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GEOTECHNICAL ASPECTS OF LINED RESERVOIRS

BY

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SYNOPSIS

Changing awareness of the environment has lead to a rapid increase in the number of lined reservoirs being built. This report examines the types of liners available and their suitability for various applications. Failure modes, particularly of a geotechnical nature, are investigated critically and suitable design procedures suggested. Finally, a number of failures are examined.

LINED RESERVOIRS

1.1 Introduction

Lined reservoirs and canals have been used since the earliest times as a means of reducing seepage losses through the foundation material. It is believed that clay liners were used for canals in Egypt up to 2500 years BC, and both bitumen and cement lined cisterns still exist dating back to 800 BC (Singer et al 1954).

Water supply still provides one of the major demands for lined reservoirs. Local holding and settling reservoirs are commonly used in water distribution schemes. Pumped storage schemes require reservoirs at elevated locations, generally high up on hills. These locations are usually unsuitable for retaining water due to high permeabilities and so lined reservoirs are often employed.

In recent years the effect on the environment of uncontrolled discharge into the ground water has been increasingly recognised. Excess discharge may cause substantial rises in the water table leading to instability in neighbouring regions, or the spoiling of agricultural land through water logging or salt deposition. Pollutants present in the discharge water can prevent the use of ground water for domestic, agricultural or industrial use. Discharge of such pollutants is commonly governed by government regulation and offenders are liable to prosecution. This has led to the development of "zero discharge" reservoirs. Typical materials stored in such reservoirs include oil, uranium mill tailings, chemical waste, sewage brine.

1.2 Liner Selection

A large number of different materials have been used successfully to line reservoirs, dams and canals. The choice of a particular liner system will be governed by the environment in which it will have to operate. The critical factors to be considered can be broadly classified as

i) Reservoir design and subgrade conditions.

ii) Material to be retained in the reservoir.

iii) Exposure to potentially damaging conditions e.g. weather, light, ice.

The material contained by the reservoir may contain a large variety of waste types and concentrates. No attempt can be made here to list the various types or to explain in detail their effect on the various liners. The more important properties of the waste such as acidity and oiliness, which may effect the particular liners are outlined in Section 1.3.

A number of liners require no cover to prevent ageing or weathering. These include most of the cement, asphalt, asphalt-cement combinations. Clay liners are subject to weathering, as outlined in Section 2.2.1. Several of the recently developed synthetic liners may be used without cover. However some synthetics deteriorate with prolonged exposure to ultraviolet light (ref Table 1.1). If other means of mechanical damage may occur, such as ice movement or seam tearing, a soil cover may be required.

The reservoir design and the subgrade conditions will to a large extent determine the type of failure modes which are likely to occur. Failure modes are outlined in Section 2 for the various categories of liner type. Some specific points to be borne in mind in selecting liners are the internal slope steepness; the compaction and preparation of subgrade material and the contact between the liner and pipes etc. which perforate the liner. Soil liners may be placed compacted on slopes of up to 3 : 1 (U.S. Bureau of Reclaimation 1963).

Unreinforced synthetic liners should be avoided on slopes greater than 1.5 : 1. Reinforced synthetic liners can be used on most stable slopes (Owen 1976).

The care with which the subgrade needs to be prepared depends primarily on the flexibility of the proposed liner. Generally, compaction and the clearing of sharp or rigid objects from the surface on which the liner will rest will decrease the risk of subgrade induced failure. These failure types are outlined in Section 2.4.1.

The cost of liners needs also to be borne in mind. The costs of the synthetic liner types has been decreasing over the last decade to make them competitive with compacted soil liners. A cost comparison from Kays (1977), is given in Figure 1.1.

1.3 Liner Types

The number of liner types to be considered when selecting to meet specific criteria seems endless. The following discussion briefly outlines the more common types of liner material, including a description of their limiting properties for reservoir waste ponds.

The failure modes for the liner types depends primarily on the rigidity of the liner material. Hence the materials will be examined in the following groups - the earth liners, flexible liners and rigid liners. These same gorupings are used to look at failure modes in Section 2. A summary of properties is given in Table 1.1.

1.3.1 The Earth Liners

Naturally occuring soils may be used directly or in combination with various admixtures to form seals for reservoirs.

a) Compacted Soils : The fine grained soils (Unified Soil Classification GW, GC, SW, SC, CL and CH) when compacted at a suitable water content can provide a sound liner. The soils should be compacted near, or preferably above the optimum moisture content. Mitchell et al (1965) show that the minimum permeability normally occurs just wet of optimum, and does not change markedly at moisture contents above that. However small changes in moisture content below optimum can change the magnitude of the soil permeability by up to two orders of magnitude. The greater the compactive effort, the lower the resulting permeability will be. The flexibility of the final soil layer also depends primarily on the compaction moisture content, as outlined in Section 2.3.2. The ability of a clay to seal a reservoir is due primarily to the fact that the clay platelets will adsorb water and swell. The expanding clay fills the soil's voids and yields permeabilities as low as 10^{-7} cm/sec. (Kays 1977). If the material stored in the reservoir contains a high cation content, exchange of the sodium ions in the clay may occur. This will reduce the swelling potential of the clay, Perhaps to such an extent as to render it useless as a sealant. Other problems with soil liners are elaborated in section 2.

b) <u>Soil Sealants</u>

Soil sealants may be natural or synthetic, and are designed to mix with existing soil and through reaction with the soil, decrease its permeability.

i) <u>Bentonite Sealant</u>:

Bentonite may be added to an existing soil either by direct application to the surface of the soil followed by harrowing in of the powder or slurry, or by washing into the soil of bentonite which has been spread over the water surface. The different sealing methods have been compared by Rollins and Dylla (1970). It was found that a layer of bentonite buried under a soil layer was most effective as this decreased the effect of the wetting and drying cycles on the seal.

The bentonite must be retained by the soil, and so this method can only be used on the finer granular soils and silts. The plastic nature of the bentonite allows "self healing" to occur by the washing in of clay particles to damaged zones. It is usual practice to renew the bentonite seal, using approximately one tenth of the original dose, each year (Kays 1977).

Sodium montmorillonite is particularly susceptible to cation exchange. The American Colloid Company has developed a number of products that resist the effect of leachants.

ii) <u>Tailings Slimes</u>:

Mill or mine tailings finer than two microns can be used to seal a reservoir in a similar manner to bentonitic clays. Permeabilities in the range 10^{-4} to 10^{-6} cm/sec have been recorded by Mittal and Morgenstern (1976).

iii) Chemical Sealants:

Chemicals have been used primarily to increase soil stability and/or bearing capacity, with a resultant lower porosity and hence permeability. Some attempts have been made to use factory by-products to seal reservoirs. These include the use of sodium rich fluid to replace calcium ions in clays and hence increase swelling potential. Other chemicals may be suitable for particular soil types.

1.3.2 Rigid Liners

There has been considerable experience and success with the rigid liner types, including steel, concrete, gunite, asphaltic concrete and soil cement. They have been used to line rockfill, and earth and rockfill dams, for irrigation canals and for reservoirs. They are ammenable to tight quality control and are subject to relatively few failure modes. However, the price of these materials and the labour content in their construction normally makes them uneconomical choices.

i) Concrete:

Reinforced or unreinforced concrete may be used as a reservoir liner. Unreinforced concrete is particularly susceptible to cracking due to subgrade movements. Cracking between panels can be minimised by using narrower panels, usually 20 - 30' width. It is not suitable for acid environments.

ii) Steel:

The main advantage of steel as a liner is its structured strength which provides additional safety to a critical installation. The cost is usually prohibitive.

iii) Gunite:

Airblown concrete placed directly on the surface to be lined gives a low permeability, brittle liner. Cracks can form readily with any differential movements. More tensile strength can be obtained by spraying the gunite over meshes of steel or fabric. Gunite is normally used in conjunction with other liner types to provide an improved seal.

iv) Asphaltic concrete:

Asphalt concrete is the conventional asphalt concrete used for road paving. The equipment and techniques for placement are readily available. It is more flexible than concrete, but still cannot tolerate appreciable movements. The field permeability can vary widely depending on placement technique, and inhomogeneities can cause appreciable leakage in thin liners. It is commonly used as an erosion protection barrier over soil liners.

v) Soil Cement:

Soil Cement is produced by mixing portland cement with insitu soil and adding water and compacting. For low permeabilities to be achieved fine grained soils must be used. The main advantages of this mix are its increased erosion resistance and improved aging properties.

1.3.3 Flexible Liners

Continuous flexible membranes, made from plastics and rubbers are widely used to provide very low leakage reservoirs. The cost of these products has been decreasing rapidly over the last decade, and they are now in common usage.

The large range of possible combinations of compounding ingredients, such as pigments, accelerators and antioxidants, together with the variation in production techniques mean that liner properties may vary considerably Generally the liner material is produced in rolls up to 1.8m in width by calendering. Reinforcement fabric may be introduced to form three or five ply sheets. Unreinforced liners may also be produced and laminated to reduce the possibility of pinholes in the finished product.

These sheets may then be joined either in the field or factory by seaming. The overall strength and effectiveness of the finished liner depends heavily on the quality of the seals. There are four common seaming techniques, and it is important to choose the correct method for a particular liner material. Dielectric or radio frequency seaming uses a high wattage radio frequency to generate heat, causing the material to become thermoplastic and flow together. Similarly heating elements may be applied to the material to form the seam. Solvents to attack the inner surfaces of the overlapping liner sheets may be applied to produce the seam. Finally, adhesives of the cold setting or two component type may be used.

The more commonly used types of materials are listed below with their more important properties.

a) <u>Plastic materials</u>

Plastic materials have been in existance longer than other materials and account for the greatest percentage of modern liner systems.

i) <u>Polyvinyl Chloride (PVC)</u>

A resin is mixed with a "plasticizer" which will vary depending upon the application and expected life of the liner. It is the loss of this plasticizer through water extraction and heat volatilization which gives PVC its poor ageing characteristic. The loss of plasticiser causes stiffening, lower tear resistance and some shrinkage. Field seaming is usually achieved using adhesives, while shop fabrication uses frequency seaming to produce wide sheets.

ii) <u>Polyethelene (PE)</u>

PE is a high molecular weight wax. No plasticiser is used to produce PE, and it is made directly into a film by an entrusion process. PE is readily shaped, has a low permeability, and is cheap. However it is very susceptible to ultraviolet light, and requires a substantial cover to protect it from ageing, and to help prevent float up of the thin material.

iii) Chlorinated Polyethelene (CPE)

Produced by Dow Chemicals, this thermoplastic liner has better flexibility and ageing characteristics than PVC or PE. It is seamed using solvents, and is produced in a reinforced form. Aromatic compounds can cause swelling and effect the seams.

iv) Hypalon:

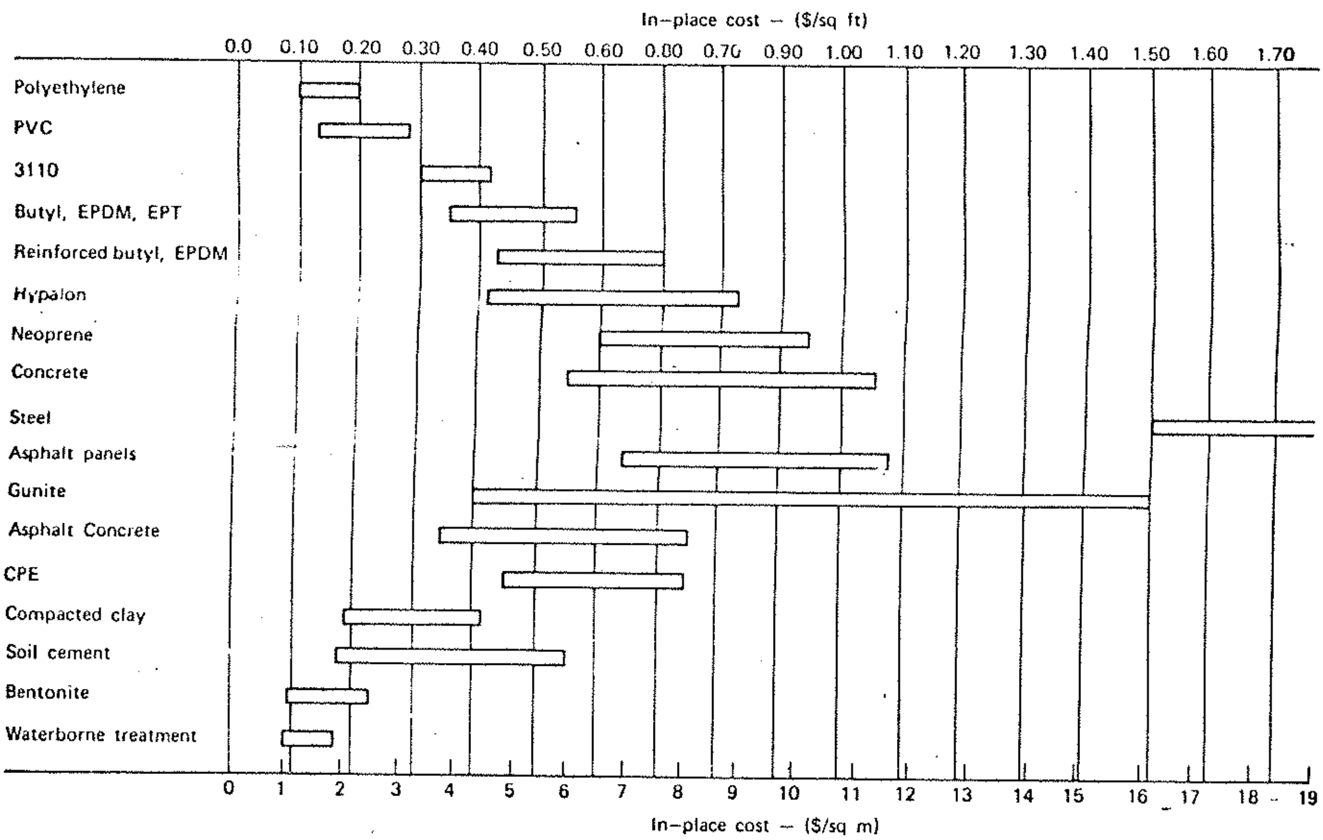
Hypalon is a chlorosulfonated polyethelene produced by DuPont. With time it slowly vulcanises to an elastomeric material. This slow curing gives it excellent weathering and ageing properties. It is also resistant to a wide variety of corrosives and chemicals. Shrinkage is a problem in unlaminated sheets, and seaming is relatively difficult.

b) Elastomers:

Synthetic rubbers, produced by vulcanising copolymers, include butyl rubber, ethylene - propylene rubber, elasticised polyolefin. Very low permeabilities are achieved in these materials because of the tight molecular structure. This structure makes the materials difficult to seam as solvents have trouble penetrating.

The elastic properties of the rubbers allow them to remain flexible over a wide range of temperatures. The original products were susceptible to ozone attack, and had poor resistance to hydrocarbons and aromatic chemicals. These deficiencies have been remedied to some extent in later products. Generally the cost is higher than the plastics.

FIGURE 1.1 : Cost comparison for liners



Cost comparison for linings in the United States.

