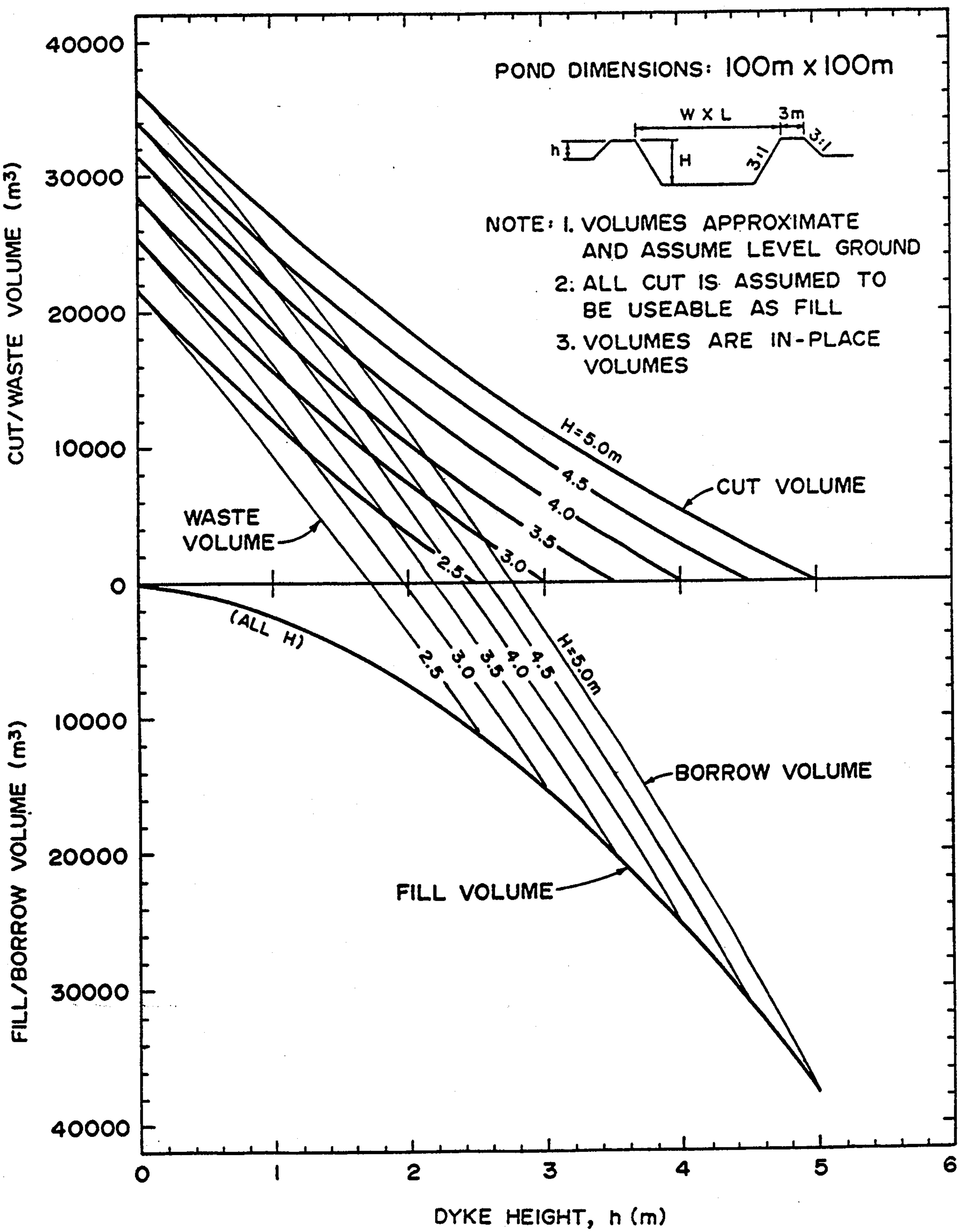

FIG. A10.4 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 100m X 100m POND

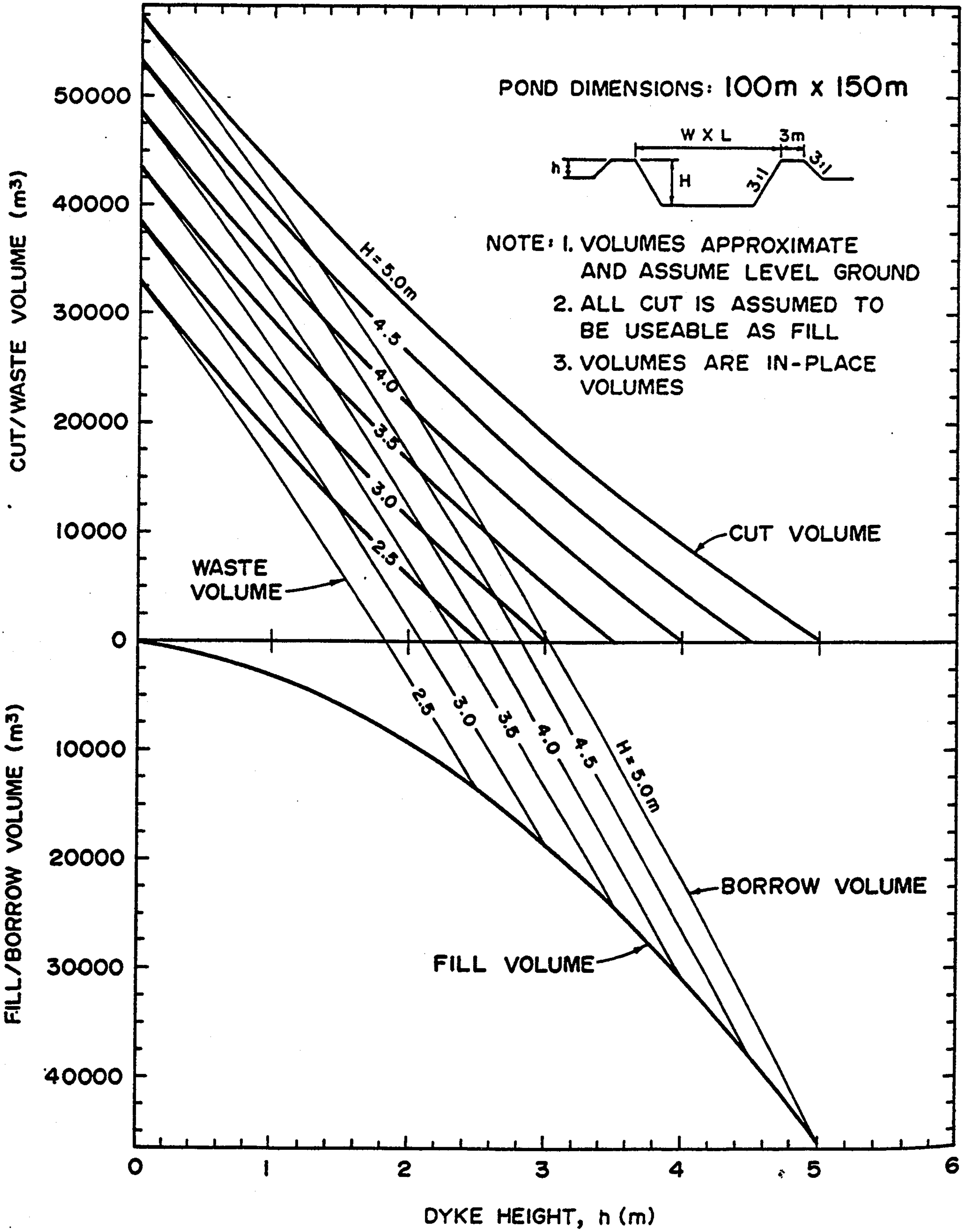

FIG. A10.5 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 100m X 150m POND

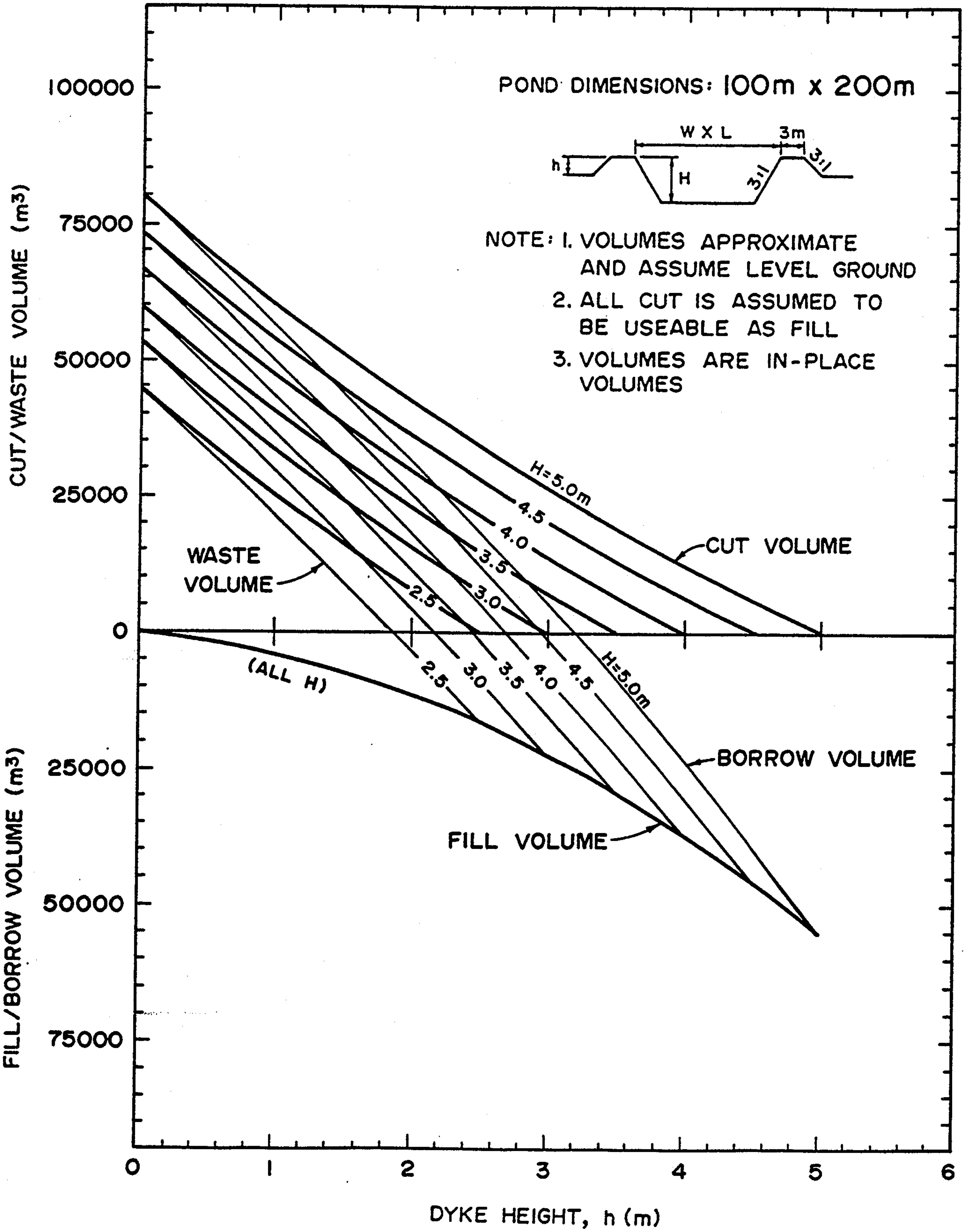

FIG. A10.6 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 100m X 200m POND

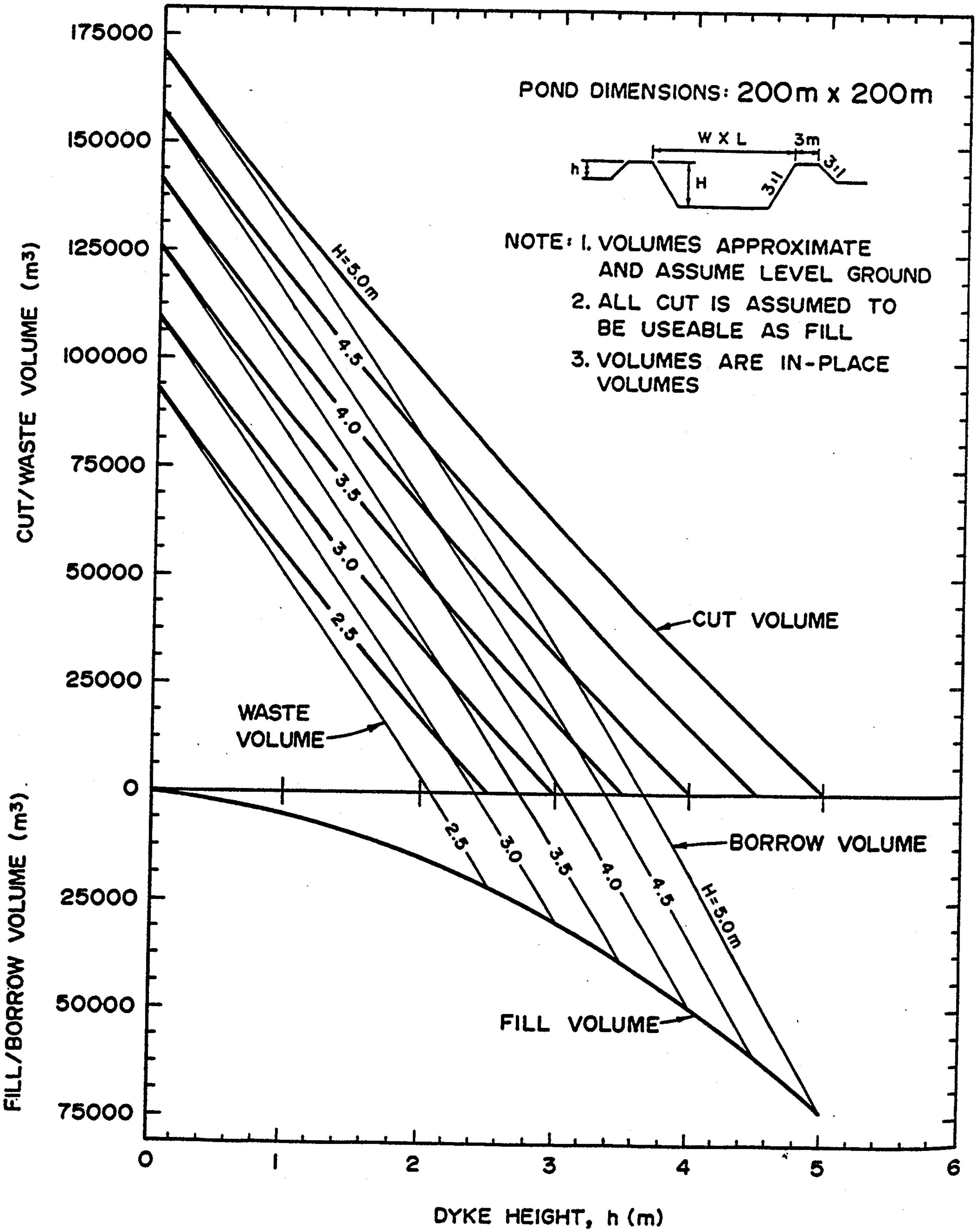

FIG. A10.7 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 200m X 200m POND