(Kays 1979)

TABLE 1.1 : Flexible Liner Selection Guide

Property	Polyethylene Low Density	Polyethylene High Density	Polyvinyl Chloride	Chlorinated Polyethylene	Poly-propylene	Nylon	Butyl Rubber	Natural Rubber	Hypalon
Specific gravity	0.92-0.94	0.94-0.96	1.20-1.5	1.35-1.39	0.9-0.91	1.08-1.4	0.92-1.25	0.91-1.25	
Tensile strength, psi.	1,300-2,500	2,400-4,800	3,500-10,000	1,800 min.	4,000-32,000	9,000-11,000	1,000-4,000	1,000-3,500	1,000-2,000
Elongation, %	200-800	10-650	60-200	375-575	40-400	250-550			
Shore "A" hardness							15-90	20-100	55-95
Operating temperature range, °F.	−70 to 180	−70 to 240	−60 to 200	−40 to 200	−60 to 220	−60 to 380	−50 to 325	−70 to 250	−45 to 25[illegible]
Resistance to acids	P-G	G	G-E	G-E	G-E	P			G
Resistance to bases	G-E	G-E	G-E	G-E	G-E	E			G-E
Resistance to oxygenated solvents	P-G	P-G	G	P					G
Resistance to aromatic and halogenated solvents	F	F	G	P	G	G	P	P	F
Resistance to aliphatic (petroleum) solvents	P-F	F-G	G	G	G	E	P	P	G
Water vapor permeability, perm mils	3-14	1.8-2.2	3-18	0.040-0.048	0.25-1	0.09-1.0	0.15		2.0
Weatherability	P	P	G	E	P	F	G	F	E
Time to crack, hr.	900	300	No crack till 2,500 hr.	No effect to 4,000 hr.	100	1,200			
Time to chalk, hr.	No effect till 2,500 hr.	600	300	Ditto	600	No effect till 2,500 hr.			
Time to fade, hr.	300	300	100	Ditto	900	200			

NOTE: Data not shown were unavailable to the authors. P = Poor, F = Fair, G = Good, E = Excellent
ASTM test methods for various properties: specific gravity, D751; tensile strength, D97-61T; elongation, D412-61T or D882; Shore "A" hardness, D676-59T; water vapor permeability, E96-66.

(a) Properties of Commonly Used Lining Materials.

(Kumar and Hedlucka 1973)

Substance	PE	Hypalon	PVC	Butyl Rubber	Neoprene	Asphalt Panels	Asphalt Concrete	Concrete	Steel	CPE	3110
	Type of Lining										
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 protection suggested.
[2]It is recommended that immersion tests be run on any lining being considered for use in an environment where a question exists concerning its longevity. Consult the lining manufacturer or an experienced testing laboratory when in doubt.
[3]Must be a one piece lining.

(b) Liner Resistance to common pollutants

(Kays 1977)

CHAPTER 2 - GEOTECHNICAL CONSIDERATIONS

2.1 Effects of seepage from a lined reservoir.

A major concern for designers of a lined reservoir is the leakage of fluid from the storage. The leakage may occur through the intact liner as in the case of a clay lined reservoir, or through a damaged or impared "impermeable" liner.

The leakage will affect both the reservoir itself and the environment. The position of the phreatic surface in the reservoir walls, and beneath the reservoir may govern the stability of the structure. Locally, the leakage will affect the height of the ground water table, and the reservoir location in the natural hydrogeological environment will affect the dispersion and distribution of pollutants present in the leaking fluid.

2.1.1 Local Effects :

The general sequence of seepage events following filling of a reservoir is;

i) A wetting front will progress through the liner and underlying partially saturated soil.

ii) When the wetting front contacts either an impermeable stratum or the ground water table, a mound will develop and rise towards the reservoir.

iii) If the mound contacts the reservoir, saturated seepage will occur through a mound whose height is fixed by the reservoir level.

A detailed mathematical analysis of this sequence of events has been outlined by McWhorter and Nelson (1979) for the case of a tailings dam in which the depth of tailings and slime is increasing with time. Kisch (1959) examines a similar situation, but with a constant thickness of liner material. The major conclusions of these studies are that the discharge through a partially saturated liner and foundation material depends to only a very minor extent on the unsaturated characteristics of these materials. For most geometries the liner is effectively saturated after a relatively short time, and then the discharge from the reservoir is governed by this permeability and the head of fluid in the reservoir. The foundation material does not significantly effect the rate of discharge unless it is very fine with a permeability less than or equal to the discharge from the liner. This is not normally the case as if the foundation material were this impermeable, a liner would probably not be required.

Once the wetting front has reached an impermeable layer or the phreatic surface the mound begins to develop. During this stage the seepage rate will be constant and independent of the depth to the aquifer or impermeable layer. The shape of the mound can be determined using Dupuit's assumption (Harr 1962) and the time rate of development of the mound is given by McWorter and Nelson (1979). A finite element study of this has been done by Freeze (1972).

If the mound reaches the reservoir level before steady state is established, then lateral spreading of the mound will occur. The resistance to lateral spreading in the foundation material will have a significant influence on the reservoir leakage. Initially an increase in pore water pressure at the interface between the foundation and liner will cause reduced leakage. With time, leakage will decrease to a rate controlled entirely by lateral flow in the foundation. This case can be analysed using standard techniques for determination of a flow net in a soil.

2.1.2 Regional Effects :

The main concern on a regional basis is the movement of pollutants from the holding reservoir into the surrounding environment.

Pollutants move through a subsurface flow system by a complex interaction of four processes : convention, dispersion, molecular diffusion and hydrogeochemical retardation. The primary mechanism is convection, where the pollutants travel with the carrier fluid. The rate of travel will be affected by the retarding effect of hydrogeochemical interaction between the soil and the polluted fluid, and the spreading of the pollutant front by dispersion. Molecular diffusion is unimportant when compared with dispersion.

The use of tailings dams, ponds and lagoons for storage of uranimum and other long term toxic wastes which may be leaked into the hydrosphere has meant that accurate estimates of the time before these wastes reappear in wells, springs or streams have been required. The final selection of a site for long term toxic wastes will depend on a number of criteria (Williams 1979) but primarily on the hydrogeologic conditions.

Cherry et al (1973) suggest the following criteria

(a) land should be devoid of surface water

(b) the water table should be several meters below ground surface, and such that large fluctuations are unlikely.

(c) the time to reappearance of water contaminated by the site should be sufficiently great.

The time taken before the pollutant reappears at the soil surface depends primarily on the hydrogeological conditions. Freeze (1972) used computer simulation to examine the steady state regional flow through a ground water basin. The geolcgical model used is shown in Figure 2.1. Flow was considered in the unsaturated and saturated zones of the soil. The heavy arrows outline the flow paths originating from several points on the surface, representing locations of proposed waste lagoons or similar. The hydrogeological data for each site is given in Figure2.1

The flow pattern is strongly influenced by the geological configuration. Relatively minor changes in the location of the waste lagoons means significantly different flow paths and travel times since flow is concentrated in the high permeability layers. For example, the flow path originating at V is one of the shortest paths, but has one of the longest times of travel, as the flow path is entirely in the low permeability material. Paths from Y and Z are similar in length, but the time of travel from Z is approximately half that from Y, since it is predominately in the high permeability soil.

2.2 Factors affecting liner permeability - Earth Liners

Earth liners placed under controlled conditions may have their permeability reduced with time by two main processes. These are the washing of fine material from the liner by the seeping water, or the breaking of the placed structure of the soil by weathering. The latter process could be expected to act over a large area of the reservoir, whilst the former is more likely to occur at isolated points, determined primarily by preferred seepage paths.

2.2.1 Weathering :

There has been considerable work done on the weathering of rock (Ollier 1969), but relatively little has been published on the weathering of an intact soil strata. Most of the processes at work in rock weathering are significant only over periods longer than the design life of most lined reservoirs. Physical weathering of a soil liner is the major factor and is mainly due to either the over-consolidation of the upper layers by dessication, or crystal growth such as ice lensing. Chemical and biotic weathering would normally be insignificant.

2.2.2 Wetting and Drying Cycles

On exposure to the air of a compacted liner the upper layers start to dry. The negative pore water pressures set up by this dessication cause overconsolidation, and the development of tensile stresses at the surface. Cracks and fissures result leaving intact nuggets or blocks of overconsolidated material between them. As the cycle of wetting and drying continues, the soil will swell and shrink, but during swelling it will not return to its original volume and the pattern of cracks and fissures becomes well established.

This type of behaviour can be observed in surface cracking in any arid or semi-arid climate. However information on the rate of development of a system of cracks, and the consequence change in strata permeability is difficult to obtain. Kjartanson (1978) studied first time slides in a road cutting in intact clay. There was sufficient strength reduction within four years to initiate some slides, and as this strength reduction was due to the formation of a blocky structure presumably the permeability was considerably altered in the same period. Rollins and Dylla (1970) ran a series of full scale tests to determine the effectiveness of bentonite sealing of reservoirs. The seals were found to be ineffective generally after two to three cycles of wetting and drying, losing two orders of magnitude of permeability in this time. It should be noted that other effects such as ice formation and leaching which may have been at work were not separated from the effects of dessication.

This type of deterioration can only take place over a wide area if the reservoir level is subject to fluctuations. A constant level reservoir will be subject to deterioration only near the full supply level.

2.2.3 Freeze-thaw cycles

Frost action can also produce overconsolidation and cracking of soils. The increase in volume upon freezing can split the soil, and water will be drawn from the soil adjacent to the ice lense to enlarge the size of the lense (Mackay 1974). Soils with high activity and high moisture contents appear more susceptible to ice lense formation (Michel 1978). Kjartanson (1978) quotes a case where a reticulate ice vein network with veins up to five millimeters thick was noted to a depth of 2.5 metres in an unprotected impervious plastic core after one winter. Such a network formed in a liner would have a major effect on its water tightness.

Attempts have been made by Geomorphologists to classify the effect of climatic regimes on the weathering of rock. Peltier (1950) relates rainfall and annual mean temperature to chemical and frost weathering. Similar relationships probably exist for weathering of soil, and such a formulation would be of use in choice of liner type.

2.2.4 Expansive Soil

An expansive soil used as a liner will suffer a reduction in permeability in the upper layers due to swelling. For an unblanketted liner, Kisch (1959) suggests that the upper 70% of the liner is ineffective. This percentage may be reduced by using a blanket over the expansive clay liner to provide an overburden pressure and hence reduce swelling.

2.2.5 Removal of Fine Grained Soils

In natural soils there is an accumulation of colloidal clay particles in the B-horizon. It is believed (Thornbury 1969) that these particles are removed in suspension by decending soil water. However, the rate of such leaching is normally slow, and significant leaching to sub-base soils would not be expected in the economic life of most water retaining structures.

At engineering time scales flow can occur directly on most non-dispersive clays with no appreciable erosion. Sherard et al (1976) showed that velocities of up to 5 to 10 ft/s could be maintained for hundreds of hours through nondispersive clay samples, and quotes examples of constant discharge leaks through homogeneous clay dams existing for decades. However there are a number of soil types which are susceptible to erosion. The most notable of these are the dispersive soils.

2.2.5.1 Dispersive Clays

Considerable work has been done on uncompacted low head homogeneous farm dams in Australia (Aitchison et al (1963), Aitchison and Wood (1965) and Rallings (1966)). They found that clays with a high percentage of exchangable sodium cations are susceptible to erosion along preferred seepage paths, particularly if the reservoir water has a low content of solube salts.

The erosion or piping of these clays is due to dispersion. The exchangable sodium cations increase the thickness of the diffuse double layer, and increase the repulsive force between clay platelets. This occurs for all clay types, but the effect is more pronounced for the smectite group. As a consequence of the reduced interplate bond, running water can detach individual particles. At low flow velocities, the clay around the flow channel swells and progressively seals the leak. However if the initial velocity is sufficiently rapid, the dispersed clay particles are carried out.

The effect is not limited to uncompacted fills. Landon and Altschaeff (1976) showed that in addition to soil type and water content composition, the compaction water content was of primary importance. The interaction of all the above factors must be considered in determining the erodability of a soil. Soils compacted dry of optimum tend to have a more porous structure due to aggregation of the clay. These large pores appear to a contributing factor in making soils dry of optimum more susceptible than those compacted wet of optimum.

There is still debate on which laboratory tests to use to determine the susceptibility of a soil to dispersion. The original test introduced was the Crumb Test (Rallings 1966). However the most commonly accepted one now is the Pin Hole Test, in which water is allowed to pass through a prepared hole in a specimen and the extent of erosion noted. Sherard and Decker (1976) suggest that both the above be done, together with a test of dissolved salts in the reservoir water and the Soil Conservation Service Dispersive Test (Decker and Dunnigan 1976). A positive result to any of these is to be regarded as indicative of a dispersive clay.

2.2.5.2 Other Susceptible Soils

Failure by piping has been noted in fine grained soils which are not dispersive, but which have a flat grading curve extending over the range of clays, silts and sands. The Balderhead dam (Vaughan et al 1970) and the Yards Creek Upper Reservoir Dam (Sherard 1973) both had gravelly clay cores. Small concentrated leaks are believed to have lead to washing of the fines out of the cores; the grading of the cores being too flat to retain the fines on the granular matrix. The clay fraction in the above cases did not have sufficient swell potential to seal the cracks.

The above clays are presumably of a different minerological composition to the usual dispersive clays, and cannot be recognised by chemical tests. However it is interesting that both gave readings up to 50% on the Soil Conservation Service Dispersion Test (Sherard et al 1972).

Inorganic silt and fine silty sands are highly susceptible to piping, but are unlikely to be used by themselves to form the impervious lining of a reservoir.

2.3 Formation of Preferred Seepage Paths - Earth Liners

Concentrated seepage through a liner can lead to washing out of soil and the development of piping. The cause of the preferred path may be present before filling of the reservoir or may be developed due to the loading imposed by the reservoir filling.

2.3.1 Soil anomolies

Various faults in construction procedure and control may lead to imperfections in the liner. Variation in compactive effort and the variable nature of the placed material may lead to zones with locally higher permeability. Conduits and other structures placed through the liner are likely sources of concentrated leakage. Careful attention to detailing and construction supervision should reduce the incidence of these faults.

2.3.2 Tensile cracking

Once the reservoir is loaded differential settlements and movements take place. Tensile strains are developed, and if the material is sufficiently brittle cracking will occur.

Tensile zones may develop where there are abrupt changes in the stiffness of the support material, or where there is an abrupt change in shape. Changes in the stiffness of the material underlying the material may be due to (1) different foundation materials

(2) poor compaction of the subbase

(3) poor or incorrect backfilling of trenches for conduits, drains etc.

(4) frozen zones in the foundation.

Abrupt changes in the shape of the liner, such as the transition from reservoir side to bottom or around an outlet conduit will lead to stress concentrations and possible tensile straining.

It is possible to calculate where zones of tensile stress and strain will occur in quite complex situations using the Finite Element method. This has been done for the El Infiernillo Dam (Lee and Shan 1969) and the Duncan Dam (Krishnaya 1973). This sort of complex analysis is, however, not normally justifiable for small lined reservoirs.

The ductility of the liner is of primary importance in determining whether cracks will develop in zones of tensile strain. Factors which affect the ductility are the type of soil, the water content, the rate of strain, the type and amount of compaction and the stress state in a direction normal to the tensile stress direction. The method which is used to test the soil also has a major effect on the results obtained.