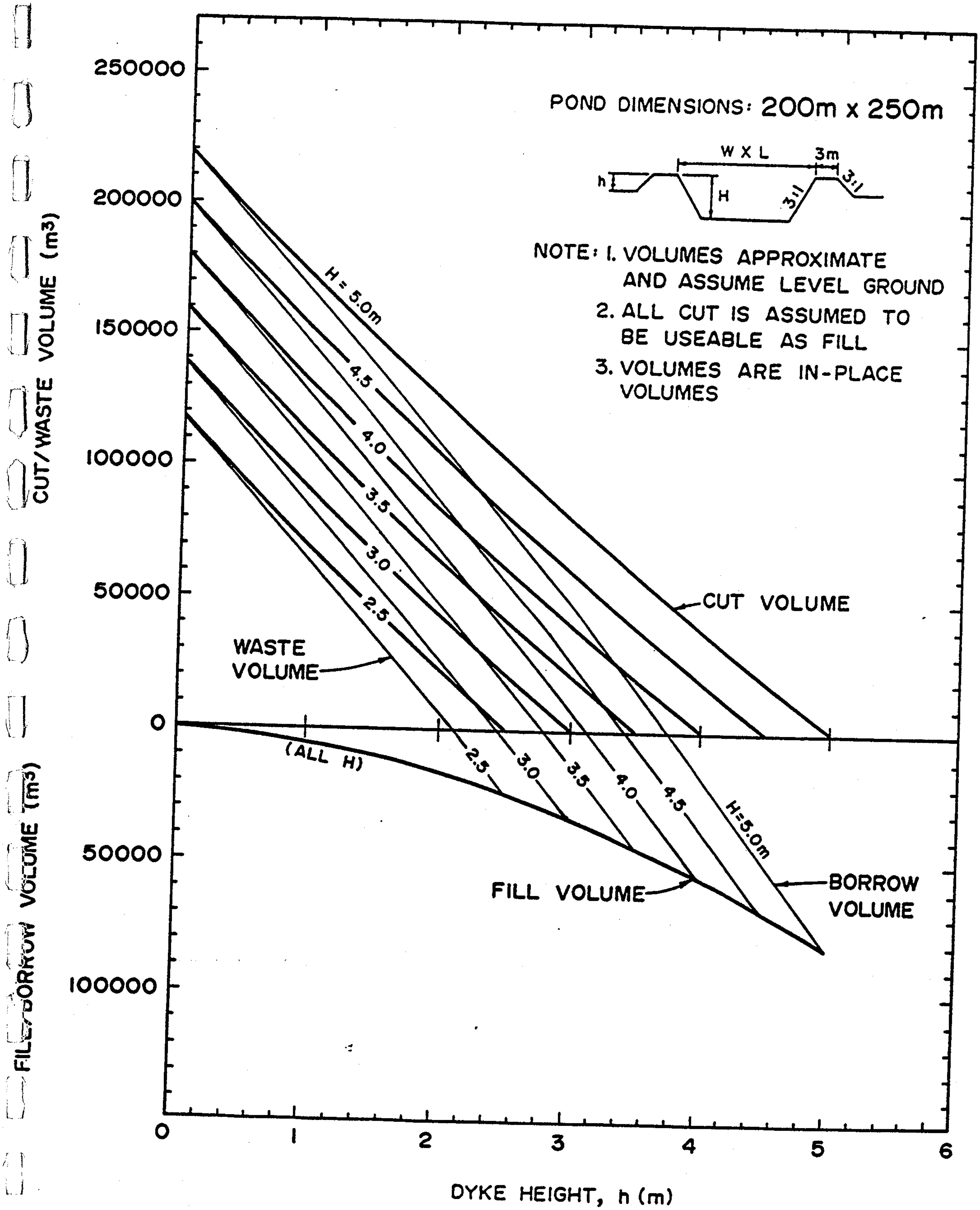

FIG. A10.8 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 200m X 250m POND

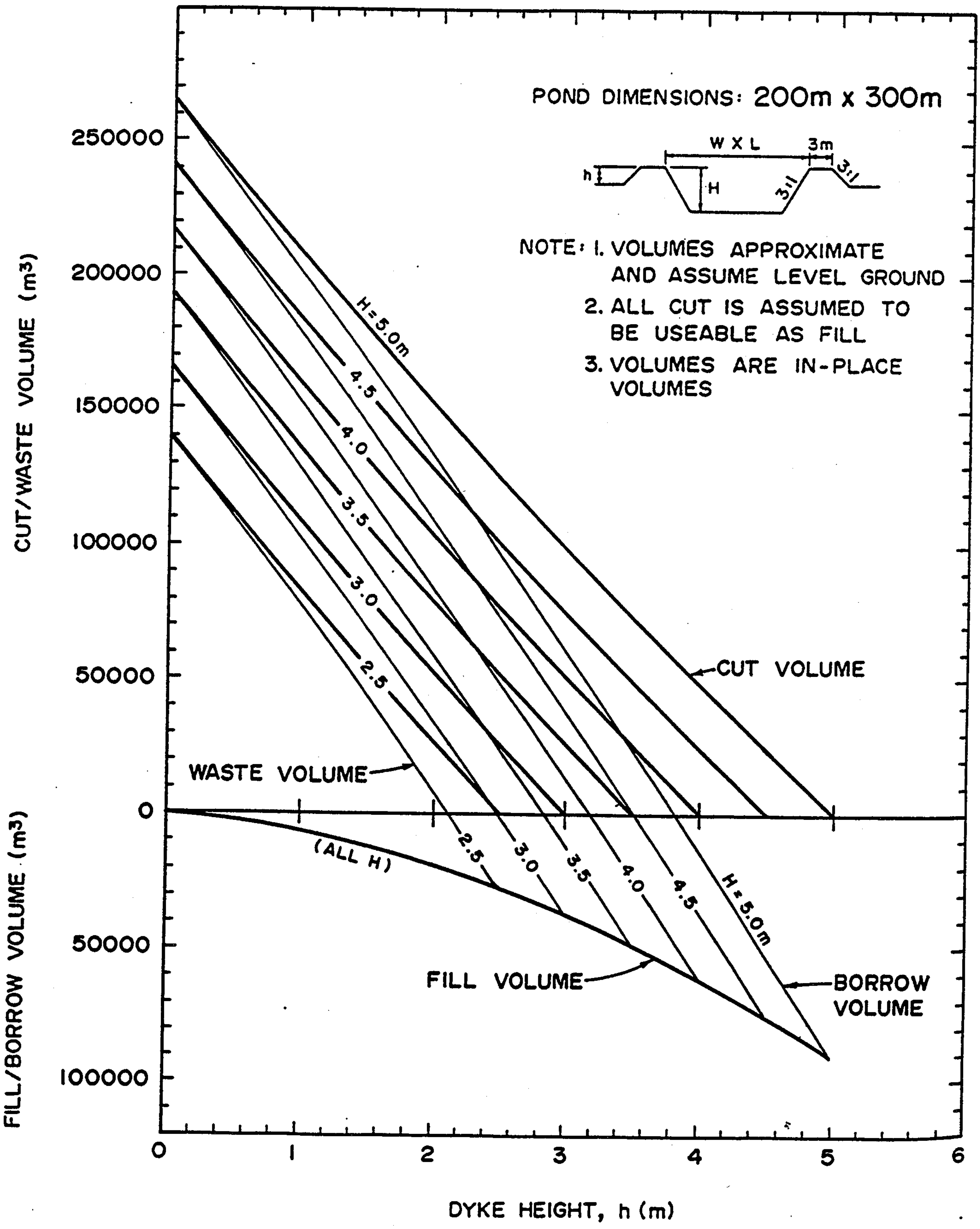

FIG. A10.9 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 200m X 300m POND

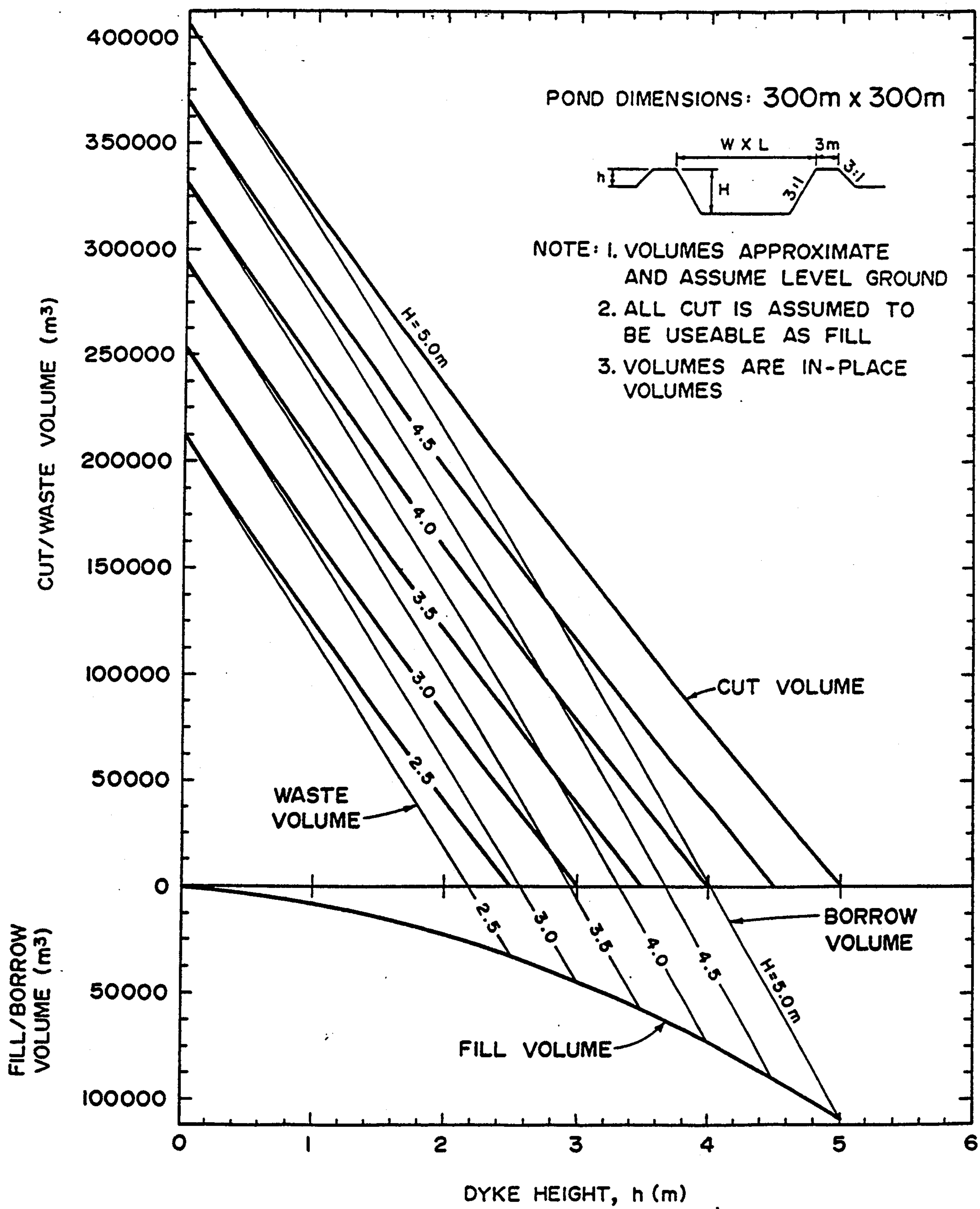

FIG. A10.10 APPROXIMATE EARTH VOLUMES FOR CUT AND FILL FOR 300m X 300m POND

APPENDIX 11
GROUNDWATER MONITORING

A11.1 GROUNDWATER MONITORING METHODS

A11.1.1 General

There are a number of methods available for groundwater monitoring. Some methods provide only groundwater table elevation data, some methods provide only water quality data and some methods allow measurement of both piezometric heads and water quality. The most practical monitoring methods are:

i) Wells and piezometers which provide water quality and groundwater table elevation data;

ii) Multilevel sampling devices which provide water quality and groundwater table elevation data;

iii) Geophysical methods which provide information on extent and rate of movement of a contaminant plume;

iv) Core sampling which provides knowledge of leakage only;

v) Grid leak detection which provides knowledge of leakage only.