Leonards and Narian (1963) tested compacted prisms of soil in bending, and found that cracking was initiated at very low strains (0.1%) at moisture contents at least 3% below optimum. Ductility increased to 0.3% at optimum and little increase was shown for additional moisture-added. An attempt was made to relate soil type to piping and cracking potential. This is given in Table 2.1.

Krishnaya (1973) observed a similar increase in ductility near and up to optimum. Increased compactive effort gave a slightly higher strain at failure, while the addition of bentonite significantly increased the failure strain.

2.3.3 Other causes

Faults or geological movements due to subsidence may lead to cracking of liners. An example of failure due to movement on faults induced by settlement is the Baldwin Hills Reservoir. This failure is examined in detail under Case Histories.

Expansive clays as foundation material may create differential settlement of a liner if overburden pressures and/or access to water varies across the site.

Gas or air trapped beneath the liner may cause rupture of the liner. Organic material in the subbase can continue to produce gas, and slight variations in the underside level of the liner may lead to concentrations forming. Venting by means of a correctly designed underdrain system can alleviate this problem.

2.4 Flexible liners

The selection of a suitable plastic or other impermeable flexible liner depends to a large extent on the material to be contained in, and the operating conditions of, the reservoir. It is beyond the scope of this report to examine individual liner types. However, information can be obtained from the references given in the Bibliography. For this discussion, plastic liners are considered as these are currently the most economic type and their failure modes are typical of those for other flexible linings.

The permeability of an intact piece of a plastic liner is very low, generally being below 10^{-10} cm/s. However holes are invariably present, either due to manufacturing faults, mishandling or missealing. Table 2.2 gives seepage rate comparisons based on the personal experience of Kays (1977). These values are a rough guide only, and field values could vary appreciably from those shown.

The main causes of damage to plastic liners are briefly listed below for reference. Additional information may be obtained from Kays (1977) and the Bibliography. Those with geotechnical import are examined in more detail.

2.4.1 Mechanical difficulties

Holes can be created in the liner during manufacture, shipping and handling and installation. Pinholes exist in effectively all plastic liners and these become focuses for tearing. Laminating of liner materials removes this problem. Larger holes due to mishandling are more readily seen and repaired during construction.

Field seams for liners are a common source of leakage. The adhesives used may be unsuitable or contiminated. Humidity and temperature can effect the seal and improper lining layout or contractor inexperience can all lead to ineffective seals.

Seals around structures are particularly susceptaible to leakage, and require particular attention. Sufficient lining material must be placed in corners to prevent bridging occuring.

The important point to note is that no matter how carefully a job is completed there are such a large number of sources of tearing and holing that it is prudent to assume that leaks will occur in the liner.

2.4.2 Soils Problems

A number of soils problems are peculiar to plastic lined reservoirs.

2.4.2.1 Subbase :

The preparation of the base for the liner has to be carefully controlled to avoid sharp objects which may puncture the membrane. In addition, organic matter must be removed so that gas formation under the membrane does not occur. Holes and cracks in the subgrade can cause the liner to extrude and may lead to tensile failure, particularly if the hole is bridged by the membrane. Bridging can also occur at structure-subbase interface. Certain reeds can grow through some liners. These should be removed, and the ground sterilized.

2.4.2.2 Leakage :

It should be expected that leakage will occur from an "impermeably" lined reservoir. The magnitude of this leakage is difficult to determine as it depends on faults in the liner, rather than the properties of the lining material. Monitoring of leakage by an underdrain system can detect faults, and possibly their location. Regional and local effects as outlined in the section on earth lined reservoirs need also to be considered.

If the groundwater rises above the base of the reservoir, either due to leakage from the reservoir, or changes in the groundwater regime a reverse hydrostatic pressure will act when the reservoir is empty. Uplift on the base of the reservoir will cause the liner to lift. Refilling will not necessarily alleviate this condition, as water tends to pond under the liner forming bubbles. As the pressure head is increased the bubble decreases in size but develops a small radius and this may lead to rupture (Kays 1977). Similar failure results from trapped gases.

2.4.2.3 Membrane Cover :

A soil cover is often required over an impermeable lining. The thickness of the soil cover will be determined by the following factors. Some lining materials, such as PVC and polyethylene are strongly degraded by ultraviolet light, whilst others, particularly rubber based lining materials are weathered by ozone. The flexibility of a liner is affected by temperature. PVC and polythelene become stiff in cold temperatures and more susceptible to damage by impact such as hail or ice. The property range of typical liners is given in Table 1.1. Wave and wind action can lift uncovered liners, causing tearing. Wave erosion and sluffing of the soil cover can reduce the thickness of the protective layer.

The slope of the inside of the reservoir will be determined primarily by the friction between the lining material and the soil cover. Foster et al (1977) used a modified direct shear box to measure values of friction for hypalon against different soil types. There are three cases which must be analysed. These are the stability of the lining soil in the moist immediately post-construction condition, with the reservoir full, and rapid drawdown. The rapid drawdown case is likely to be most critical unless a freely draining granular material is used for the soil cover.

It is necessary to anchor the lining material at the embankment level. Typically a trench is dug and liner layed in it and backfilled. A similar detail may be used at the toe if the liner does not extend across the floor of the reservoir.

If large settlements are expected, allowance should be made for this extension of the liner by providing "tucks" in both directions. If the settlements are likely to be significantly different in the cut and fill zones of the reservoir wall, a berm between these zones will alleviate high tensile strains.

The placement of the soil cover must be carried out in such a manner to prevent tearing. A minimum thickness from a practical construction viewpoint is 0.6m perpendicular to the slope.

2.5 Rigid Liners

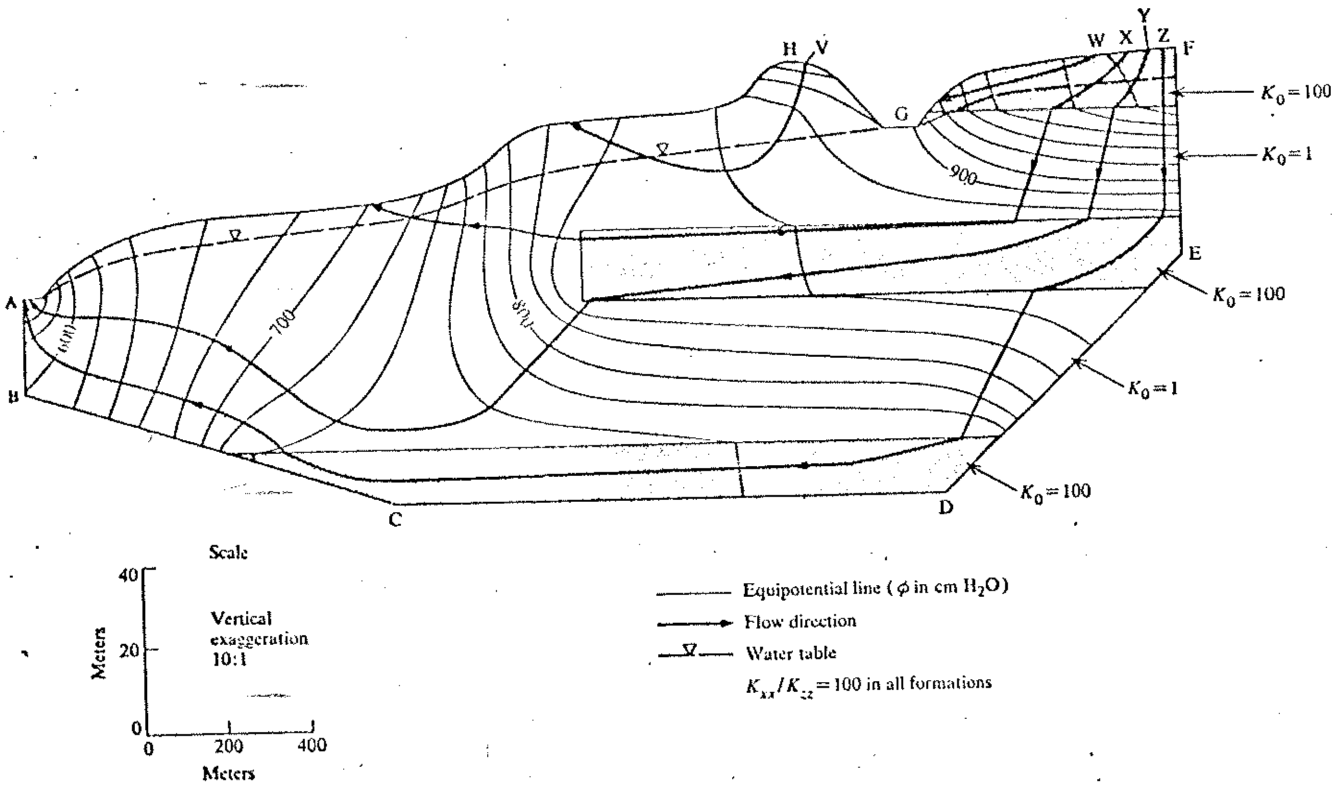
Rigid liners are particularly sensitive to subgrade variations. Their rigidity prevents them from following the shape of the subgrade and so they have to span across any weak zones, holes etc., perhaps leading to cracking.

Holes or weak zones may form beneath a rigid liner due to inadequate subgrade compaction, or settlement of the ground. Certain soils are subject to subsidence when wetted, and since it must be assumed that a reservoir will leak, precautions against this should be taken, such as prewetting.

Seepage from a liner can cause bedding or filter material to be washed out from under the liner, again leading to shell-like cave-ins of the rigid lining. This has been noted with concrete linings and clay concrete liners (Ipfelkofer 1961), particularly where the bedding has been placed on jointed rock.

Swelling or bulging of the foundation material can also lead to failure of rigid liners. Expansive soils may be differentially wetted by leakage leading to substantial differential movements, (Holtz & Gibbs 1954). Frost heave may also cause problems, but this can be generally avoided by the design of an adequate underdrain system. Excess hydrostatic pressure beneath a liner, such as may occur after the rapid lowering of the reservoir level, can also cause cracking of the liner.

FIGURE 2.1 : Regional Flow Modelling



Steady state regional flow through a ground water basin

	Site				
	V	*W*	*X*	*Y*	*Z*
Saturated hydraulic conductivity at surface (cm/day):					
Horizontal	50	5000	5000	5000	5000
Vertical	0.5	50	50	50	50
Depth to water table (m)	20	12	12	12	12
Length of flow path (m)	540	360	1900	2800	2900
Time of travel (yr)	330	1.02	123	660	226
Entry rate at site (cm/day)	0.06	58	30	10	2.
Discharge rate at exit point (cm/day)	0.75	78	1.5	3.5	5.

Hydrogeological data for comparison of the waste disposal sites indicated above

TABLE 2.1 : Classification of Embankment Materials - Piping and Cracking

Group number	Soil type	A. Casagrande's airfield classification system symbols	Approximate Ranges of Soil Properties: Median grain-size D_{50} (mm)	Plasticity index[c]	Liquid limit	Per-cent clay sizes (0.005 mm)	Number of dams in each soil group	Piping[a]: Degree of resistance (1) greatest to (6) least	Piping resistance	Cracking[b]: Degree of susceptibility (1) greatest to (6) least	Susceptibility to cracking when compacted dry	Relative Importance of Moisture-Density Control: Degree of importance of control (1) greatest to (6) least	Consequence of inadequate moisture control
I	Sands and gravels with plastic fines	SC SF GC GF	0.15-5.0	8-15	20-50	5-30	6	(3)	Intermediate resistance. Heavier compaction and higher plasticity index increase resistance.	(3)	Intermediate susceptibility may crack only under extreme combinations of conditions.	(5)	May fail by cracking or piping only under severe combination of detrimental conditions.
II	Sands and gravels with non-plastic fines	GF SF	0.15-5.0	0-8	10-30	0-15	6	(5)	Low to intermediate resistance. Heavier compaction and higher plasticity index increase the resistance.	(4)	Intermediate susceptibility.	(3)	Most likely to fail by piping. May possibly fail by cracking.
III	Inorganic silts of low compressibility and fine silty sands	ML ML-CL ML-SC ML-SF	0.03-0.15	0-10	10-45	0-25	12	(6)	Uniform sand with P.I. < 6 has lowest resistance. Well-graded material with P.I. > 6 has intermediate resistance.	(2)	High susceptibility. The finer and more uniform the soil, the greater the susceptibility.	(1)	High probability of failure by piping and cracking.
IV	Inorganic silts and clays of low medium plasticity	CL ML	0.10	10-25	20-50	10-40	30	(4)	P.I. < 15--intermediate resistance.	(1)	Material with $D_{50} > 0.02$ mm and P.I. < 15 has highest susceptibility.	(2)	Most likely to fail by cracking. May fail by piping.
								(2)	P.I. > 15--high resistance.	(5)	Material with $D_{50} < 0.02$ mm and P.I. > 20 has high post-construction settlement but sufficient deformability to follow without cracking.	(4)	May fail by cracking or piping only under severe combinations of detrimental conditions.
V	Inorganic clays of high plasticity	CH CL-CH	0.02	25-40	40	30	6	(1)	High piping. Resistance not severely lowered by very poor compaction.	(6)	Unlikely to crack. High post-construction settlement but high deformability.	(6)	Least likely to fail by either piping or cracking.

[a] In general, the coarser the soil and the less the plasticity, the greater the increase in piping resistance due to increased compactive effort.
[b] Susceptibility to cracking was not observed to be decreased appreciably by increase in compactive effort. Rapidly disintegrating residual soils may be especially susceptible to cracking.
[c] No dams constructed of soils with plasticity index greater than 40 were included in the investigation. (After Sherard)

(Leonards & Narian 1963)

TABLE 2.2 : PERMEABILITIES OF LINERS

Liner Type	Permeability Range (Cm/S)
Earth Liners	
Compacted Soil	10^{-4} - 10^{-7}
Bentonite Sealant	10^{-5} - 10^{-7}
Rigid Liners	
Concrete	10^{-7}
Steel	Nil
Gunite	5×10^{-7}
Asphaltic concrete	10^{-7} - 10^{-9}
Soil Cement	10^{-7}
Flexible Liners	
Polyvinyl Chloride	5×10^{-9}
Polyethelene	10^{-9}
Chlorinated Polyethelene	10^{-10}
Hypalon	10^{-10}
Elastomers	10^{-10}

Note : Data primarily from Kays (1977), Williams (1979) and manufacturers specifications.

CHAPTER 3 - CASE HISTORIES

CASE HISTORY A

A plastic membrane lined reservoir was built in a northern Alberta town in 1966 to act as a stilling basin for the removal of silt from the river water used as the town water supply. A piping failure developed through the embankments in the winter of 1979 discharging large quantities of water into the surrounding area.

A. 1. Description of the Reservoir

The reservoir is approximately 500 ft x 400 ft and holds 19 ft of water. It is lined with a 6 mil "polythene" liner placed directly on the excavated soil and compacted embankment. The liner was protected by a one foot thick layer of selected fill material. The general layout is shown in Figure 3.1, and a typical section in Figure 3.2.

A. 1.1 Site Description

The results of the original site investigation for the stilling basin were not available. A number of later investigations close to the site were examined. The location of these boreholes is shown on Figure 3.1, and typical borehole logs are shown in Figure 3.3.

These boreholes show a stratum of variable medium plastic-clay containing some organics, stratified and interbedded with silt and sand lenses. The thickness of this stratum varies between 7 ft and 24 ft in the boreholes, and probably averages about 20 ft in the region of the reservoir. Beneath the clay-silt layer a dense grey gravel containing pockets of sand and silt extended to the end of the borehole at depths up to 32 ft below the surface.

The area is low lying, being about 18 ft above the normal local river level and considerably below the surrounding plain level. A contour map showing the reservoirs proximity to the river is shown in Figure 3.4. The silt-clay deposit is alluvial, probably deposited as a point bar at the junction between two major rivers.

A. 1.2 Construction of the Reservoir

No details on materials used, or procedures employed in the construction of the reservoir were available. The only sources are the initial working drawings, observations made at the time of failure and hearsay information. Hence most of the following is conjecture.

The reservoir was constructed by using the material excavated from the interior of the reservoir for the embankment. Organic material is believed to have been removed and the embankment compacted.

The completed interior surface had a constant grade of 1 : 4 over its full height. The material was then raked to remove rocks and the 6 mil polythene liner installed. Lapped joints were used. The liner was brought up to the underside of the concrete slab supporting the inlet and outlet pipes, and was anchored at the embankment top by running the liner 5 ft onto the crest and covering with 2 ft of soil. The liner was then covered with 1 ft of "select fill material". This material can be seen in Photo 7, and appears to be a sandy silt containing pebbles. It is unlikely that this covering soil was compacted since compaction of such a thin cover would require special equipment.

A. 2. Description of Failure

In the winter of 1979 a sudden excape of water occured through the central portion of the south-west dyke. According to witnesses the water flowed from two seperate sources. One was a horizontal crack located on the downstream slope of the dyke about 6 ft below the crest being about 6 - 8 ft long and 6 - 8 ft wide. The other was at a hole located at the opposite side of a trench adjacent and parallel to the dyke. The location of these holes is shown on Figure 35, and the first leak hole is shown in Photo 1, 5 and 6.

The discharge was fairly constant from the time the leak was noticed. The water level in the reservoir was lowered by draining through the inlet and outlet pipes and by discharge from the leaks to a level estimated at 12' of water.

The ice on the reservoir at the time of failure was approximately 4 - 6' thick and extended right up to the crest of the dam, and in some places was piled over and above the crest. This may be seen in Photo 1 and 3.

An inspection soon after the failure indicated a number of cracks parallel to the crest, and extending for about 90' along the crest. They are indicated on Figure 3.5 and shown in photo 2. Inspection in winter was hampered by the ice and snow covering.

The failure was re-examined in spring. The soil movements were found to be minor and shallow, extending to some 4' below the crest on the downstream side. Some blocks of soil had toppled down the slope. This region is shown in Photos 4 and 5.

The liner was inspected and in a number of places the plastic was exposed where slumps had occured. The liner was severely degraded and in some locations a finger could readily be poked through it. Bubbles of water were noted in some areas underneath the liner. In the region immediately upstream of the failed zone a large area of the embankment was exposed where the liner had apparently been pulled down with the slumping cover soil. This region is shown in Photos 7, 8 and 9.

The downstream exit of the pipe is shown in Photo 6. Its location on the embankment is indicated on Photo 5. The outlet is approximately circular and 8" in diameter. Silt which has been carried in the flow can be seen downslope from the opening.

A. 3. Operation of the Reservoir

Water was pumped into the reservoir from the neighbouring river by manually controlled pumps. The water-level fluctuated widely during normal operation and one case of overtopping two months prior to failure has been reported. This was in the same region as the failure.

The embankment was grassed on the crest and upstream face to approximately 4' below the crest. Beneath this there is evidence of wave action having washed the fine material from the soil cover leaving a beach of coarse gravel cobbles. This can be clearly seen in Photo 7, and indicates that normal operating level is 4 - 6' below crest level.

During the spring of 1978 several slippages occured on the U/S grassed slopes of the reservoir. The liner was exposed in certain areas where the soil cover slumped down (Photo 8). These areas were not repaired.

A. 4. Factors contributing to failure

The exact sequence of events which led to failure is difficult to reconstruct with the limited information available. The following discussion presents an hypothesised series of events which explains most of the phenomena observed. Other explanations are certainly possible.

A. 4.1 Degradation of the soil cover

The action of freeze-thaw and the continual wetting-drying of the soil cover would gradually have reduced the soils cohesion. Eventually the material could be expected to act as a cohesionless soil.

In addition, wave action has removed fines from the beach area and re-deposited it in deeper water. A beach of cobbles was left which would have reduced the effective cover on the plastic liner in this region to an estimated 4". A small cliff was created at the edge of the grassed section above the beach. This cliff can be seen in Photo 8.

A. 4.2 <u>Sliding of the liner covering material</u>

The slope of the U/S face of the reservoir is 4 : 1. From Photo 9 it would appear that the soil cover slumped along the plastic sheeting as a unit.

The angle of friction between the plastic liner and the soil cover is not known. A rough laboratory test gave an angle of friction of 30° between the liner plastic and dry Ottawa sand. This would be the upper limit of friction angle.

The behaviour of the soil liner is complicated by the mode of operation of the reservoir. Wide variations in water level occur over short time periods. On a 24 hour basis 3000 000 gallons may typically be withdrawn. This corresponds to a drop in water level from full supply level of about 4'.

To check whether rapid drawdown is a concern, the permeability required to allow the saturation line to follow the reservoir level was calculated using the method proposed by Cedegren (1967). Assuming a porosity of 0.3 and a head difference from one side of the soil cover to the other of 0.4' it was found that a minimum permeability of 1.8×10^{-2} cm/s was required.

This is much greater than the expected permeability of the soil cover which should be in the range 10^{-4} to 10^{-6} cm/s. Hence a condition of rapid drawdown should be considered.

The undercutting of the grassed section of the U/S slope removed or reduced the end restraint on the liner. Since the soil cover was at the point of failing, a back analysis was used to determine the friction angle between the plastic liner and soil cover. This gave a friction angle of 25° which is in the range proposed by Foster et al (1977).

Since the reservoir had been operating for 12 years without slippages occuring, some action was required to reduce the factor of safety below unity. The most likely have already been discussed under degradation of the liner. However it is possible that some other mechanism may have been the trigger.

One possibility is that the operation of the reservoir may have led to the saturation and freezing of the grassed section of the U/S reservoir slope. If this then remained covered with ice it would be possible for melting to occur first at the plastic liner - soil cover interface. High pore pressures could then develop, leading to block sliding.

A. 4.3 Liner Failure

A. 4.3.1 Degradation of the plastic liner

The liner material is described as '6 mil polythene'. The properties of polyethelene sheeting may vary widely depending on the quantities and types of additives added during manufacturing. Typical property values have been given in Table 1.1.

Polyethelene is notoriously sensitive to ultraviolet light. Serious degradation can be expected within one year of exposure (Kays 1977). Reduced tensile strength and increased brittleness result.

Freezing weather will cause polyethelene sheet to stiffen even further, so that in the winter of 1979 the exposed liner was brittle and susceptible to tearing under impact loads.

A. 4.3.2 Tearing of the plastic liner

The liner may have been torn in two ways :

A. 4.3.2.1 By sliding soil cover

The initial strength of the liner would be about 160 lb/ft run, estimated from recent tests on similar material (Staff 1978). An estimate of the force exerted by the sliding soil gives a force of approximately 50 lb/ft^2 to the liner. At least 3' of the soil cover has slid, and hence there is sufficient force to cause failure, particularly if the liner were deteriorated.

It is not clear how much reduction in strength would have occured through exposure of the liner to organics and sunlight. It is however certain that a reduction would have occured. The presence of bubbles of water or gas beneath the membrane would have aggravated the situation.

A. 4.3.2.2 By ice action

Thicknesses of ice up to 6' are reported on this reservoir. The water levels flucuate widely, which would cause the ice to grind against the exposed membrance leading to localised punctures.

It is not definitely known if the liner was punctured at the end of summer 1978. However, the water level during the summer months is more closely monitored and water levels would not be expected to rise above the beach level.

A. 4.4 Embankment Failure

A. 4.4.1 Piping

The embankment was constructed from material excavated from the reservoir. Since these deposits are variable it should reasonably be expected that the embankment is heterogeneous with local zones of higher silt and sand content. These, together with irregularities in construction compaction could lead to preferred seepage paths.

The embankment material adjacent to the failed zone was tested and found to be a clayey silt. The grading curve is shown in Figure 8. Sherard et al (1963) states that this material is very susceptible to piping.

The embankment is founded directly on the clay stratum. The presence of organics in this material and its low strength suggest that the soil is probably quite compressible. Variations in embankment crest level of up to 1.5 m as shown on the contour plan Fig.3.4 indicate that substantial differential movements have occured with almost certain cracking of the embankment accompanying these movements. The limits for susceptible soils suggested by Sherard et al (1963) are superimposed on the grading curve (Figure 3.6).

An overtopping of the embankment occured two months prior to the final failure, in the same region as the final failure.

No piping was noted at this time, although this would be difficult to determine due to the snow and ice cover. At the least this overtopping must have increased the water content in the embankment. The matter is complicated by the lack of any substantial information concerning this overtopping, and particularly the state of the embankment at the end of this incident.

In the week prior to the final failure, unusual ice build up occured along the embankment top in the region which failed. This was presumably due to the high water levels being maintained in the reservoir.

It is of interest to speculate about the advance of a wetting front through the embankment. This liner had been in service for 12 years and had been subject to significant straining. Certainly some leakage would have been occuring from the reservoir and a relatively steady seepage pattern would have developed. The height of the phreatic surface in the embankment would have depended on the quantity of discharge and the local ground water level.

The embankment is likely to be anisotropic since it was compacted in layers with varying properties. Some of these layers may have acted so as to produce perched water tables. In addition the overtopping incident would have increased the water content of the upper layers.

During the week prior to failure the wetting front was established through the upper portion of the dyke. Gradual erosion by piping would have occured along preferential seepage paths till finally a complete pipe was established through the dam. Rapid erosion of the silt followed, creating a substantial pipe and noticeable flows.

A. 4.4.2 Slope Failure

The cause of the shallow slope failure near the crest of the downstream face is not clear. The cracks extended over a length of about 90' and may not be associated with the development of the piping failure, but rather with the overtopping incident.

The toppling nature of the failure suggests that a horizontal force was applied, probably to ground that was frozen. Possible sources of such a force include ice thrusting, ice lense formation or excess pore water pressure. Mechanisms for the application of these forces are not obvious.

A. 5. Future Reservoir Operation

The water level in the reservoir was reduced to approximately RL 810 m. No further leakage has been reported with this water level being maintained.

The reservoir is adjacent to a river which floods to levels of approximately RL 813 m. The stratigraphy of the area indicates that the water level under the reservoir would follow fairly rapidly variations in river level.

If a reverse hydrostatic pressure were allowed to develop beneath the liner, the liner may lift allowing water bubbles to form. Increasing the head again may not solve such a problem as the usual effect is to localise the water under the liner creating smaller bubbles with higher resultant stresses in the liner. With the deterioriated state of the liner, this would be risky.

A. 6. Conclusions

The reservoir with its polythene liner performed satisfactorily for 12 years. That it did so reflects well on the polythene sheets ability to remain intact while sustaining substantial strains. Minor leakages from the liner created no problems.

Lack of provision of wave and ice protection for the soil cover, and the failure to instruct maintenance staff of the importance of maintaining the soil covers integrity were major omissions.

A manually controlled pumping system requiring visual monitoring of the reservoir water level is unwise, particularly in regions where an ice cover will form. An automatic cut off system to prevent overfilling should be standard practice.

No backing system, such as underdrains, filters, were used to control seepage in the embankment in the case of liner leakage. Where the embankment is susceptible to cracking and piping provision of such a control would be prudent.

Figure 3.1 : GENERAL LAYOUT AND LOCATION OF BORE-HOLE

(Case History A)

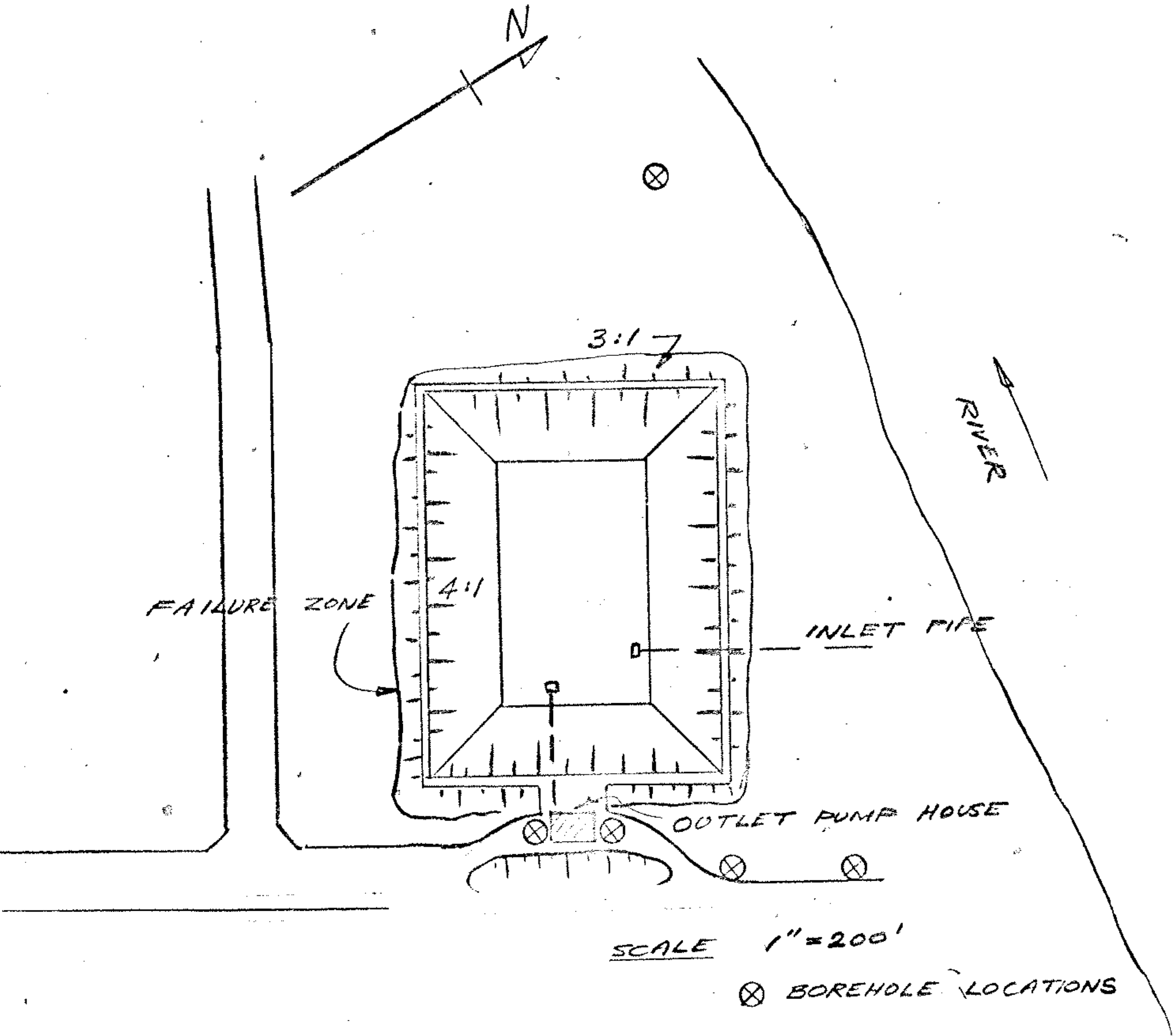


Figure 3.2 : RESERVOIR LINING DETAIL

(Case History A)