The key issue that must be decided before proceeding with the design of a groundwater monitoring network is the scale of sampling that is desired. This refers to the volume or vertical interval over which piezometric heads are averaged and from which samples are to be drawn. With the depth-integrating approach samples are drawn from wells with long screened intervals (Fig.A11.1). With the point-sample approach

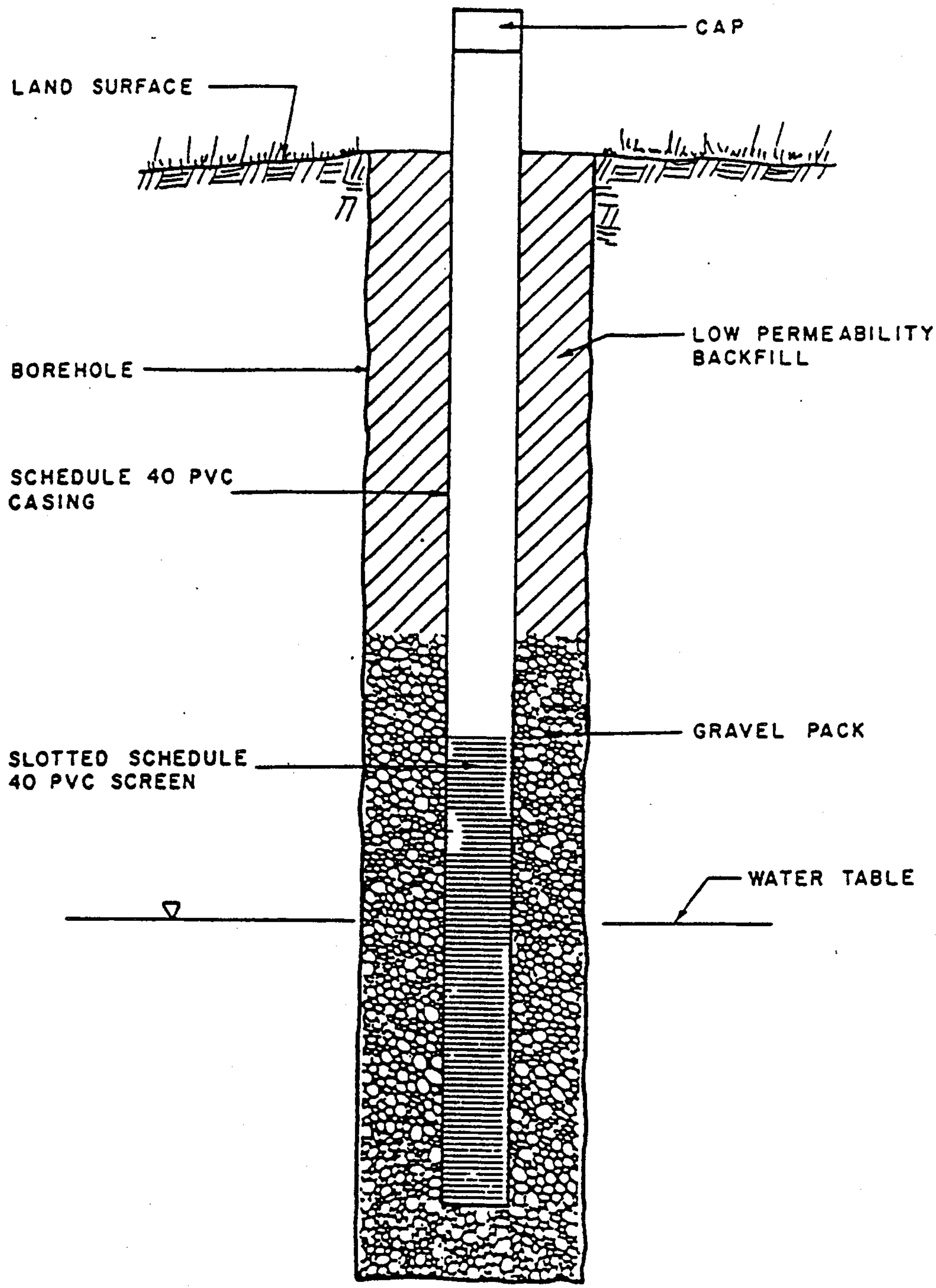

FIG. A11.1 DEPTH INTEGRATED WELL

water samples are drawn from discrete levels in the groundwater zone. If the point-sampling approach is used, several or numerous samples are usually needed at each monitoring site so that a depth profile of samples can be acquired. Piezometric heads and water quality data are known for various elevations within the profile. If the depth-integrated approach is used, water is drawn from all or many zones in the vertical interval intersected by the well screen. If a zone of nutrient enriched groundwater occurs in the interval and if unaffected zones also occur in it, nutrients will be definitely included in the water sample pumped from the well or piezometer, although at a lower concentration than occurs in the aquifer. In some cases the dilution may cause the nutrients to go undetected.

Piezometric heads are generally controlled by the most permeable strata in a vertical interval and are not representative of hydraulic heads in the less permeable strata. In fact, the presence of an open borehole can drastically distort the flow pattern in an anisotropic sedimentary sequence. Most hydrogeologists prefer to avoid depth-integrated sampling because of the complexities or ambiguities that it introduces into the interpretive process. Concentrations obtained from depth-integrating devices have no meaning in terms of the equations that are used in solubility, adsorption, or redox computations and they can have little discernable significance with respect to dispersion. Data obtained from point sampling can be averaged to provide depth-integrated samples if required, but not vice versa.

A11.1.2 Wells and Piezometers

A well is a hydraulic structure, usually vertical, designed for bringing groundwater to the surface. Wells consist of casings with diameters ranging from less than 100 mm to over 1200 mm usually with a screen section at the bottom. Well design has to satisfy two requirements:

i) accommodate the pump or bailer with proper clearance for installation and efficient operation

ii) assure hydraulic efficiency of the well

Wells and piezometers are very similar structures, the only major differences being casing diameter, purpose of installation, and length of the slotting interval. Wells usually have larger diameters than piezometers, which should be large enough to allow accurate and rapid measurements of the water levels, and longer screened intervals.

Piezometers used in groundwater monitoring are usually open pipes with diameters in the range of about 10 to 80 mm with screened or slotted segments at the bottom. The pipe and screen assembly is usually sealed in the borehole in a manner that prevents flow in the hole above the screen. Piezometer screens are usually between 0.1 m and 2 m in length. Depending on the nature and scale of the problem, they may provide water samples that are significantly influenced by depth integration, or they may provide what is essentially a point sample if the screen is short.

For groundwater quality monitoring, piezometers are often used in clusters or nests with a single piezometer in each borehole in the nest or with several piezometers sealed at different levels in a single borehole (Fig.A11.2). Installation of several piezometers in a borehole minimizes drilling costs but often provides uncertainty with respect to cross-connection from one zone to another if it cannot be established that adequate sealing between piezometers has been achieved.

A11.1.3 Multilevel Samplers

In recent years various types of multilevel sampling devices have been developed for obtaining water quality and hydraulic head profiles in cohesionless deposits of sand or gravel. Two of these devices are

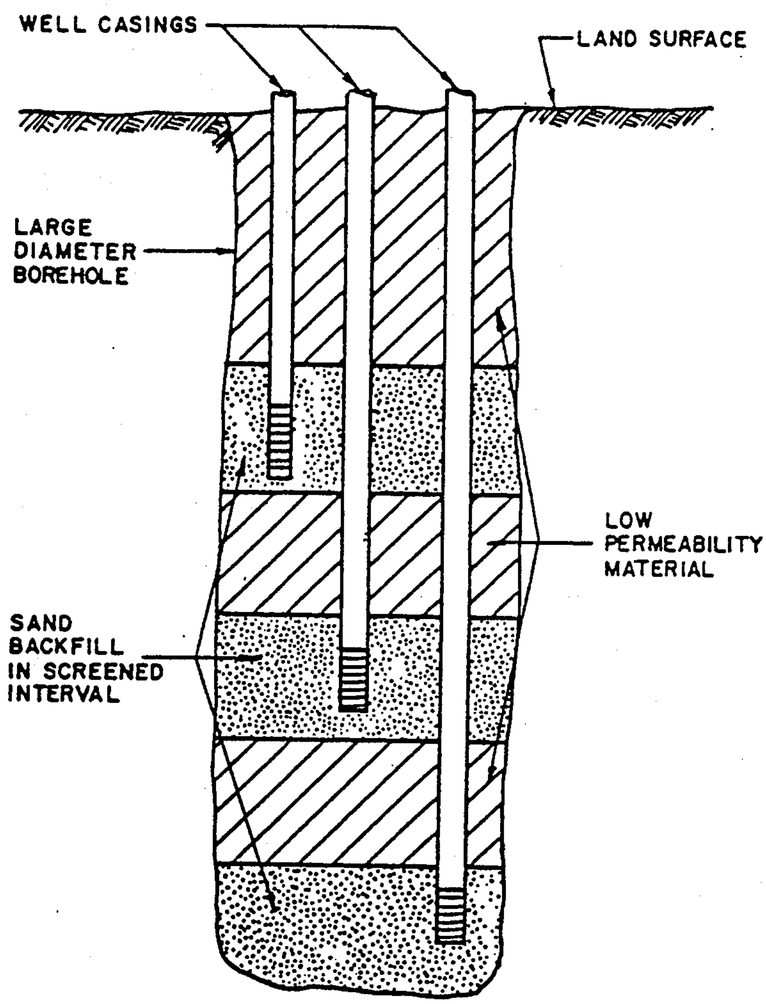

FIG. A11.2 SEVERAL PIEZOMETERS SEALED IN ONE BOREHOLE shown in Fig.A11.3. Each consists of a group of polyethylene or polypropylene tubes that extend down a single borehole, each tube to a different depth.

The device referred to as a multilevel point sampler has a group of tubes that extend down a PVC casing. Each tube is attached to a small screened sampling port that is sealed in a hole in the casing. Water samples are acquired by vacuum pumping if the static water level is less than about 8 m below ground surface. This type of multilevel sampler is described in detail by Pickens et al (1978). More recently, an inexpensive valve system for attachment to each tube near the sampling ports has been developed so that samples can be collected under positive-pressure displacement regardless of the depth to static water level.

The second type of multilevel sampler shown in Fig.A11.3 is referred to as a bundle piezometer. This simple device consists of a bundle of tubes bound to a central PVC pipe. Each tube and the PVC pipe has a short screen or slotted segment at the bottom. Bentonite seals can be attached in netting to the bundle between the tips to prevent vertical movement of groundwater along the tubes. Water samples are collected by drawing water by suction or, if the static water level is deeper, by insertion of a narrow tube down the individual piezometer tubes. The sampling tube has a check value on the bottom so that water can be drawn up in the tube by vacuum and then removed using the inner tube as a bailer.

The monitoring devices shown in Fig.A11.3 are conveniently installed in sand and gravel to depths of about 15 metres using the hollow-stem auger method of drilling. Each borehole can therefore provide between 1 and 5 sampling points when these devices are used.

The devices shown in Fig.A11.3 have been used at numerous sites of groundwater contamination in central Canada. Other useful multilevel devices have appeared in the literature in recent years. Each device