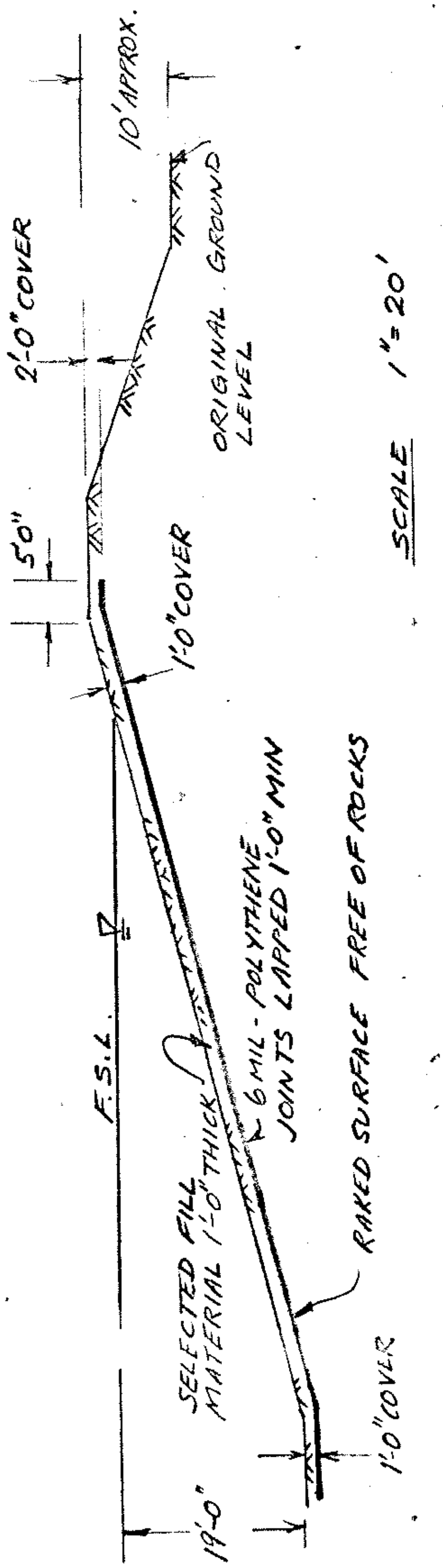


Figure 3.3 : TYPICAL BOREHOLE LOG

(Case History A)

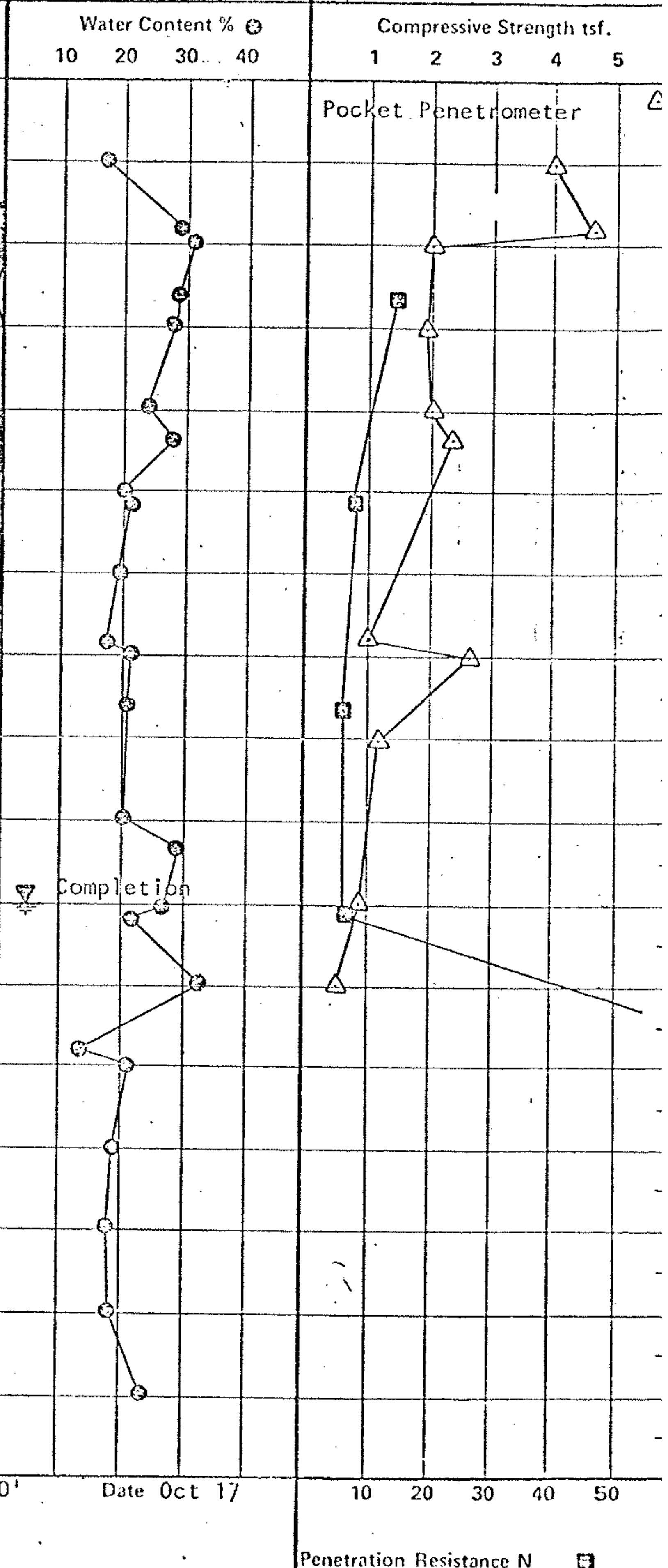
PROJECT New Water Treatment Plant -

SURFACE ELEVATION 812.3

TESTHOLE No. 3

JOB No.

Depth ft.	SOIL DESCRIPTION
0–4	CLAY(FILL) med. to dark brown, sandy & silty, some organics, low plastic, very stiff black, very silty, organics, pieces of trees
4–13	CLAY med. brown, silty, trace of organics, some horizontal sand seams & pebbles, low plastic very silty, low plastic trace of organics, clayey silt to very silty clay, low plastic, firm to stiff
13–20	SILT clayey & sandy, med. brown, fine sand lenses, low plastic fine sandy, some clay lumps light brown, sandy & clayey, non-plastic grey to white sand lenses 1" thick
20–25	CLAY med. brown, very silty, low plastic, firm, med. brown, very sandy, silty & gravelly, wet
25–32	GRAVEL med. brown to grey brown, sandy, med. to coarse, some tarsand, some gravel @ 26' to 1-1/8", very silty & clayey sandy & very silty, some clay, pebbles to 1" very silty & clayey gravel to 1/2", very silty & sandy
32	END OF HOLE



Completion Depth 32.0'

Date Oct 17

Depth to Water in Boring 20' on Completion

Page 1 of 1

Dwg. No.

Figure 3.4 : LOCALITY PLAN,

(Case History A)

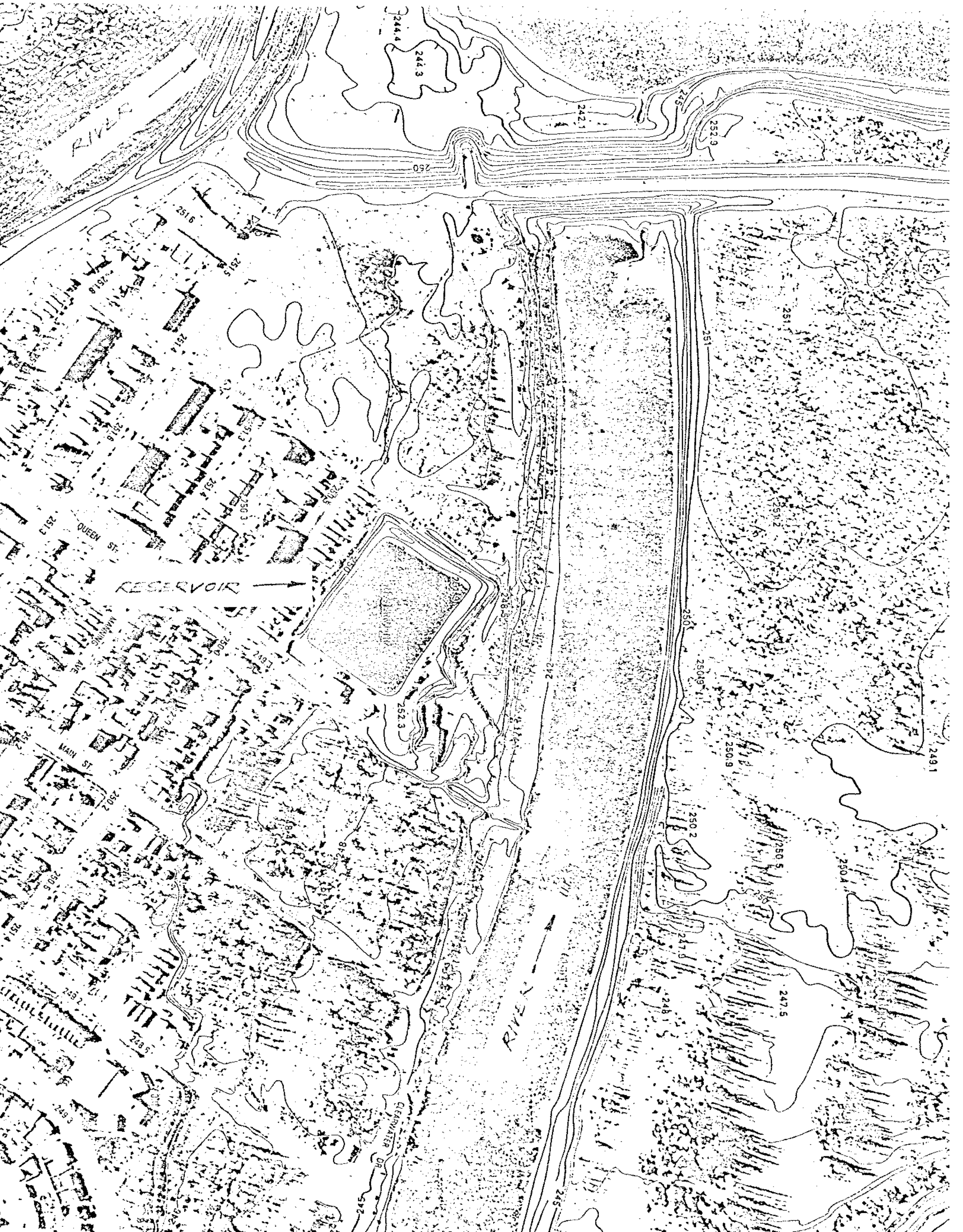


Figure 3.5 : ISOMETRIC VIEW SHOWING LOCATION OF LEAKS AND CRACKS (Case History A)

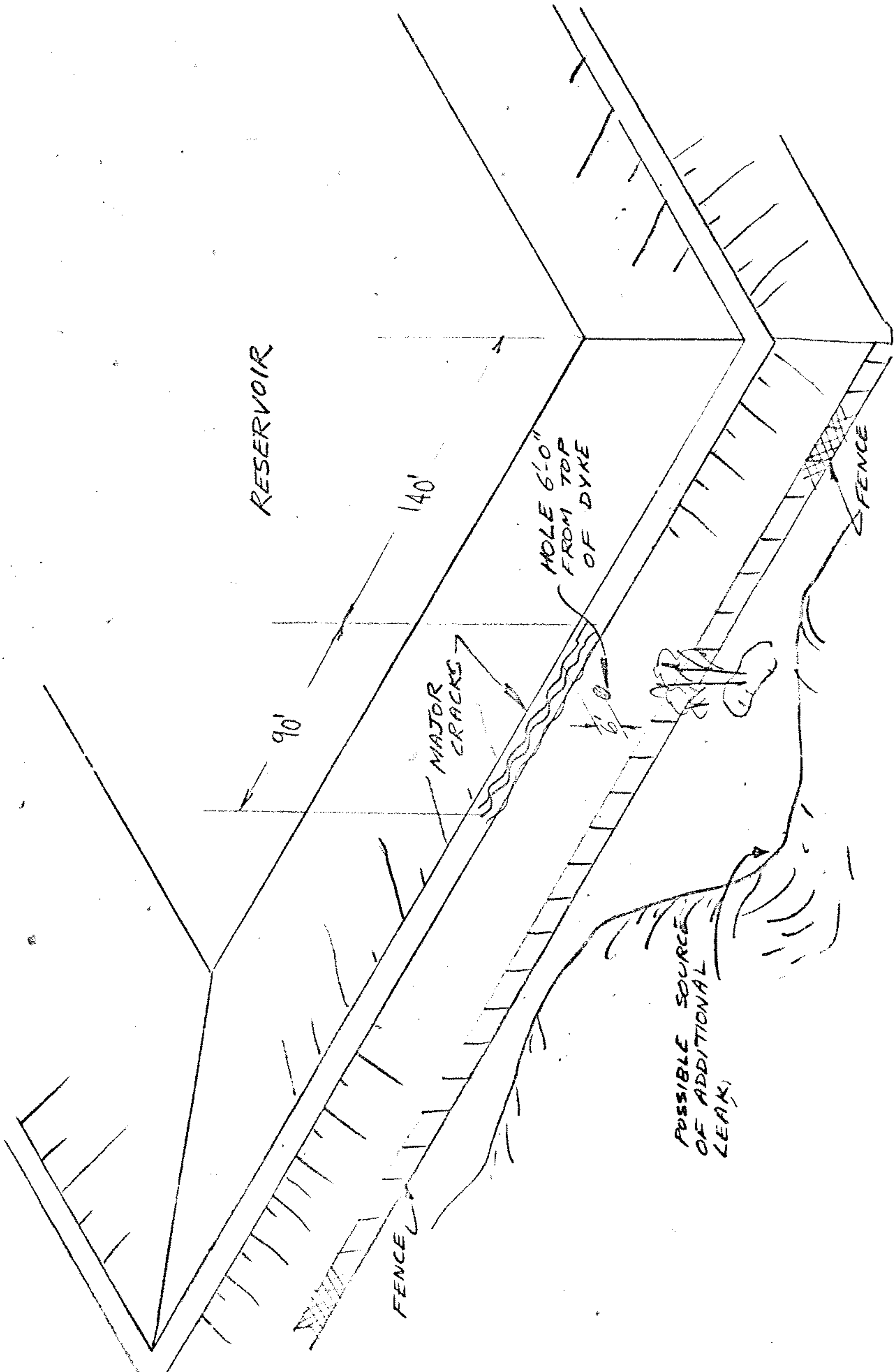


Figure 3.6 : GRAIN SIZE CURVE FOR EMBANKMENT.

(Case History A)

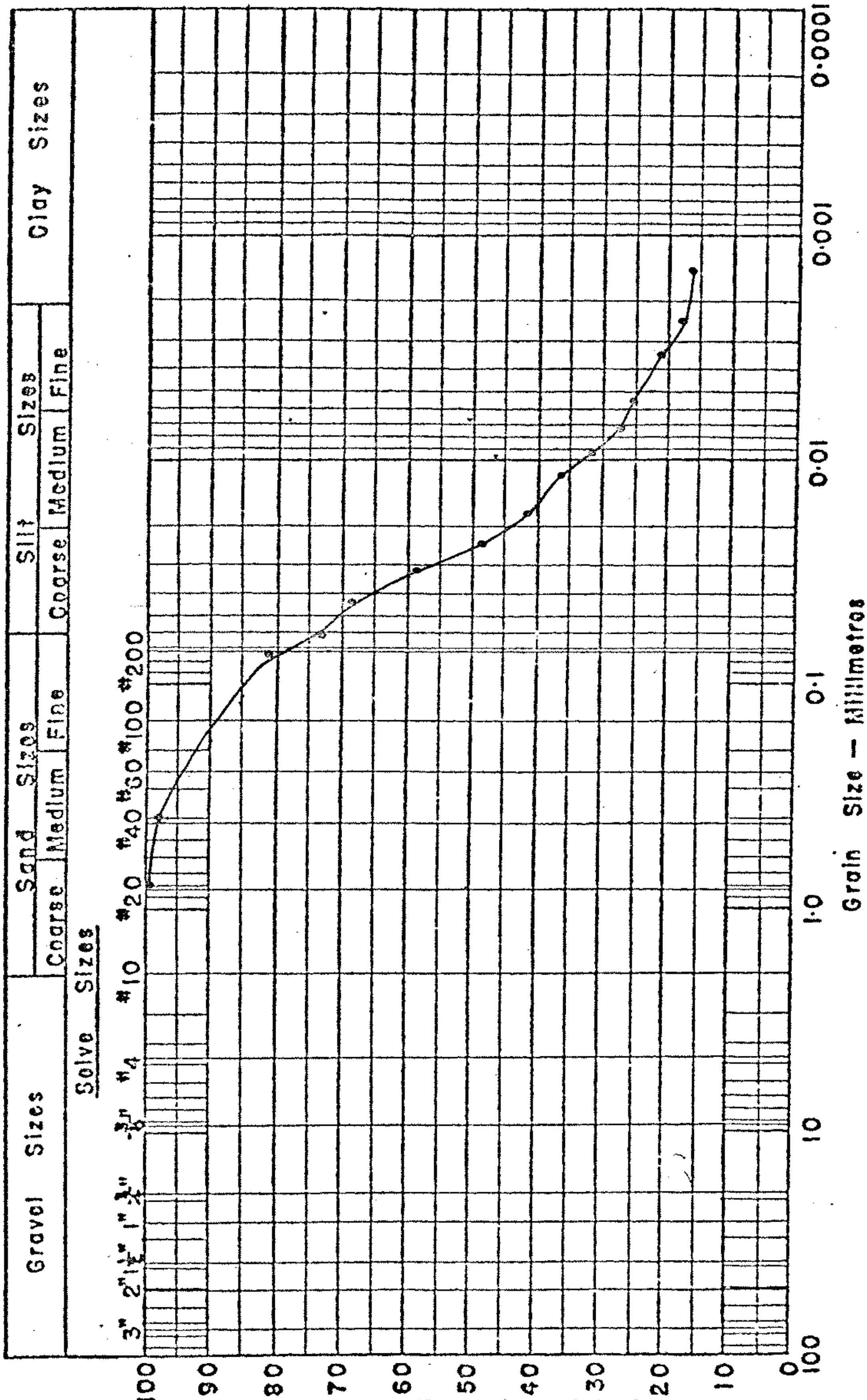


Figure 3.7 PHOTOGRAPHS : CASE HISTORY A

Photo 1

Major leak location immediately after failure. Note the ice buildup on top of embankment.

Photo 2

Cracking on top of the embankment immediately after failure.

Photo 3

Cleared embankment after failure.
The depth of ice pileup may be seen.

Photo 4

View along D/S edge of embankment in failure zone.
Note toppling of soil blocks near rim.

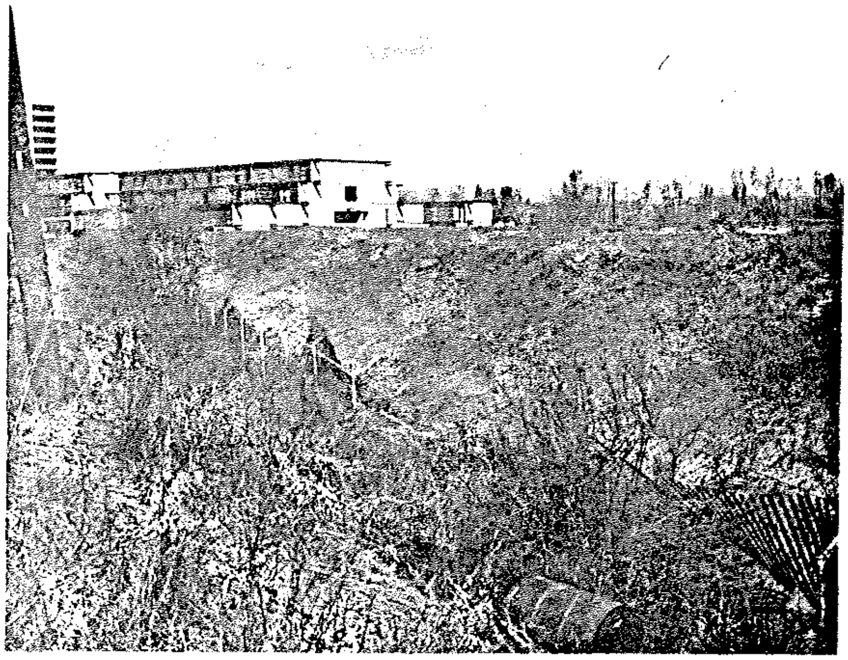


Photo 5

General view of failed embankment.
Toppled soil blocks and piping outlet are visible.

Photo 6

Closeup of piping outlet.

Photo 7

View of embankment U/S of piping failure outlet. Note the slumping of grassed areas and the formation of a beach.

Photo 8

Damaged section of polyethene liner showing exposed embankment material.

Photo 9

Slumping of soil cover on apparently intact polyethene liner.

CASE HISTORY B

A water retention structure for an industrial plant in central Alberta was constructed during 1976. Underdrains showed excessive leakage once the reservoir was filled; and upon inspection of the compacted clay lining cracks and potholes were observed.

B. 1. Description of Reservoir

The reservoir is approximately 1200 x 600' and holds 25' of water. It is lined with 3' of silty clay compacted to 95% proctor optimum at moisture contents at or wet of optimum. This overlies a gravel blanket drain containing pipes which discharge through the embankment. The general layout is shown in Figure 3.8 and a typical section in Figure 3.9.

B. 1.1 Site Description

The reservoir is located on a river terrace approximately 40' above the normal river level. Floods come to within 20' of the terrace.

Boreholes indicate a thin layer of topsoil overlying a variable stratum of stratified and interbedded silty clay containing five grained sand and silt lenses. This stratum varies in thickness from 5 to 15'. Beneath the clay layer a deposit of dense poorly graded gravel overlies sandstone. The gravel layer varies considerably in depth up to 13'.

It is probable that the gravel was deposited in the bed of a river originally cut in the sandstone, and that the stratified clay in an alluvial deposit. The exact geological history of the area was not determined.

B. 1.2 Lining Material

The 3' thick clay liner was compacted from material obtained from a borrow pit adjacent to the reservoir. The soil was a medium plastic clay till, very silty with traces of sand, coal and rust specks. Atterburg limits and insitu permeabilities were determined and are summarised in Figures 3.10 and 3.11 Sieve analysis are shown on Figure 3.12.

There appears to have been two groups of soil present in the borrow area; one CI and the other CL. The permeability range for both these were similar, between 10^{-6} and 10^{-9} cm/s. Laboratory falling head tests gave values between 2×10^{-4} and 6×10^{-9} cm/s. Dispersive properties were also investigated.

B. 1.3 Underdrain System

A one foot thick gravel blanket was placed under the reservoir lining as shown in the section, Figure 3.9. Its stated purpose was to ensure that excess hydrostatic pressure could not build up beneath the reservoir and perforate the clay liner.

Drainage pipes are imbedded in the gravel blanket as shown in Figure 3.8. The spacing of drains was calculated considering the expected permeability of the liner (10^{-7} cm/s) and the expected seepage from surrounding areas. It appears that much of the site is drained by a natural gravel layer. The thickness of this bed beneath the clay liner has been estimated from the borehole logs and is shown in Figure 3.13. The location of the gravel outcrops in the adjacent valley which act as discharge channels for the reservoir area are not readilty determinable due to the complex variability in depth to the sandstone strata.

B. 1.4 Other considerations

The action of freeze-thaw cycles on the clay liner were considered. It was believed that gradual deterioration was inevitable, and hence regular inspections and repair as required were specified.

To ensure that the underdrains did not freeze and cause pressure buildup they were installed at sufficient depth to prevent this.

Wave action and ice scour have been allowed for by the provision of gabbions at the operating water level.

Piezometers were installed in the embankment to monitor pore water. pressure. No measurement from these were available.

B. 2. Description of Failure :

The reservoir was first filled towards the end of November 1978. Considerable discharge of up to 50 gals/min was noted from pipe 1. Divers were sent under the ice to inspect the region and a hole was found adjacent to the inlet pipes. An attempt to seal this with concrete was made and this reduced the discharge from the pipe by about 20 gals/min.

Between the 21st and 23rd December rapid increases in discharge particularly from pipes 3 and 4 were noted. Over this period the total discharge increased from about 50 gals/min to 200 gals/min. The rate of discharge continued to increase, but at a decreasing rate till by the end of April the discharge was approximately 600 gals/min.

Detailed discharge records for individual pipes are shown in Figure 3.14.

In May the reservoir was emptied and inspected. A detailed report of this inspection was not made avalable. However a verbal communication indicates that some cracking was noted trending east-west. The cracking was small in extent. Two sets of pot holes were noted. One of the sets consisted of two potholes 2½ ft and 6 inches in diameter about 3 ft apart, and lying over an east-west drain. The other set consisted of about a half a dozen potholes 2 to 3 inches in diameter located over the junction between a major drain and a lateral drain.

These were repaired and the reservoir refilled. Discharges increased again to about 190 gals/min and were still increasing at the time of writing.

B. 3. Possible causes of excess leakage

The following mechanisms are hypothesised on the basis of limited information. To determine the importance of each, additional tests and information would be required.

B. 3.1 Liner cracking

The liner material is susceptible to cracking. The limits proposed by Sherard et al (1963) are superimposed on the liner grading curves in Figure 3.12. However, it is also necessary to develop tensile strains in the liner before cracking can occur.

Possible sources of tensile straining are differential settlements or movements

(i) At junctions between zones of different compressibility supporting the liner.

(ii) At stress concentrations due to changes in liner slope or at structures in the liner.

B. 3.1.1 Variable compressibility :

Variable stiffness in the material supporting the liner can arise in a number of ways.

B. 3.1.1.1 Reservoir subbase : Sandstone was intercepted in the excavation along the northern side of the reservoir. An estimate of the outcrop based on borehole logs is shown in Figure 3.13. The balance of the excavation was in dense gravel, with a small zone of topsoil near the south embankment. This topsoil was to be removed and replaced with compacted pit run gravel.

The sandstone is generally non-cemented or poorly cemented, although occasional layers of more indurated sandstone do exist.

All these subbase material would be relatively stiff and differences in stiffness would not be expected to cause significant differential movements.

B. 3.1.1.2 Embankment - Foundation Interface : The embankment is compacted from the silty clay material excavated from the reservoir. In most places this embankment rests on the silty clay stratum. Occasionally the gravel layer is directly under the embankment. No tests were performed to determine the stress stain curves of these materials. In addition the variable nature of the alluvial deposit used in the embankment means that unless particular care was taken in mixing and compaction that heterogeneities could have occured.

It is not possible to decide on the basis of the information available whether differential movements occured at these locations or whether they were sufficiently large to cause cracking.

B. 3.1.1.3 Underdrains

The trenches for the drain pipes under the gravel blanket are cut into the substrata from 2 to 4 ft under the floor of the reservoir to allow sufficient slope for drainage. Where the collector pipes are taken through the embankment the required trenches would be up to 10 to 15 ft. in depth, depending on how much of the surface soil was removed from the embankment foundation.

It is presumed that the sides of these trenches were cut vertically, probably using back hoe equipment. The specification only requires that "the width of the trench shall be such as to permit the pipe to be laid, jointed, bedded and backfilled". Compaction of the backfill was to be to 95% proctor optimum.

The relative compressibility of the backfill material when compared to the natural material is important. Unless the compressibilities are similar, then differential settlement must occur over short distances, resulting in tensile strain, with a high likelihood of cracking of the liner.

In this case, it has been admitted that the compaction of the backfill was not carried out completely to specification. A more compressible zone resulted which may have led to cracking of the liner.

B. 3.1.2 Stress Concentrations

B. 3.1.2.1 Structure - liner interfaces

Where structures perforate the clay liner problems are likely to occur as a result of the difficulty of compaction around the structure, the sealing up to the structure, and the rigidity of the structure compared to the liner and supporting soil.

There are two structures penetrating the liner. One is a 48 inch diameter pipe and the other is two 24 inch diameter pipes. These are supported at the clay liner by a concrete slab. The detail is shown in Figure 8. Compaction around these pipes was specified, but compaction adjacent to pipes is notoriously difficult. No cutoffs were provided on the pipes, and more plastic material was not specified for the back fill soil.

A construction error occured where the two 24 inch diameter pipes intercepted the liner. The concrete slab was intended to lie on a minimum of 3 ft. of clay liner. Instead the liner was run 3 ft thick parallel to the face, leaving no impermiable soil beneath the slab. This is sketched in Figure 3.15.Prominent flow channels developed down the sides of the concrete creating a gap of approximately 1 inch. Repair was attempted by placing large quantities of concrete in the area. This reduced the flow from the underdrain in the area by approximately 20 gals/min. (See Figure 3.14)

It is not known whether the gap was created by material washing out, or by shrinkage of the liner. However shrinkage appears unlikely since the material was not allowed to dry substantially after placement. This point will be considered later under liner piping.

B. 3.1.2.2 Changes in slope

In general sharp changes in slope lead to stress concentrations which may produce tensile cracking. Sharp corners should be rounded off and sudden changes in grade avoided. At a number of locations in the reservoir this practice has not been observed, notably at the junction between the side slopes and the reservoir bottom. With a brittle material this is an unwise practice.