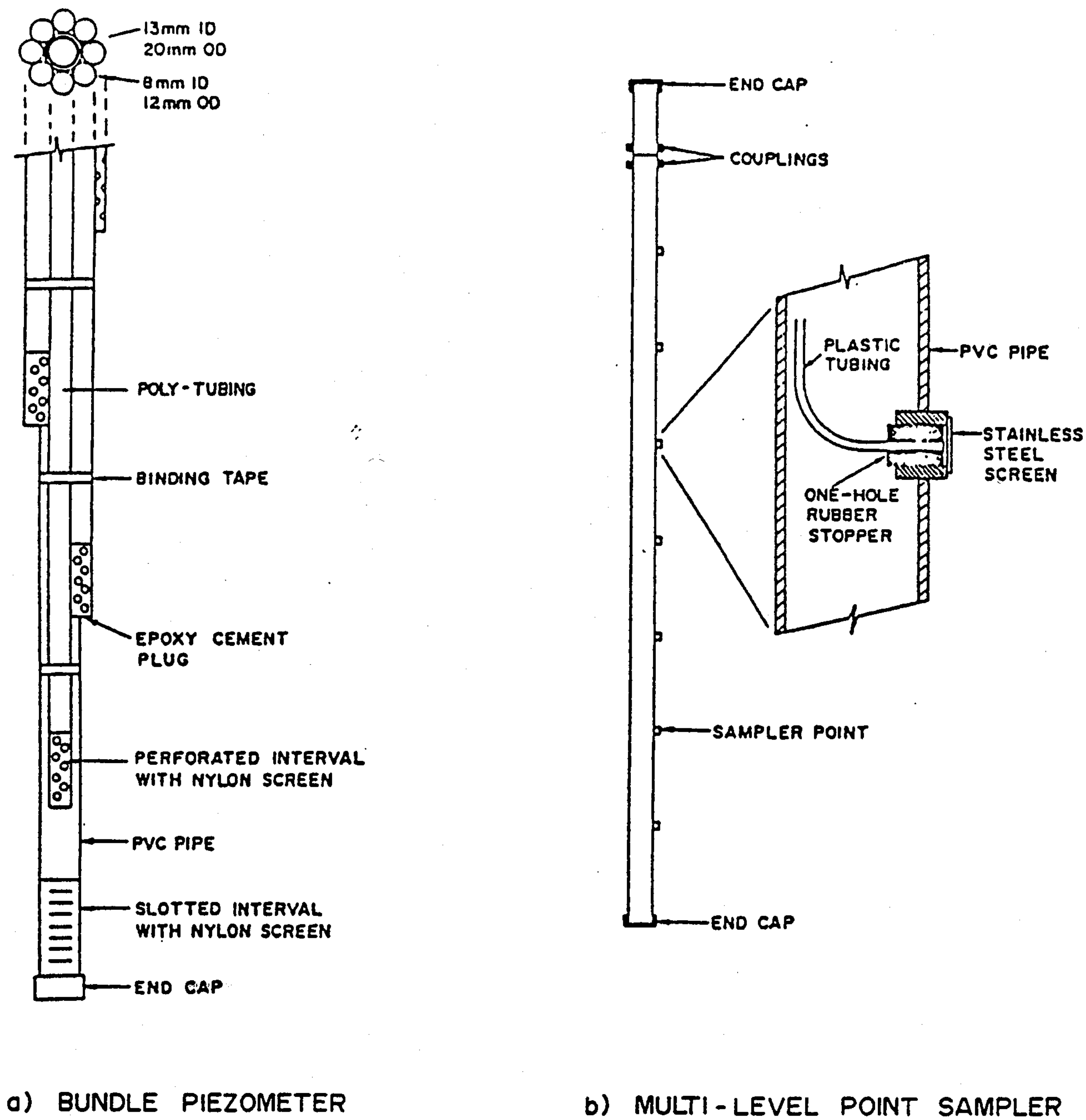

FIG. A11.3 MULTI-LEVEL SAMPLERS

has advantages and disadvantages that are specific to the particular site or type of contaminant source.

A new type of multiple piezometer has recently been used for groundwater quality monitoring along with conventional means of multiple-level monitoring. It is illustrated in Fig.A11.4. This device was first developed at the University of Waterloo, Ontario (Cherry and Johnson 1982). It consists of a single string of jointed PVC pipe with ports for pressure measurements and for water sampling at whatever number of depth levels is desired. Packers made out of an expanding rubber compound are installed between the ports. This rubber will expand after it becomes in contact with water that is introduced through the centre of the PVC pipe. This multiple-port device has the potential for providing groundwater quality profiles at numerous depths in single boreholes in soil or rock at costs that are considerably less than conventional piezometers. It is particularly useful in fractured rock where many sampling levels are necessary because of the complexity of the permeability network and the uncertainty with regard to the specific zones through which contaminants will travel. The cost of drilling in rock is much higher than that of drilling in unconsolidated deposits and therefore there is a strong economic incentive for construction of multiple sampling points in a single borehole.

A11.1.4 Geophysical Methods

If seepage water has a high enough TDS to contrast significantly with the natural groundwater, it is possible to delineate the extent of contaminated groundwater using surface resistivity techniques. Resistivity profiles can be run along a number of lines, and the interface between natural groundwater flow and contaminated flow can usually be detected. This method does not provide a measure of water quality, but if the survey is repeated once every few months, the movement of the contaminant plume can be monitored.

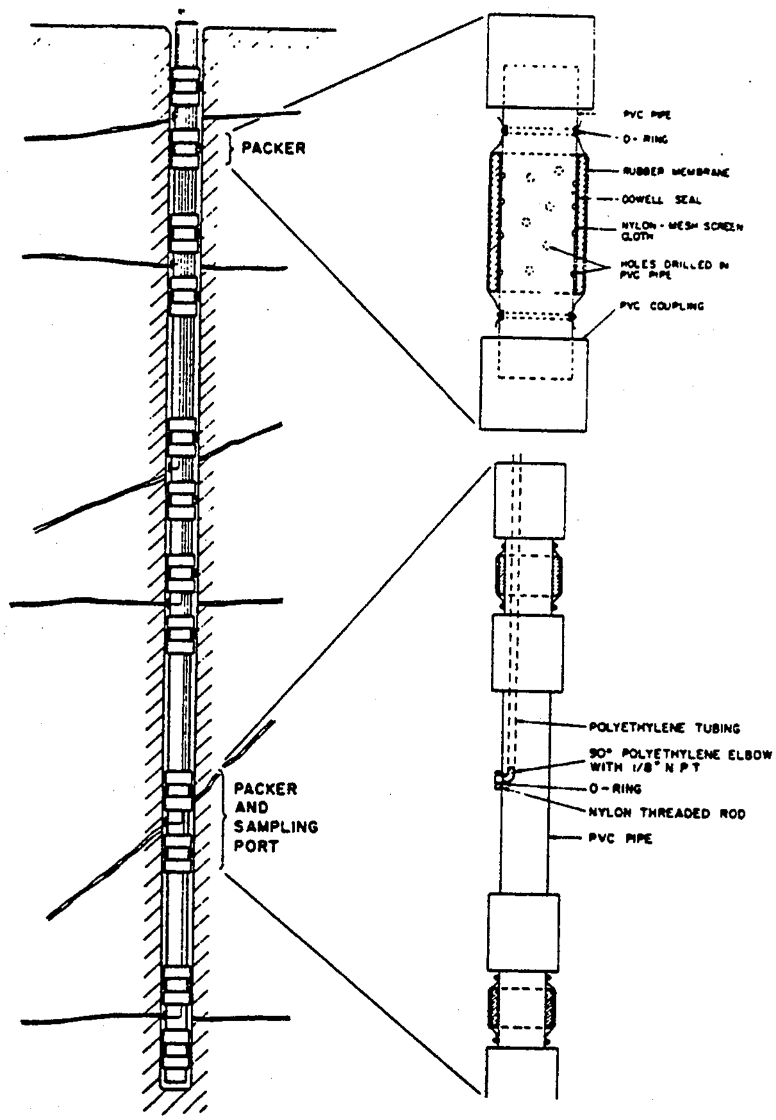

FIG. A11.4 MULTIPLE PIEZOMETER
(FROM CHERRY AND JOHNSON 1982)

A11.1.5 Pore Water Extraction

An alternative approach to groundwater monitoring is to extract cores from the geological domain and extract pore water from the cores. This approach is useful in areas of non-indurated deposits and can provide samples from above and below the water table. A comprehensive description is provided by Paterson et al (1976). In some cases coring and pore-water extraction can be used to supplement piezometers and multilevel sampling devices. In other cases the coring and extraction technique has advantages that make it the best choice. It is particularly useful for acquiring water samples from above the water table (i.e from the unsaturated zone).

One of the main disadvantages of the coring and squeezing approach is that for monitoring of contaminant migration over time, repetitive coring is necessary. In contrast, a good network of piezometers or multilevel sampling devices can be sampled on different occasions without additional drilling. One of the major advantages of the coring and extraction technique is that contaminant concentrations in both the pore water and the solids can be determined.

A11.1.6 Grid Leak Detection

A new grid leak detection system has recently been developed (Johnson 1983) which allows early identification of any leakage which occurs from a pond. The system consists of a grid of coated or uncoated aluminum, copper or steel wires. The wire and coating can be tailored to the expected chemistry of the seepage water. For instance, a .040" copper wire will completely corrode within two days if a 5% HNO_3 solution is present, while a .040" aluminum wire will not corrode appreciably. This system only indicates the occurrence of seepage and not the quality of the seepage water.

A11.2 LOCATION OF GROUNDWATER MONITORING INSTRUMENTS

Well, piezometer or multi-level sampler installation should be designed to provide:

i) Baseline data for both groundwater flow and groundwater quality at a site

ii) Control chemistry data for a point up-gradient from a sewage lagoon.

iii) Vertical and horizontal hydraulic gradients to delineate groundwater flow after sewage lagoon is constructed

iv) Position and concentration of nutrient enriched groundwater

To achieve the above listed requirements a monitoring system for a typical sewage lagoon should consist of a minimum of one instrument (well, piezometer, multi-level sampler) per 2 ha of lagoon area with a minimum of five instruments per site. In the case of a site which is being considered as critical by the Department, the number of installations should be increased according to the geological conditions and Department requirements.

The instruments should be located based on the results of preliminary geological, geotechnical and hydrogeological exploration results. In general, instrument screens should be placed in the most permeable formations. Configuration of the groundwater table should also be considered in locating instruments. At least one instrument should be located up gradient from the lagoon (Fig.A11.5). A second line of instruments may be installed further down gradient when traces of contamination are detected in some of the original wells. A geophysical survey could also be run to infill between piezometers if water chemistry results show the presence of a nutrient enriched seepage front. The

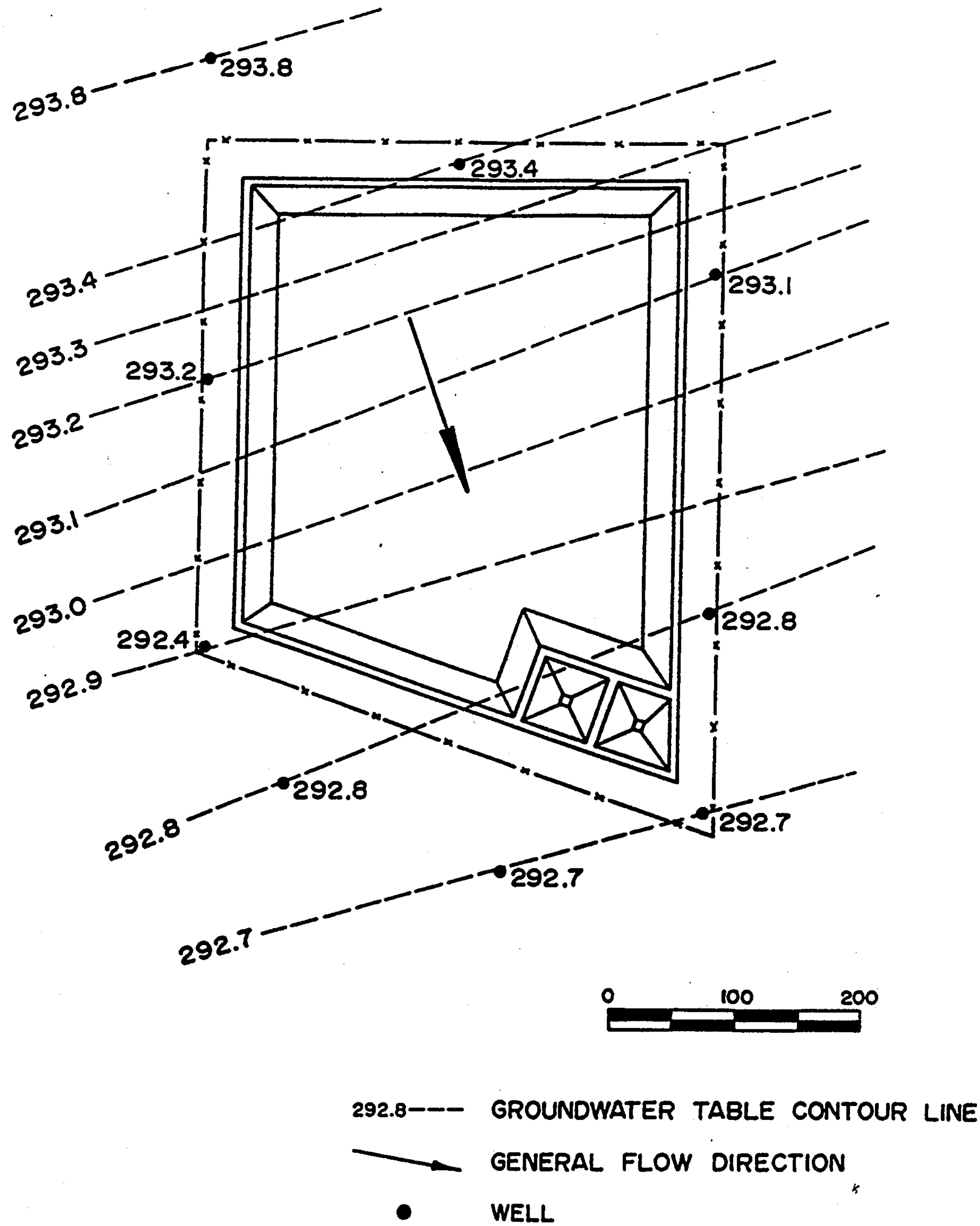

FIG. A11.5 EXAMPLE OF MONITOR WELL LAYOUT AROUND LAGOON survey would be able to delineate the position of the front around the pond, and could locate any areas where subsurface flow was relatively rapid and potentially more of a problem.

A11.3 INSTRUMENT DEVELOPMENT

After groundwater monitoring instruments are installed they should be properly developed. The aim of development is to clean the instrument by removal of drilling cuttings, remains of drilling mud, and finer material from the aquifers. As a result of development, formation water can enter the well more freely, thus increasing the instrument efficiency.

Development of instruments serves a number of purposes. These are:

i) correction of damages to aquifers which occurred during drilling

ii) determination of reliable hydraulic properties

iii) collection of uncontaminated water samples

The most common methods used in well and piezometer development are:

- mechanical surging
- air lift using compressed air or gas
- backwashing
- high velocity jetting

Detailed descriptions of well development procedures, which can be applied to piezometers as well, are given in the hydrogeological literature (e.g. Johnson 1972; Campbell and Lehr 1977).

A11.4 MONITORING, SAMPLING AND HYDROCHEMICAL ANALYSES

After development, each instrument should be pumped out. Based on groundwater table recovery data, values of hydraulic conductivity for tested zones should be calculated (see Appendix 3). Water levels should be monitored frequently at all observation points during the instrument installation program until full groundwater table recovery occurs. After this early stage, water levels should be monitored on a monthly basis for the first six months, and on a quarterly basis thereafter. Field measurements of water quality (i.e. pH, electric conductivity and temperature) should be made every month, and water samples should be collected for analysis from piezometers on a quarterly basis unless field measurements show significant change.

Groundwater sampling techniques may vary depending on the instrument diameter and depth to the groundwater table. In shallow wells, samples may be collected using a simple bailer, argon (gas) driven pumps placed down the piezometer tube, peristaltic pumps, or other similar devices.

Sampling should be performed in a manner which excludes potential introduction of contaminant into the instrument. Collected samples should be preserved, stored in a dark and cool place and sent as soon as practical to a laboratory for analysis. Procedures of samples, preservation, storage and testing should follow the standard methods of Alberta Environment.

The routine quarterly analysis should include direct indicators of groundwater contamination (e.g. total nitrates, phospate, total coliform). When traces of contamination are detected, more complete analysis should be done. Recommended scope of the analysis is given in Table 3.4. Depending on chemical characteristics of lagoon sludge the analysis may have to include trace metals (e.g. mercury, lead, arsenic, boron, etc.), methylene blue active substances (MBAS), phenolic substances, biocides, organics (carbon chloroform and alcohol extractibles, CCE & CAE), biological oxygen demand, etc. Decisions regarding the range of analysis should be made based on local conditions.

APPENDIX 12
PROPERTIES OF MUNICIPAL WASTEWATERS

A12.1 RAW MUNICIPAL WASTEWATER

A12.1.1 Introduction

It is difficult to make general statements about the constituents and properties of municipal wastewater as received at waste treatment facilities. Many factors contribute to the sewage properties, including the size of the community served by the treatment plant and the types of industries in the community. Seasonal fluctuations in wastewater constituents and concentrations are important. Metcalf and Eddy (1979) give an excellent discussion of seasonal and hourly variation in sewage quantities which may be expected from various residential, commercial, institutional and industrial sources.

The composition of typcial untreated municipal wastewater is shown in Table A12.1 (from Metcalf and Eddy 1979).

A12.1.2 Indicators of Wastewater Quality

A12.1.2.1 Solids

The major constituent of raw sewage is water, comprising more than 99% of the influent wastewater at the treatment plant. A high level of impurities greatly diversified in their nature exist in raw wastewater. However, only a few substances exist at a measurable level (Kugelman 1976). Many of these impurities are solids, defined as "all matter which remains as residue after evaporation of wastewater at 103 - 105°C" (Metcalf and Eddy 1979).

TABLE A12.1 TYPICAL COMPOSITION OF UNTREATED DOMESTIC WASTEWATER (AFTER METCALF AND EDDY, 1979)

(All values except settleable solids are expressed in mg/L)[a]

Constituent	Concentration		
	Strong	Medium	Weak
Solids, total:	1200	720	350
Dissolved, total	850	500	250
Fixed	525	300	145
Volatile	325	200	105
Suspended, total	350	220	100
Fixed	75	55	20
Volatile	275	.165	80
Settleable solids, mL/L	20	10	5
Biochemical oxygen demand, 5-day, 20°C (BOD_5, 20°C)	400	220	110
Total organic carbon (TOC)	290	160	80
Chemical oxygen demand (COD)	1000	500	250
Nitrogen (total as N):	85	40	20
Organic	35	15	8
Free ammonia	50	25	12
Nitrites	0	0	0
Nitrates	0	0	0
Phosphorus (total as P):	15	8	4
Organic	5	3	1
Inorganic	10	5	3
Chlorides[b]	100	50	30
Alkalinity (as $CaCO_3$)[b]	200	100	50
Grease	150	100	50

[a] mg/L = g/m^3.
[b] Values should be increased by amount in domestic water supply.
Note: 1.8(°C) + 32 = °F.

The total solids in wastewater may be classified into several catagories. As shown in Fig. A12.1, most of the solids in a medium-strength wastewater may be described as 'filterable' (those solids of size   m) or 'suspended'. The filterable solids are further divided into 'colloidal' (particles of size 1nm to 1 m which cannot be removed by settling) and 'dissolved' fractions. Suspended solids are either settleable or non-settleable. Settleable solids give an estimate of the quantity of sludge which will be produced by sedimentation. These fractions may be classified as either 'organic' (that portion which will volatize at 600°C) or 'mineral'.

A12.1.2.2 Organic Content

A large portion of the solids that are present in wastewater are organic in nature. Organics are typically composed of oxygen, carbon, and hydrogen, with traces of nitrogen, phosphorous, sulphur and iron. These compounds include proteins (10-60%), fats and oils (10-20%), urea and carbohydrates (25-50%), as well as synthetic organic chemicals such as phenols and pesticides.

The total organic carbon (TOC) test is applicable to wastewater with small organic contents. The amount of carbon dioxide that is produced by oxidizing organic carbon (in the presence of a catalyst) is measured. The test may be performed rapidly and has gained wide acceptance in recent years (Kugelman 1976). The measured TOC may be less than the (actual) TOC in the sample, because some compounds cannot be oxided by this process. The measured value of TOC may correlate to BOD_5, with the ratio of BOD_5 to TOC lying in the range of 1 to 1.6.

The total oxygen demand (TOD) is measured as the amount of oxygen in the nitrogen carrier gas treated by combustion of organic and inorganic substances in a platinum - catalysed chamber. The test result may be obtained rapidly and correlated with the COD value.

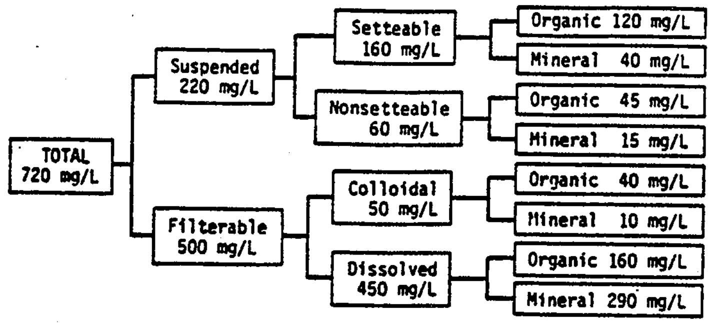

FIG. A12.1 CLASSIFICATION OF SOLIDS FOUND IN MEDIUM STRENGTH WASTEWATER (AFTER METCALF AND EDDY 1979)

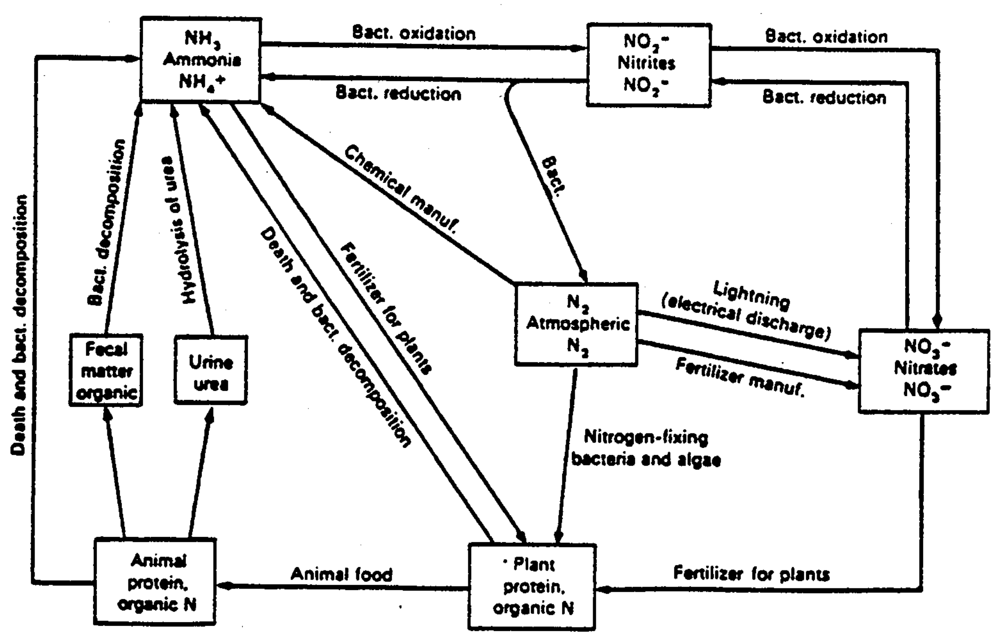

FIG. A12.2 NITROGEN CYCLE (AFTER METCALF AND EDDY 1979)

The theoretical oxgyen demand (ThOD) is calculated using the chemical formulae of each constituent of the wastewater.

A12.1.2.3 Inorganic Matter

Inorganic substances are also important in wastewater treatment. The hydrogen ion concentration, expressed as pH, is a crucial parameter because a highly acidic wastewater will hinder many of the biological processes necessary for waste treatment and may be harmful to ground and surface waters. The pH range of raw sewage is typically 7 to 8 (Bolton and Klein 1971). The concentration of alkalinity in wastewater is critical to the effectiveness of chemical treatments and exists in raw sewage at approximately 400 mg/L. The presence of chloride is expressed in terms of $CaCO_3$ (Bolton and Klein 1971).

Many inorganic elements, such as arsenic, fluorides, selenium, and boron, may be toxic in various concentrations.

Sulfur occurs naturally in most waters, and is required in the synthesis of protein. Sulfates may be reduced to sulfides and H_2S gas which is toxic to most higher forms of life and is corrosive to gas pipelines.

Nitrogen is an important element in wastewater and exists in many forms during the waste treatment process (see Fig. A12.2). In raw sewage, nitrogen varies in concentrations from 15-50 mg/L, of which 60 per cent exists as ammonia or amonium ion (NH^{+}_{4}) and the remainder as organic nitrogen. Nitrites and nitrates account for less than one per cent of the total nitrogen content in raw wastewater.

Nitrogen undergoes a series of transformations during the waste treatment process. Ammonification transforms organic nitrogen to ammonia or ammonium; however, much of this conversion occurs before the sewage reaches the treatment facilities. In a two-step process known as nitrification, ammonia is oxidized first to nitrite (NO_2^-) by the bacteria Nitrosomonas, and then to nitrate (NO_3^-) by Nitrobacter. Nitrification occurs under aerobic conditions, and the chemical state in the treatment pond must favour both organisms. Most of the nitrogen released to the environment from waste treatment facilities is in the form of either ammonium or nitrate.

Nitrates and ammonium, if released to the environment in sufficient quantity, may have a number of serious effects. Both cause the biostimulation of surface waters, known as 'eutrophication' (an excess grown of algae), and nitrate may cause methomoglobinemia, a sometimes fatal blood disorder in infants. (Many jurisdictions require that nitrate levels in drinking water be kept below 10 mg/L to prevent methomoglobinemia). Under the right conditions, nitrate may be reduced to ammonia (Fig. A12.2) which is toxic to fish life, and lessens the efficiency of chlorine disinfection or drinking water.

The concentration of phosphorous compounds in waste treatment facilities is also of major concern. Phosphorous ranges in concentrations from 6 to 20 mg/L in raw domestic wastewater (Metcalf and Eddy 1979), although it has decreased in recent years (Thomas and Law 1977). Thirty to fifty per cent of this phosphorous originates from human waste (urine and feces), 50 - 70% from detergents and 2 - 70% from corrosion and scale control chemicals. Industrial sources of phosphorous include potato processing, fertilizer manufacturing, animal wastes, and metal finishing wastes. In wastewaters, phosphorous exists in three forms: orthophosphate, polyphosphates, organic P(2-5 mg/L) and inorganic P (4-15 mg/L) (Metcalf and Eddy 1979). Orthophosphates may be used directly in biological metabolism. Polyphosphates (polymers of phosphoric acid) undergo hydrolysis and revert to orthophosphates.

A number of indicators have been devised to measure the organic content of wastewaters. The most widely used indicator is the biochemical oxygen demand (BOD). This test measures the dissolved oxygen required by micro- organisms for the biochemical oxidation of organic matter over a specific period, commonly taken as 5 days. The results of the BOD_5 test allows a designer to determine the approximate amount of oxygen required to biologically stabilize organic matter, and to estimate the required size of treatment facilities. BOD_5 also serves as measurement of quality of treatment processes. A thorough discussion of BOD_5 and the analysis of its results is given by Metcalf and Eddy (1979).

Although the BOD_5 test is widely accepted as an indication of wastewater organic content, it has a number of limitations. A high concentration of active, acclimated seed bacteria is required. Pretreatment of wastewater samples is required if toxic wastes are present, and the effects of nitrifying bacteria must be reduced. The test measures only the biodegradeable portion of the organic waste. The test is not stoichiometrically valid after the soluble organics have been used. The five day period which is commonly used in the test does not necessarily correspond to the time required for the decomposition of the organic matter in the sample.

The limitations of the BOD_5 test have prompted the development of other indicators. Chemical oxygen demand (COD) may be used in wastewaters with toxic compounds, and the test requires only a short time period (approx. 3 hours). More compounds in wastewater can be chemically oxidized than biochemically oxidized. The value of COD is usually higher than BOD_5 for the same sample, although COD may be correlated with BOD_5 (the ratio of BOD_5/COD is usually in the range of 0.4 - 0.8).