B. 3.2 Liner Piping

B. 3.2.1 Dispersive Clay

Clay soils will not generally erode under quite high velocities. However dispersive clays and silts are both subject to erosion where concentrated flow occurs. A number of mechanisms have already been noted for the formation of cracks in the liner. Local hetrogeneities in the liner could also provide concentrated flow. The liner material was tested for soil chemistry and dispersive characteristics. Negligible erosion was reported during a series of pin hole tests. In addition the sodium absorption ratio and the exchangeable sodium percentage were low; (1.12 and 1.23 respectively). The samples were taken from five test pits in the clay till borrow area, and should be representative of the average liner properties.

The pin hole test is not regarded as a sufficient test for dispersive soils. Sherard and Decker (1976) suggest that the Soil Conservation Service dispersion test, the crumb test and a test of the dissolved salts in the pore water, in addition to the pin-hole test, should be run on each specimen. However they agree that the pin hole test is the most reliable.

As noted earlier, pot holes were observed in the base of the reservoir, and material was most probably carried away from the liner where the two 24" pipes perforated the liner.

The fact that clayey material was removed from the liner at both these locations incidates that the liner material is erodible, and that the filter at these points was not fully effective.

3.2.2 Filter

The liner is underlain by a gravel blanket. Some grading curves for this layer are shown on Figure 3.12.The material generally satisfies the filter requirement for the silty clay liner. However in some cases it is poorly graded with few sand sizes. This material may still act as a filter but testing would be required to confirm this. The lack of sand sizes is particularly important when considering the sealing of filters under dispersive clays.

The mode of action when a pipe develops in a dispersive soil protected by a filter has been described by Sherard et al (1976). Laboratory filter tests on compacted dispersive clay samples discharging into a medium sand filter under low head all gave the same action. That is

(a) At the beginning of the test colloidal coloured water came directly through the filter. The volume of flow was limited by the permeability of the filter.

(b) The flow rate gradually decreased reaching a value of approximately one tenth the original rate within 10 - 15 mins. Clear flow continued at a low rate.

(c) On examination of the samples, it was observed that the hole in the compacted clay specimen was partially or completely filled with soft mud, and that a thin skin of fines had formed on the face of the filter.

The filter apparently did not prevent the colloidal particles passing through it. However, silt sized particles carried in the flow could not enter the sand filter and gradually sealed of the leak.

It is probable that this is the type of action responsible for the formation of the pot holes in the liner. Detailed examination of the holes would be required for confirmation, but it seems very likely that the liner material is, at least in some zones, dispersive.

3. 4. Probable Sequence of Events Leading to Failure

Upon first filling differential settlement urred in the liner, certainly above the main drain system and possibly at other locations. The differential settlements would have occured rapidly, and cracking would have developed as tensile strains developed.

Preferential seepage paths were thus created along the cracks, above possible voids in the filter, and also where heterogeneities occurred in the liner. Gradual erosion of the dispersive clay into the gravel would have continued until a pipe developed. At this stage a rapid increase in discharge occured carrying out silt with the clay particles. These silt particles gradually sealed the pipes leaving a steady clean seepage flow.

Pipes would continue to develop at different rates at locations where cracks were not as severe, or where local heterogeneities led to preferred seepage paths.

B. 5. Conclusions

From the previous considerations, the following points are made -

1. The liner material is brittle and subject to erosion once cracked. Recognition of this fact is central to the correct design of the reservoir liner.
2. To maintain the liner integrity all differential movements must be limited. Control of material selection and placement, together with attention to detailing at any rapid changes in section, such as drains, structures, slope changes are necessary, but may not be sufficient.
3. Since cracking will be difficult to control and should be expected, an adequate filter system should be used to prevent serious damage to the reservoir.
4. A monitoring system which checks the flow from known parts of the liner should be used. Liner repair can then be expidited. To this end, the monitoring system must be independent of seepage not originating from the liner. An impermiable barrier beneath the drainage blanket would achieve this.
5. The specification of an acceptable permeability for a liner may not be reasonable, particularly when based on a plasticity index to permeability correlation. Care in the selection and placing of the material would be a better way of assuring the required property.

B. 6. Possible remedial measures

Remedial measures would have to be based on substantive information. However, if it assumed that the liner is disperse then a number of alternatives are available.

B. 6.1 Membrane Liner

The entire reservoir could be covered with an impermeable liner, protected by a soil cover. This alternative is bound to be expensive but good results could be assured.

B. 6.2 Chemical modification

Chemical treatment of dispersive soils have been used successfully to prevent erosion. Sherard and Decker (1977) contains a number of papers dealing with this problem. The choice of chemical and treatment method depends on the soil chemistry.

B. 6.3 Bentonite liner

Bentonite is commonly used as a sealing agent for reservoirs. This is discussed in detail in Section 1.3.1 of the report. The bentonite may in this case be subject to piping also, and laboratory and field tests over a period of years would be needed to establish this fact.

Figure 3.8 : GENERAL LAYOUT, SHOWING UNDER DRAINS
(Case History B)

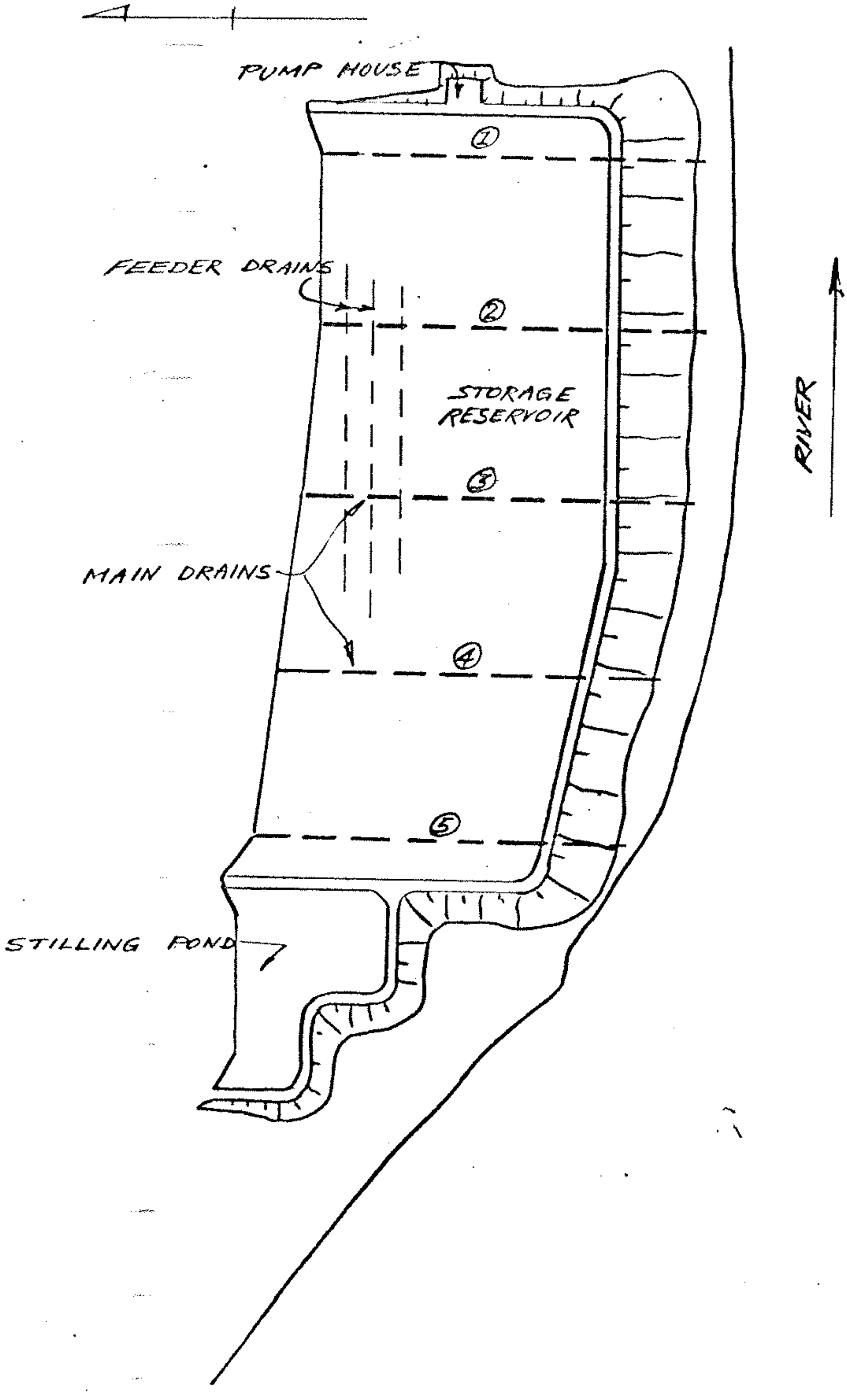


Figure 3.9 : TYPICAL CROSS-SECTION
(Case History B)

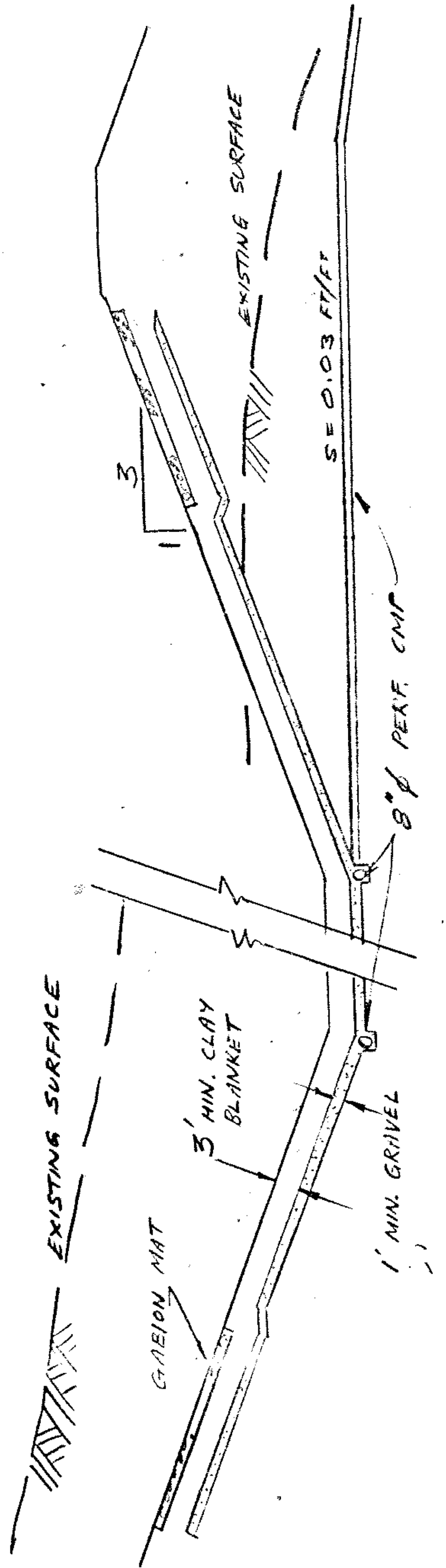


Figure 3.10 : CASAGRANDE CLASSIFICATION FOR LINER MATERIAL
(Case History B)

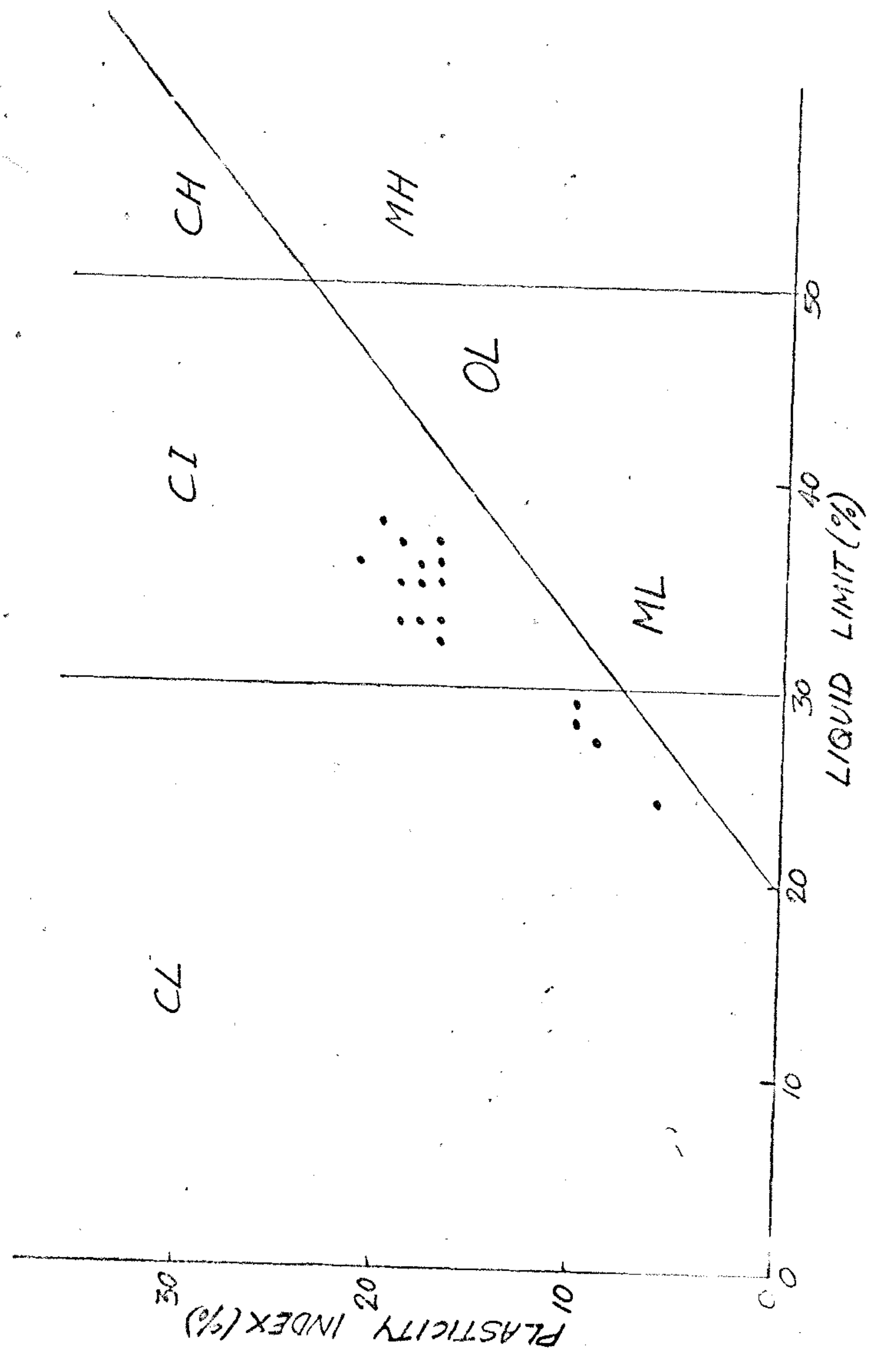


Figure 3.11 : RANGE OF PERMEABILITIES, LINER MATERIAL
(Case History B)