A12.1.2.4 Trace Metals

Many metals are necessary for growth of biological life, but are toxic in excessive quantities. The effects of trace metals on the quality of groundwater and drinking water are still under study (Freeze and Cherry 1979). Thomas and Law (1977) reported that, generally, only iron occurs in large quantities in raw wastewaters, while other metals exist in trace amounts.

In water, metals tend to form hydrolyzed species combining with inorganic ions. These reduced forms of metals are generally soluble. Many immobilized forms of metals are included in insoluble organic compounds (Sabey 1973).

A12.1.2.5 Microorganisms

The microorganisms present in wastewater may be divided into two groups: protista, which included bacteria, fungi, protozoa and algae; and viruses. Most of these organisms are found in the waste excreted by humans and animals.

Most of the bacteria excreted by humans are classified as coliforms, which are generally harmless to humans and are beneficial in waste treatment. Coliforms help degrade organic matter in waste treatment (e.g., Nitrosomonas and Nitrobacter aid in the nitrification of ammonia to nitrate). The presence of coliforms in ground and surface waters serves as an indication of wastewater contamination and the possible presence of disease-carrying organisms (pathogens). A summary of pathogenic organisms and their sources and effects is presented in Table A12.2. Reports of coliform counts in raw sewage vary widely, ranging from 350,000 to 50,000,000 per 100 ml of wastewater.

A12.1.3 Concentration of Raw Wastewater Constituents

Metcalf and Eddy (1979) summarized published data on the concentrations of the various constituents of raw wastewaters (Table A12.1). Reports

TABLE A12.2 PATHOGENIC ORGANISMS COMMONLY FOUND IN WASTEWATER (FROM METCALF AND EDDY, 1979)

Organism	Disease	Remarks
Ascaris spp., *Enterobius* spp.	Nematode worms	Danger to man from wastewater effluents and dried sludge used as fertilizer
Bacillus anthracis	Anthrax	Found in wastewater. Spores are resistant to treatment
Brucella spp.	Brucellosis. Malta fever in man. Contagious abortion in sheep, goats, and cattle	Normally transmitted by infected milk or by contact. Wastewater is also suspected
Entamoeba histolytica	Dysentery	Spread by contaminated waters and sludge used as fertilizer. Common in hot climates
Leptospira icterohaemorrhagiae	Leptospirosis (Weil's disease)	Carried by sewer rats
Mycobacterium tuberculosis	Tuberculosis	Isolated from wastewater and polluted streams. Wastewater is a possible mode of transmission. Care must be taken with wastewater and sludge from sanatoriums
Salmonella paratyphi	Paratyphoid fever	Common in wastewater and effluents in times of epidemics
Salmonella typhi	Typhoid fever	Common in wastewater and effluents in times of epidemics
Salmonella spp.	Food poisoning	Common in wastewater and effluents
Schistosoma spp.	Schistosomiasis	Probably killed by efficient wastewater treatment
Shigella spp.	Bacillary dysentery	Polluted waters are main source of infection
Taenia spp.	Tapeworms	Eggs very resistant, present in wastewater sludge and wastewater effluents. Danger to cattle on land irrigated or land manured with sludge
Vibrio cholerae	Cholera	Transmitted by wastewater and polluted waters
Virus	Poliomyelitis, hepatitis	Exact mode of transmission not yet known. Found in effluents from biological wastewater-treatment plants

on the constituents of raw sewage produced by small communities shown that concentrations fall in the medium-weak range, but exceptions are common depending on the source of the waste.

A12.2 PRIMARY TREATMENT IN ANAEROBIC PONDS

Primary treatment is defined as the physical operations to remove settleable and floating solids in wastewater (Metcalf and Eddy 1979). In waste stabilization pond systems primary treatment commonly occurs in the first ponds. Solids, which sometimes are passed through a preliminary screening or grit removal, settle to the bottom of the pond for degradation under anaerobic conditions.

There is not a great amount of data on the characteristics of primary treatment pond effluents. Due to the wide variation in primary treatment ponds, many reports give 'average' values combining primary treatments. Characteristics of sedimentation pond effluents as reported by various authors are summarized in Table A12.3, while the effectiveness of sedimentation treatment is shown in Table A12.4.

Primary sedimentation ponds appear to be effective in reducing the concentration of total and suspended solids, biochemical oxygen demand and organic nitrogen, thus reducing these loadings on the secondary treatment process. They are only moderately effective, however, in reducing the concentration of trace metals (Thomas and Law 1977).

A12.3 CHARACTERISTICS OF OXIDATION POND EFFLUENTS

An oxidation pond is defined as 'a relatively shallow body of water contained in an earthen basin of controlled shape and design for the purpose of treating wastewater' (Metcalf and Eddy, 1979). Oxidation ponds are beneficial for small communities in that they are inexpensive

TABLE A12.3 PROPERTIES OF SEDIMENTATION POND EFFLUENTS

	Miner and Hazen (1977)	Thomas and Law (1977)		Kugelman (1976)	Fischer (1976)
Solids:		range	aver.		
Total Dissolved	1402	200-1500	500	-	-
Suspended	-	50-150	100	100	95 ppm
BOD_5	152	65-200	135	100	143 ppm
COD		150-750	335	220	-
Nitrogen:					
total (mg/ℓ)	37	10-60	40	-	-
organic	-	-	-	-	-
ammonia	23	7-40	30	-	-
nitrate	0.3	-	<0.1	-	-
Phosphorous	11	5-17	8	9	-
Chlorides	461	-	-	-	-
Coliforms $\left(\frac{MPN}{100\ ml}\right)$	-	-	-	15 million	-

NOTE: all values in mg/L unless noted.

TABLE A12.4 EFFECTIVENESS OF SEDIMENTATION TREATMENT

	Kugelman (1976)	DeRenzo (1978)	Fischer (1976)
Settleable Solids	-	-	95%
Suspended Solids	57%	-	50%
BOD_5	66%	-	30-50%
COD	21%	-	-
Phosphorous	25%	-	-
Nitrogen:			
total	-	5-10%	-
organic	-	10-20%	-
ammonia	-	nil	-
nitrate	-	nil	-
Coliforms	70%	-	-

to construct and operate in relation to other secondary treatment processes. Wastewaters contained in oxidation ponds vary in quality from 'primary-treated' to 'effluent quality' as determined by such factors as detention time, pond depth, etc.

A number of factors affect the operation and performance of an oxidation pond for secondary treatment of sewage. Ponds may be classified with respect to the nature of biological activity taking place (i.e., aerobic, anaerobic, or facultative). Primary treatment in anaerobic ponds is often used in series with anaerobic/aerobic systems in order to provide a more complete waste treatment. Other factors affecting pond performance are the type of influent, pond overflow conditions, and the method of oxygenation. A shallow pond is necessary if wind aeration is used. The detention time of the wastewater in the pond is also a major factor affecting effluent quality. The common recommended detention period is 60 days, but some American states require detentions of 90 to 120 days. Effluents from Alberta lagoon systems are commonly discharged after 7 to 12 months detention, although larger towns, with six or more lagoon cells in series have experimented with a continuous effluent discharge during the non-winter period (Alberta Environment 1981). Typical properties of oxidation pond effluents are summarized in Table 12.5.

TABLE A12.5 PROPERTIES OF OXIDATION POND EFFLUENTS

	Thomas and Law (1977)	DeRenzo (1978)	Kugelman (1976)	Fischer (1976)
Solids:				
Total Dissolved mg/ℓ	-	-	-	-
Suspended mg/ℓ	83 (facultative)	-	26	70-75% removed
	66 (aerated)	-	-	-
BOD_5 mg/ℓ	40	-	30	85-95% removed
COD mg/ℓ	-	-	40	-
Nitrogen: (mg/ℓ)				
total	-	20-90% removed	-	-
organic	-	15-25% removed	-	-
ammonia	-	<10% removed	-	-
nitrate	-	-	-	-
Phosphorous mg/ℓ	-	-	6	-

APPENDIX 13
ATTENUATION OF WASTEWATER CONTAMINANTS IN GROUNDWATER SYSTEM

A13.1 CONTAMINANT TRANSPORT MECHANISMS

An understanding of contaminant transport mechanisms in the groundwater system is essential in order to properly evaluate the pollution potential of a wastewater impoundment. Contaminants may be defined as any constituent of the seeping wastewater (solute or solvent) that are introduced to the groundwater by other than natural means. Pollutants may be defined as those contaminants whose concentrations have exceeded arbitrary limits (e.g. drinking water standards). The pollution potential of an impoundment depends on the initial concentrations of contaminants in the pond (C), the rate of seepage through the liner system, the attenuation mechanisms that act to renovate the groundwater, and the distance of resources from the impoundment.

Contaminants may be transported through the soil/bedrock formations by three basic mechanisms. The most common and simplest is advection, whereby the groundwater flow carries non-reacting solutes and mixed solvents through the sediments (porous medium) at a rate equal to the groundwater velocity. If the groundwater velocity were uniform throughout the porous medium and no other mechanisms affected contaminant transport, then the break-through curve[1] for a monitor point somewhere outside the pond would be a step function, as shown in Fig. A13.1. The time required for the seepage plume containing the contaminated water to reach the monitor point is equal to the distance divided by the seepage velocity v_s (i.e. the discharge velocity v divided by the porosity (n) of the medium).

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[1] A break-through curve is a plot of contaminant concentration versus time for a particular point in the porous medium.

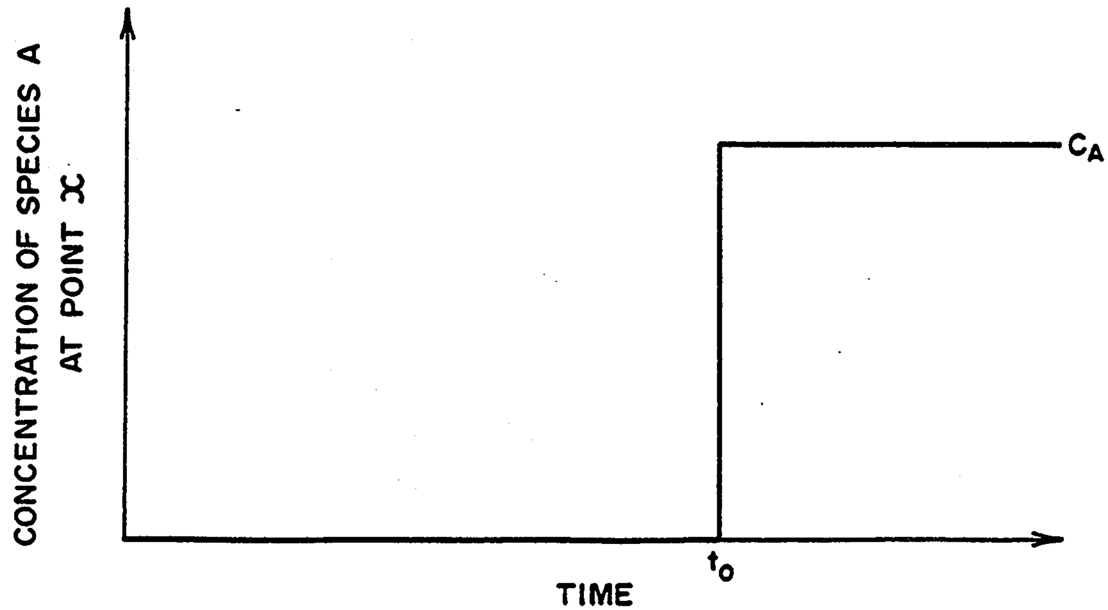

FIG. A13.1 BREAK-THROUGH CURVE FOR PISTON FLOW (ADVECTION TRANSPORT)

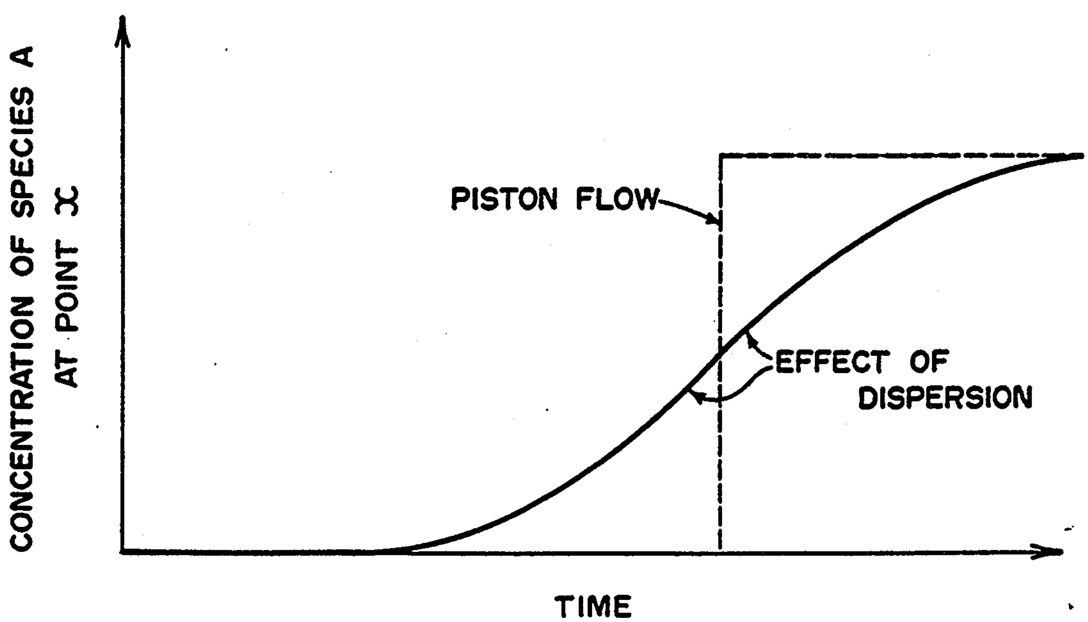

FIG. A13.2 BREAK-THROUGH CURVE FOR COUPLED ADVECTION AND DISPERSION TRANSPORT

A second contaminant transport mechanism is mechanical dispersion which results from local deviations in groundwater velocity due to variations in pore sizes and the tortuous nature of the seepage paths. These deviations cause the leading edge of the seepage plume to become dispersed, with a gradual increase in concentration toward the main body of flow. As a result, the break-through curve has the appearance shown in Fig. A13.2. Lateral dispersion (at right angles to the flow direction, or normal to the seepage plume) also occurs due to deviations in groundwater flow direction caused by the soil grains.

Molecular diffusion, the third transport mechanism, occurs when solutes move from concentration highs to concentration lows by chemical diffusion gradients (e.g. osmosis). Diffusion is generally a very slow process, and is only significant (relative to advection and dispersion) at very slow seepage velocities (Freeze and Cherry 1979). It may also be a significant mechanism contributing to seepage through liners, where seepage gradients and/or velocities are low, but concentration gradients are high (Goodall and Quigley 1977).

The combined effects of dispersion and diffusion are described by the coefficient of dispersion, D, where

(A13.1) $$D=\alpha v_s + D'$$

and where α is the dispersivity of the porous medium (due to mechanical dispersion), v_s is the seepage velocity, and D' is the coefficient of diffusion. Dispersion dominates at high seepage velocities, while diffusion is important at very slow velocities.

The coupled, one-dimensional advection - dispersion equation for contaminant concentrations versus time and distance is:

(A13.2) $$\frac{dC}{dt} = D\frac{d^2C}{dx^2} - v_s\frac{dC}{dx}$$

Where C is the source concentration of the solute and t and x represent time and distance respectively.

TABLE A13.1 MUNICIPAL SLUDGE LEACHATE ATTENUATION MECHANISMS (from Makeig 1982)

Sludge Components	*Filtration*	*Attrition**	*Microbial Metabolism*	*Plant Uptake*	*Adsorption*	*Chemical Precipitation*	*Dilution*	*Sedimentation*
Heavy metals	3	3	3	2	3	1	1	3
Pathogens	1,2	1	2	2	1,2	3	3	1
Exchangeable bases	3	3	1,2	2	1	3	2	3
Nitrogen/Nitrate (N)	3	3	1,2	1	3	3	1	3
Phosphorus	3	3	1,2	2	1	1	2	3

* due to die off and decay.
Key: 1 = Primary
2 = Significant
3 = Insignificant

This equation does not consider attenuation mechanisms and usually leads to conservative design (NRC 1982).

A13.2 ATTENUATION MECHANISMS

Attenuation may be defined as the permanent or temporary decrease in the total concentration of contaminants migrating from an impoundment, and/or a decrease in maximum concentration levels, due to physical, chemical, or biological reaction within the groundwater system. Attenuation processes are also referred to as renovation in some of the literature.

In general, soils are relatively efficient media for attenuation of municipal wastewater contaminants. A list of potential contaminants from sewage sludge and applicable attenuating mechanisms are shown on Table A13.1. Attenuation mechanisms affecting the principle municipal wastewater contaminants are discussed in the following sections.

A13.3 ATTENUATION OF SPECIFIC CONTAMINANTS IN SOIL AND GROUNDWATER

A13.3.1 Nitrates

Nitrate (NO_3^-) is the most common groundwater contaminant (Freeze and Cherry 1979) and is the most stable form of nitrogen in strongly oxidized groundwater, especially at shallow depths and above the water table. Nitrate tends to move with the groundwater with little transformation or retardation, and can travel great distances in very permeable soils. As nitrate is an anion, only a small proportion of NO_3^- is absorbed by clay minerals.

The concentration of nitrate in groundwater may be decreased by denitrification where nitrate is reduced to N_2 gas if the pH is above

6.0, or NO and N_2 at lower pH levels. These gases are not detrimental to the quality of drinking water and will come out of solution when the groundwater discharges. There are several requirements for denitrification in soil. The appropriate bacteria must be present, with sufficient organic matter as a food supply. The groundwater must have a low redox potential (i.e. a saturated environment such as exists below the water table, and in micropores in the unsaturated zone). A high pH and temperature are required (denitrification is very slow at pH levels less than 5.5, and temperatures below 10°C, ceasing at 2°C (Lance 1975)). Denitrification can be quite effective in reducing nitrates in slow moving groundwaters (i.e. soils of low permeability).

Preul (1968) studied the attenuation of contaminants near waste stabilization ponds with high percolation rates. High levels of nitrates were found as far as 90 m from the pond, but concentrations were well below drinking water standards.

A13.3.2 Ammonia

Nitrogen also enters the soil in a form of ammonia, NH_4^+. Ammonia concentrations in groundwater may be depleted by adsorption by soils with high cation exchange capacities and organic matter, and by nitirification of ammonia to nitrate in well oxidized groundwaters. The amount of ammonia that enters the soil below sewage lagoons may not be significant since much of it is converted to NO_3^- during the waste treatment process. Ammonia concentrations were found by Preul (1968) to vary from 2.5 mg/L at the edge of a lagoon to undetectable amounts at 90 m from the lagoon.

A13.3.3 Phosphorous

Phosphorous compounds are not usually harmful in drinking water (very high concentrations of phosphates are harmful to the human stomach), but can cause eutrophication of lakes. The main sources of phosphorous in groundwater are agricultural fertilizers, detergents and human wastes. Dissolved phosphorous occurs in groundwater in several forms of phosphate, but most commonly as HPO_4^{-2} and $H_2PO_4^{-1}$ under normal groundwater pH levels.

The mobility of phosphorous in groundwater is limited by adsorption (Freeze and Cherry 1979). Preul (1968) found that phosphate levels were reduced significantly in sandy and silty soils, from up to 21.5 mg/L at pond edge to less than 1.0 mg/L at 9.2 m. Phosphates were found to migrate greater distances in fine to very coarse alluvial sediments (Wilson *et al* 1973).

A13.3.4 Trace Heavy Metals

Metals tend to form hydrolyzed species in groundwater by combining with organic ions. When the total concentration of a metal is measured, each of its hydrolyzed forms must be considered (Freeze and Cherry 1979). Only iron (Fe) exists in significant concentrations in natural groundwaters; other metals occur at concentrations less than 1.0 mg/L. The total concentration of metals decreases with increasing pH of the groundwater. The solubility of some minerals in anaerobic groundwaters tend to limit metal concentrations.

It is difficult to predict the transport and attenuation behaviour of metals in groundwater flow systems because of their complex environmental chemistry. Adsorption reduces metal concentrations to low levels in the presence of clay minerals and organic matter, especially in oxidizing environments. The process of oxidation, both in lagoons and in soils above the water table, removes large quantities of metals from wastewater effluents. Relatively few instances of groundwater pollution have been reported, but where it does occur the consequences can be serious (e.g. contamination of rivers and lakes by industrial wastes).

Griffin and Shimp (1978) found that heavy metals are strongly attenuated in clayey soils and clay liners. Precipitation was the main removal mechanism for these metals, but adsorption was found to increase with the pH of the soil.

A13.3.5 Organic Substances

Natural organic substances do not usually cause concern in groundwaters; however, man-made organics are now present in increasing numbers. Common sources of man-made organic contaminants include pesticides, industrial sludge disposal, sanitary and liquid waste dumps, leakage from liquid waste storage ponds and accidental spills.

There have been relatively few investigations of the effects of man-made organics in groundwater and their attenuation in soils. No general statements can be made about their behaviour because of the diversity of their makeup. Freeze and Cherry (1979) proposed some factors which are assumed to affect the attenuation of organic substances. Several mechanisms which tend to retard the transport of organics through soils are chemical precipitations, chemical and biological degradation by bacteria, volatization in the presence of a gas phase, biological uptake and adsorption.

The behaviour of several synthetic organic compounds, such as PCB's, PBB's and HCB's, were studied by Griffin and Chian (1979) and Griffin and Chow (1980). The mobility of these compounds are strongly related to their solubility in the leaching fluids. Solubilities were found to be many times greater in landfill leachate than in distilled water and also to be strongly correlated to the presence of dissolved organics. The adsorption of PCB's, PBB's, and HCB was found to correlate to the TOC and surface area of the soils. Each of these chemicals were highly resistant to microbial degradation.

Some organic chemicals are insoluble in water, thus limiting appreciable migration. Refractory compounds are not readily degraded by bacteria. Insoluble, non-volatile refractory compounds pose the greatest environmental threat (some are very toxic at low concentrations). However, very little is known about the migration of these compounds.

A13.3.6 Micro-ogranisms

There is considerable evidence that bacteria and viruses travel only small distances in most geologic materials. Hagedorn et al (1981) reviewed many studies of bacterial and viral migration from septic tank effluents. There are relatively few reports of field studies on bacteria migration in soils because sampling techniques have yet to be perfected. Laboratory results of viral adsorption may be misleading because conditions may be too artificial in the laboratory. Several conclusions were made from this review:

1) Coliforms and other microorganisms move only a few dozen centimeters with percolating waters in unsaturated soils, but greater distances are possible under saturated conditions.

2) The degree of bacterial retention by the soil is inversely proportional to the size of the particles in the unstructured soil matrix.

3) Physical straining of organisms by soil particles is the major mechanism for limiting bacterial transport. The sedimentation of bacteria clusters also occurs under saturated conditions.

4) Adsorption is an important factor in the soil retention of bacteria, especially in soils with a large clay content.

5) For long retention periods and unsaturated conditions, the die-off of microorganisms is an important factor. Because groundwater velocities are usually slow and the soil environment may not be conducive to bacterial reproduction, most bacteria perish before significant migration can occur. However, in very permeable soils and fractured bedrock where groundwater velocities are high, bacteria and viruses may be transported great distances, creating possible health hazards.

Other factors that affect bacteria movement in soils are the moisture content, pH, and temperature of the soil, exposure to ultraviolet radiation at the soil surface, and the rate of sewage effluent infiltration into the soil (Bilton 1975).

Viruses are most often controlled in their migration by adsorption. Because of their small size, straining of viruses by the soil is not a significant factor. The adsorption of viruses is strongly influenced by pH, the presence of cations and ionizable groups on the virus, salt concentration, and the presence of organic matter. The mechanism of viral adsorption is not known, although it has been found to be slower in natural clays than in laboratory made clays. Because of the wide variety of viruses, their adsorption behaviour may not be similar under identical environmental conditions. Viruses are not decomposed by the soil and may be released into the groundwater after a number of years.

A notable field study of viral transport in soils was conducted at a wastewater irrigaion site in Kerrville, Texas by Moore et al (1981). The soils at the site consisted of well-drained loams and clays, with permeabilities from 0.4 to 1.4 x 10^{-5} m/sec, pH from 7.7 to 9.0, and cation exchange capabilities from 25 to 50 meq per 100 g. Viral and bacterial movement was measured by lysimeters in saturated zones and by monitor wells at depth (the water table was at 23 m depth). The presence of viruses in lysimeter samples, but not in the monitor wells, suggests that the depth to groundwater is an important factor in the selection of irrigation sites, and that viral attenuation was not sufficient over a depth of 1.37 m.

APPENDIX 14
WASTEWATER - LINER COMPATIBILITY

A.14.1 GENERAL

Tables summarizing the results of various wastewater - liner material compatibility tests are presented in this Appendix. In general, there is a lack of detailed information concerning wastewater - liner compatibility, but the information contained herein will help indicate when liners are likely to be suitable from a chemical resistance standpoint, and when compatibility tests should be run (see Section A14.2). Particular attention should be paid to wastewater - liner compatibility when industrial or other liquids alter the characteristics of municipal wastewater beyond the normal range of properties (see Table A14.1).

It should be noted that the data on the following tables are based on laboratory tests of varying quality, procedure, and duration, as well as on observed performance in ponds.

Considerable information on polymeric liner - waste compatibility is available from polymeric liner manufacturers and installers. Much of this information may be based on short term immersion tests.

A14.2 LINER - WASTE COMPATIBILITY TEST METHODS

A14.2.1 Immersion Tests

One of the most common and simplest compatibility test procedures is to immerse the liner material in the waste liquid for various periods of time and to observe the effects on material properties. Matrecon (1980) provides detailed procedures for immersion tests on polymeric membrane liners. They recommend exposure times of 0.5, 1, 2, and 4 months, based on extensive long term tests. Absorption of waste liquids or loss of weight are apparently good indicators of liner resistance to a particular waste. Results for landfill leachate and membranes are shown on Fig. A14.1 and A14.2. A simple test with clays is to perform Atterberg limit tests using water and the waste fluid to mould the clay. Significant differences in limit values indicates the potential for structural changes in the clay liner when the wastewater is introduced. This test is more suitable for polar or ionic liquids, and organic fluids that may result in cation exchange or displacement of adsorbed water on clay mineral surfaces, than for acids or bases that cause dissolution with time.

Matrecon (1980) outlines procedures for two variations of the immersion test on polymeric membranes. The first is described as a pouch test where the membrane is formed into a pouch, filled with the waste fluid, and then sealed. The sealed pouch is in turn sealed into a larger polybutylene bag filled with deionized water. The inner membrane pouch is checked for weight loss, and the outer deionized water is tested for pH and conductivity. Measurements are made frequently over a period of several months, until failure of the material or definite trends are observed. The test results indicate the permeability of the liner to the solvent (weight loss) and the rate of mass transfer due to osmosis (conductivity).

The second variation on the immersion test is the tub test where tubs lined with membrane liners are partially filled with the waste liquid, and then are exposed to sun, weather, and the like. The liquid level is allowed to fluctuate, and the liners are visually inspected for cracking, separation of seams, etc. Material properties are measured at the end of the designated exposure period, which should be as long as possible.

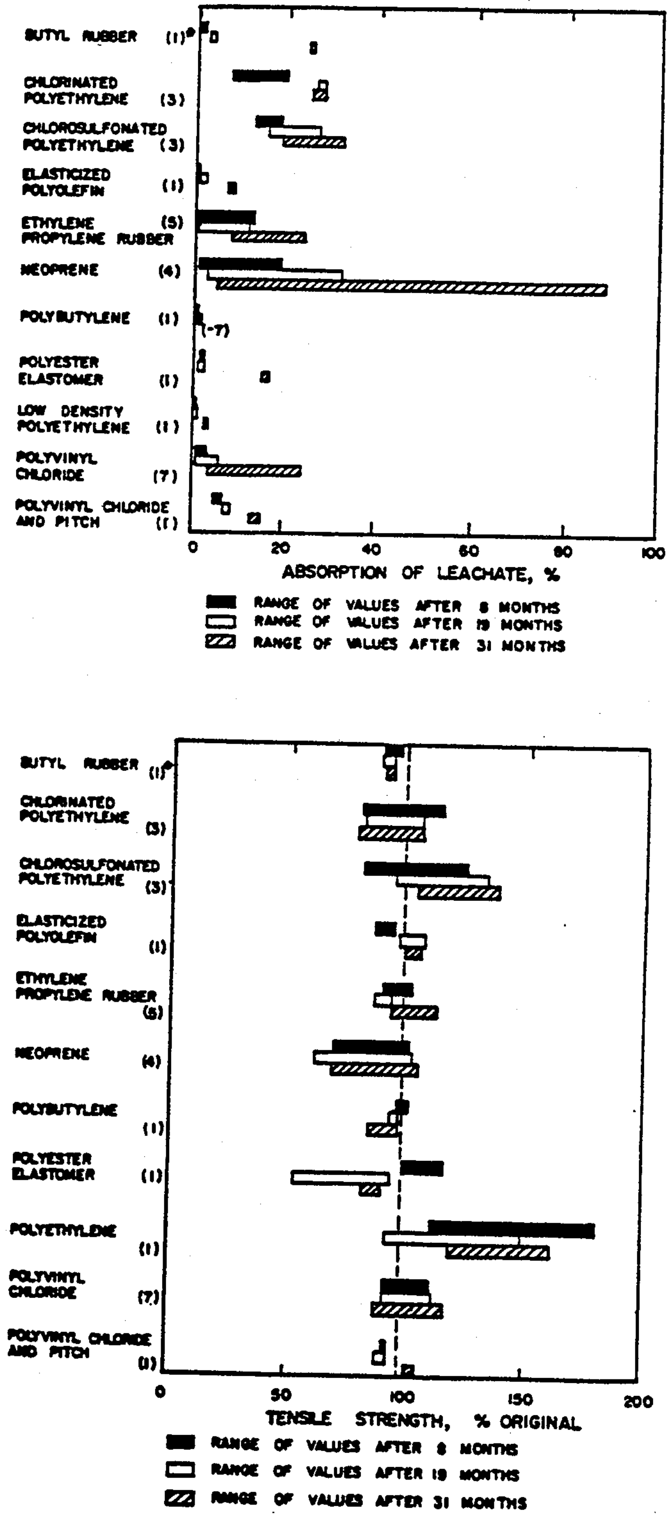

FIG. A14.1 EFFECTS OF LANDFILL LEACHATE ON VARIOUS MEMBRANE LINER MATERIALS (AFTER MATRECON 1980, 1981)

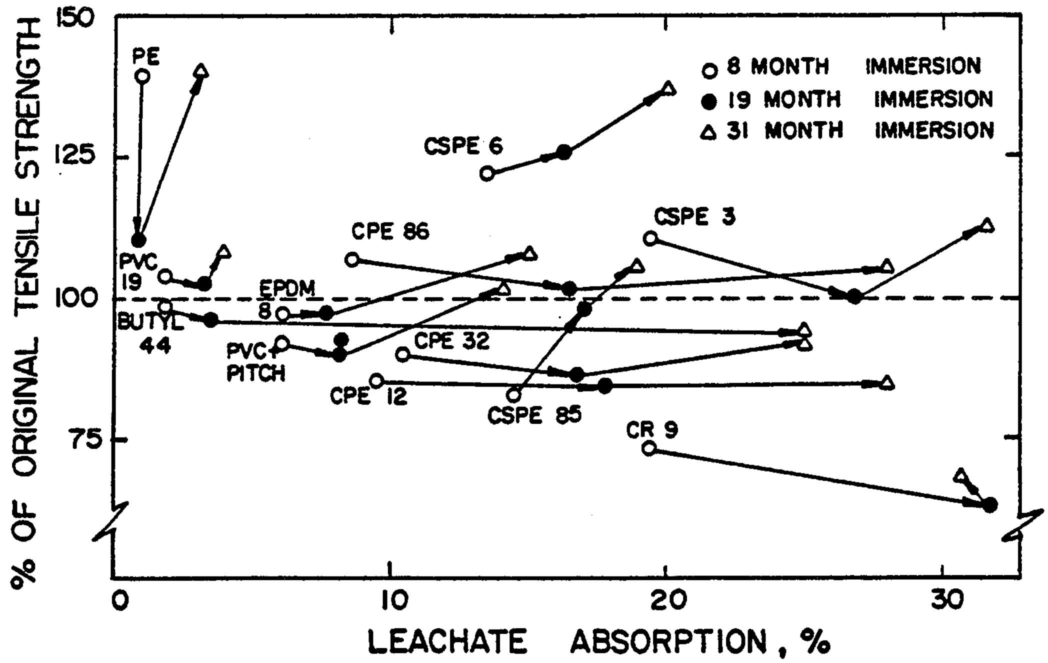

FIG. A14.2 LANDFILL LEACHATE ABSORPTION VERSUS TENSILE STRENGTH OF MEMBRANE LINERS (MATRECON 1980)

A14.2.2 Hydraulic Conductivity Tests Using Waste Liquids

One of the most promising ways to test liner - waste compatibility in porous materials appears to be hydraulic conductivity testing using the waste fluid as a permeant. Test cell materials must be resistant to corrosion when in contact with the waste fluid (e.g. teflon coated, teflon gaskets), and care must be taken when handling hazardous materials (e.g. vent hoods, protective clothing).

Test procedures are described by Matrecon (1980), Anderson and Brown (1981), and Gordon and Forrest (1981). The tests are usually conducted by first permeating the sample with a standard water, and then introducing the waste fluid. Variations in hydraulic conductivity are observed with time. Anderson et al (1982) suggest that at least 1 to 2 pore volumes (volume of voids in sample) of the waste liquid be passed through the sample. Acids and strong bases may initially cause a decrease in hydraulic conductivity due to plugging of pore spaces with small particles freed from the clay minerals during dissolution. Eventually, however, the hydraulic conductivity is likely to increase due to the formation of channels. The change in hydraulic conductivity should be compared with expected changes due to change in liquid density, viscosity, and temperature.

TABLE A14.1 LINER/INDUSTRIAL WASTE COMPATIBILITIES[1]

Liner Material	Caustic Petroleum Sludge	Acidic Steel-Pickling Waste	Electroplating Sludge	Toxic Pesticide Formulations	Oily Refinery Sludge	Toxic Pharmaceutical Waste	Rubber and Plastic
	INDUSTRIAL WASTE[2]						
Polyvinyl chloride (oil resistant)	G	F	F	G	G	G	G
Polyethylene	G	F	F	G	F	G	G
Polypropylene	G	G	G	G	G	G	G
Butyl Rubber	G	G	G	F	P	F	G
Chlorinated Poly-ethylene	G	F	F	F	P	F	G
Ethylene Propyl-ene Rubber	G	G	G	F	P	F	G
Hypalon[3]	G	G	G	F	P	F	G
Asphalt Concrete	F	F	F	F	P	F	G
Soil Cement	F	P	P	G	G	G	G
Soil Asphalt	F	P	P	F	P	F	G
Asphalt Membranes	F	F	F	F	P	F	G
Soil Bentonite (Saline Seal)[4]	P	P	P	G	G	G	G
Compacted Clays	P	P	P	G	G	G	G

[1] From Stewart (1978).

[2] P = poor, F = fair, G = good.

[3] Registered trademark of Du Pont.

[4] Registered trademark of American Colloid Company.

TABLE A14.2 EFFECTS OF MSW LEACHATE ON ADMIX LINER MATERIALS[1]

LINER MATERIAL	EFFECT OF LEACHATE AFTER 56 MONTHS EXPOSURE
Hydraulic Asphalt Concrete, 61 mm thick e_o = 2.9%	k decreased from 3.3×10^{-11} m/s to 10^{-11} m/s q_u = 6% original strength
Paving Asphalt Concrete 56 mm thick e_o = ?	k decreased from 1.2×10^{-10} m/s to 10^{-11} m/s q_u = 9% original strength
Soil Asphalt 100 mm thick e_o = ?	k decreased from 1.7×10^{-5} m/s to about 10^{-9} m/s (deformed specimen) q_u = 2% original strength
Soil Cement 114 mm thick e_o = ?	k decreased after 1 year from 1.5×10^{-8} m/s to 1.5×10^{-10} - 4.0×10^{-9} m/s, then icreased to about 10^{-7} m/s. Discrete leaks detected in some cores. q_u = 62% original strength
Bituminous Seal Catalytically blown 8 mm thick	Became very soft and "cheesy" in some areas, no seepage observed
Fabric and Asphalt Emulsion 8 mm thick	Slight swelling but no seepage observed.

[1] From data presented by Fong and Haxo (1981).

TABLE A14.3 EFFECTS OF INDUSTRIAL WASTES ON SOIL AND ADMIX LINERS [1]

LINER MATERIAL	ACIDIC (HNO_3,HF,HOAC)	ALKALINE (SPENT CAUSTIC)	LEAD (LOW LEAD GAS WASHING)	OILY (AROMATIC OIL)	OILY (OIL POND 104)	PESTICIDE (WEED KILLER)
Compacted fine-grain soil 305 mm thick	NOT TESTED	measureable rate of seepage V_s - 10^{-10} to 10^{-9} m/s, waste penetrated 3-5 cm after 30 months (a)		$k = 1.8 \times 10^{-10}$ (a)* $k = 2.4 \times 10^{-10}$ $k = 2.6 \times 10^{-10}$ tests on soil after 30 months		(a)*
Soil Cement 100 mm thick	NOT TESTED	no measureable seepage after thirty months				
Modified Bentonite and Sand (2 Types) 127 mm thick	NOT TESTED	measureable seepage after thirty months, channelling of waste into bentonite (b)			"failed" waste seepage through liner	(b)**
Hydraulic Asphalt Concrete 64 mm thick	"failed"	"satisfactory"	waste stains below liner asphalt mushy	NOT TESTED	NOT TESTED	"satisfactory"
Spray-on Asphalt and Fabric 8 mm thick	NOT TESTED	"satisfactory"	waste stains below liner	NOT TESTED	NOT TESTED	"satisfactory"

* same as (a)
** same as (b)

[1] From data presented in Haxo (1981).

TABLE A14.4 ABSORPTION OF WASTE BY POLYMERIC MEMBRANE ON IMMERSION IN SELECTED WASTES [1]
(Data in Weight Percent)

Polymer [2]	Type [3]	WASTE AND IMMERSION TIME IN DAYS (d)							
		Pesticide 807 d	HNO_3 751 d	HF 761 d	Spent caustic 780 d	Lead 786 d	Oil 104 752 d	Aromatic oil 761 d	Weed oil 809 d
Butyl	VZ	1.6	3.8	3.7	0.8	28.7	103.9	31.2	64.2
CPE	TP	12.7	19.9	12.9	1.1	118.9	36.9	226.4	-- [4]
CSPE	TP,R	17.3	10.0	9.0	4.3	120.7	49.5	105.2	368.4
CSPE	TP	15.7	10.9	7.7	3.3	116.2	55.0	110.5	347.5
ELPO	TP	0.5	7.6	1.1	0.6	17.0	28.9	29.4	38.1
EPDM	TP,R	4.5	4.2	3.1	1.6	24.8	26.5	19.8	84.4
EPDM	VZ	20.4	50.9	23.9	1.3	34.7	84.7	34.2	76.2
Neoprene	VZ	11.4	17.4	12.0	1.5	59.1	26.3	142.6	89.3
Polyester	TP	4.2	6.4	2.0	1.5	7.4	8.5	16.6	14.7
PVC	TP	5.1	22.1	18.1	0.4	-1.5	-10.4	18.5	15.3
PVC	TP	1.0	-6.1	0.9	-0.9	7.4	-0.5	28.9	24.7
PVC	TP	1.6	28.2	14.3	1.1	-5.2	-9.8	14.1	25.2

[1] From Matrecon (1980) and Haxo (1981)

[2] See Table 2 for full name of polymer.

[3] Vulcanized (VZ), thermoplastic (TP), fabric reinforced (R)

[4] Specimen was lost; some indication that is dissolved in the waste.