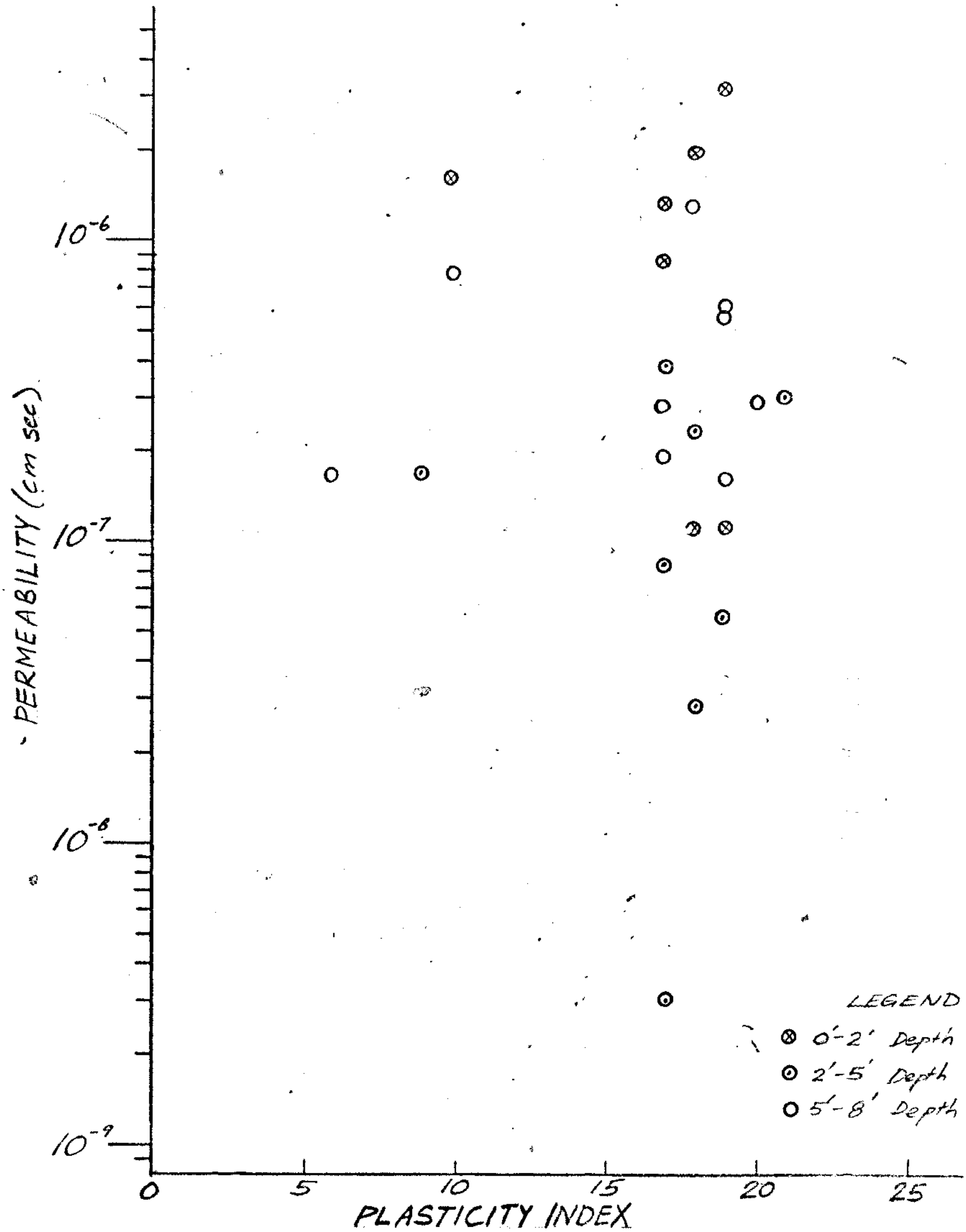


Figure 3.12 : GRAIN SIZE CURVES FOR LINER AND FILTER MATERIALS
(Case History B)

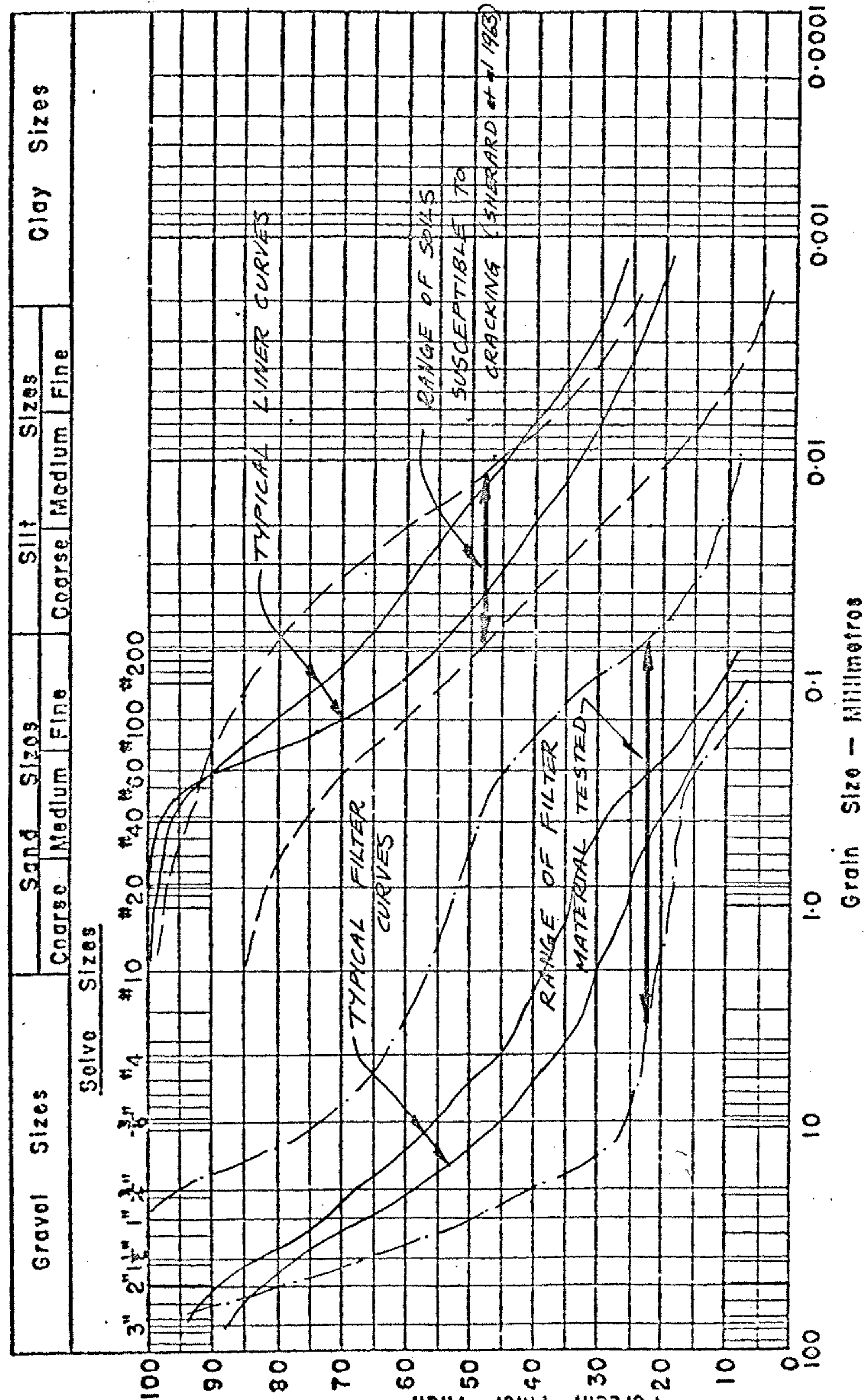


Figure 3.13 : DEPTH OF GRAVEL LAYER BENEATH CLAY LINER
(Case History B)

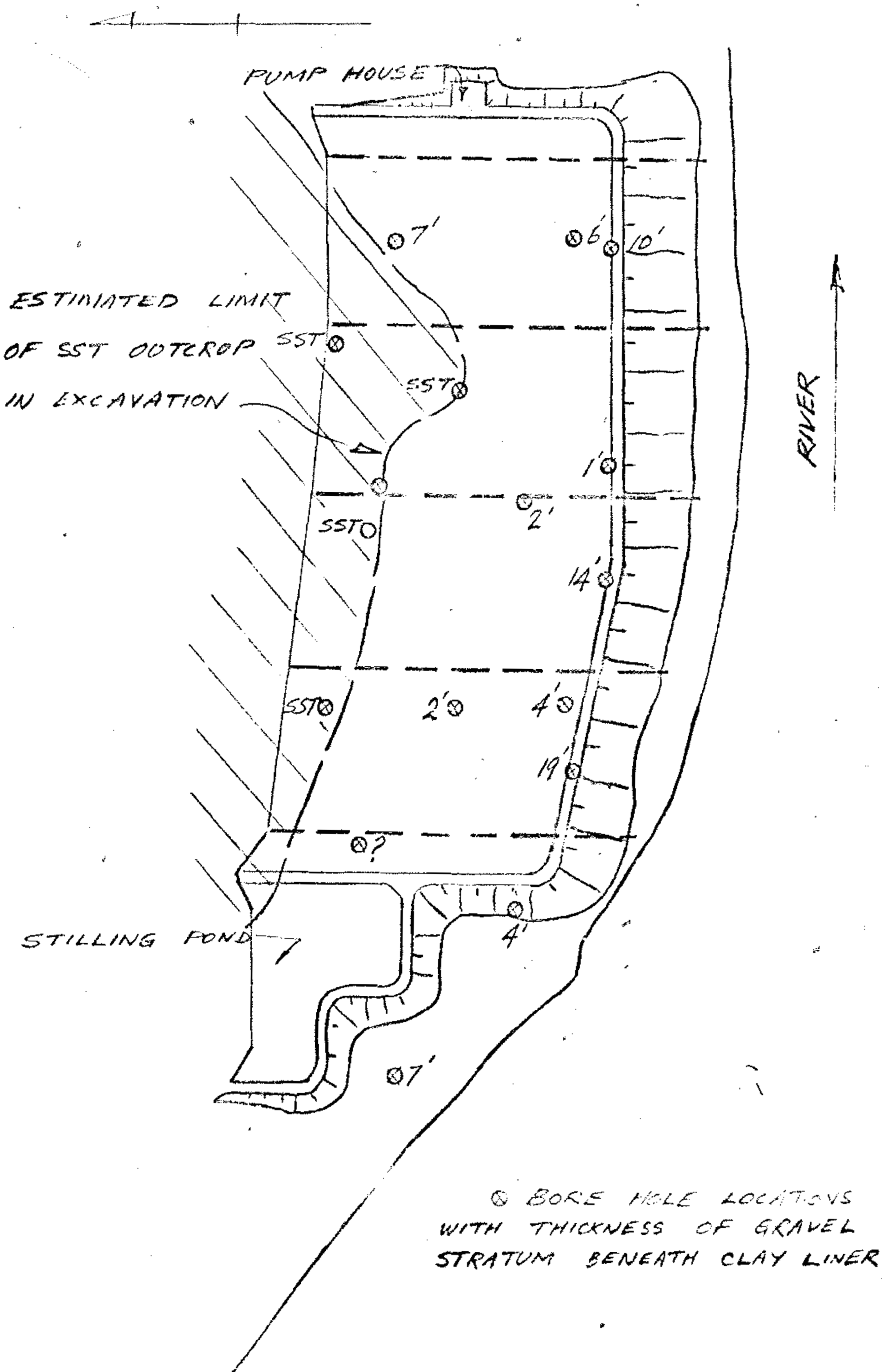


Figure 3.14 : FLOW RATE FROM UNDER DRAINS

(Case History B)

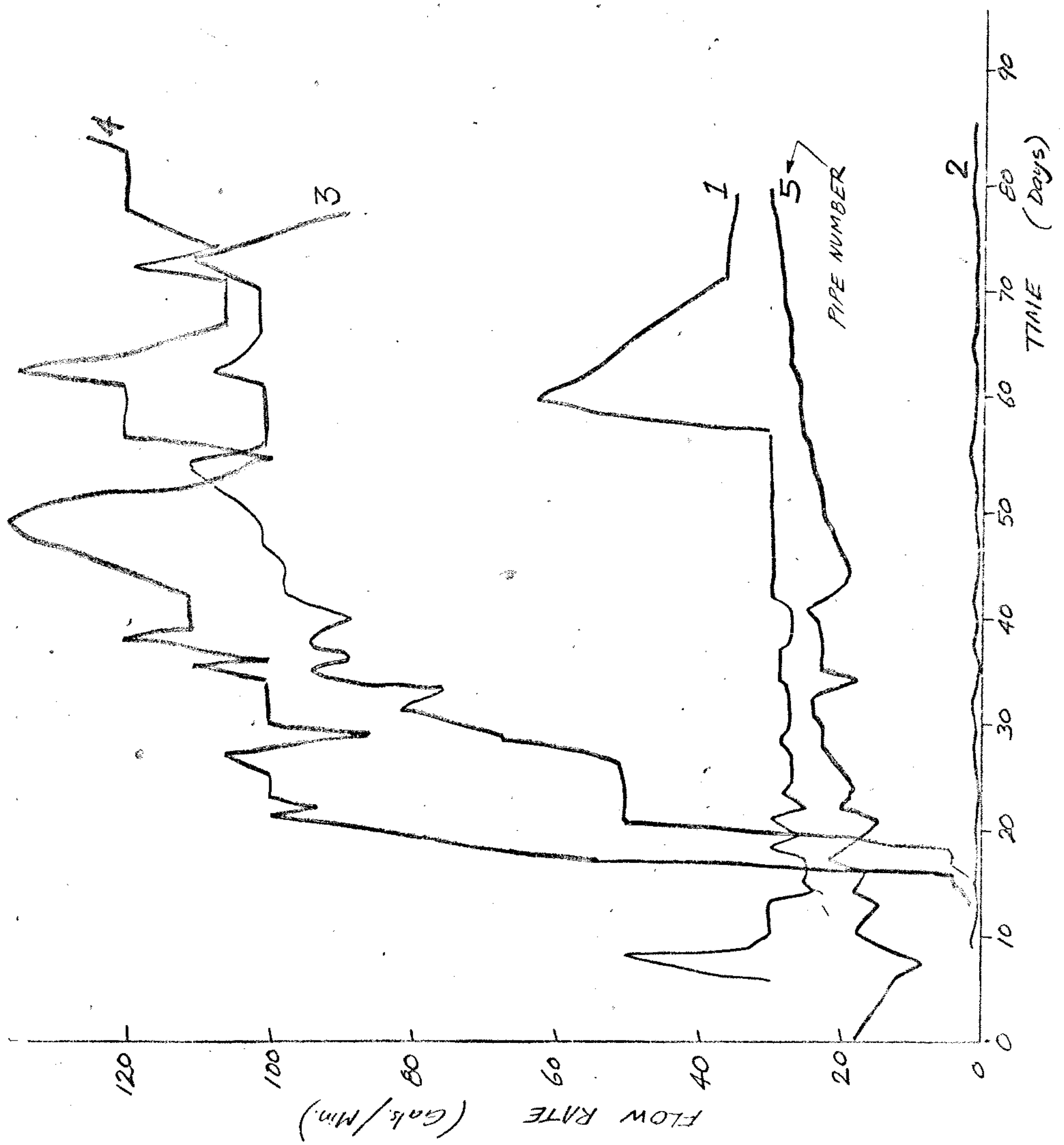
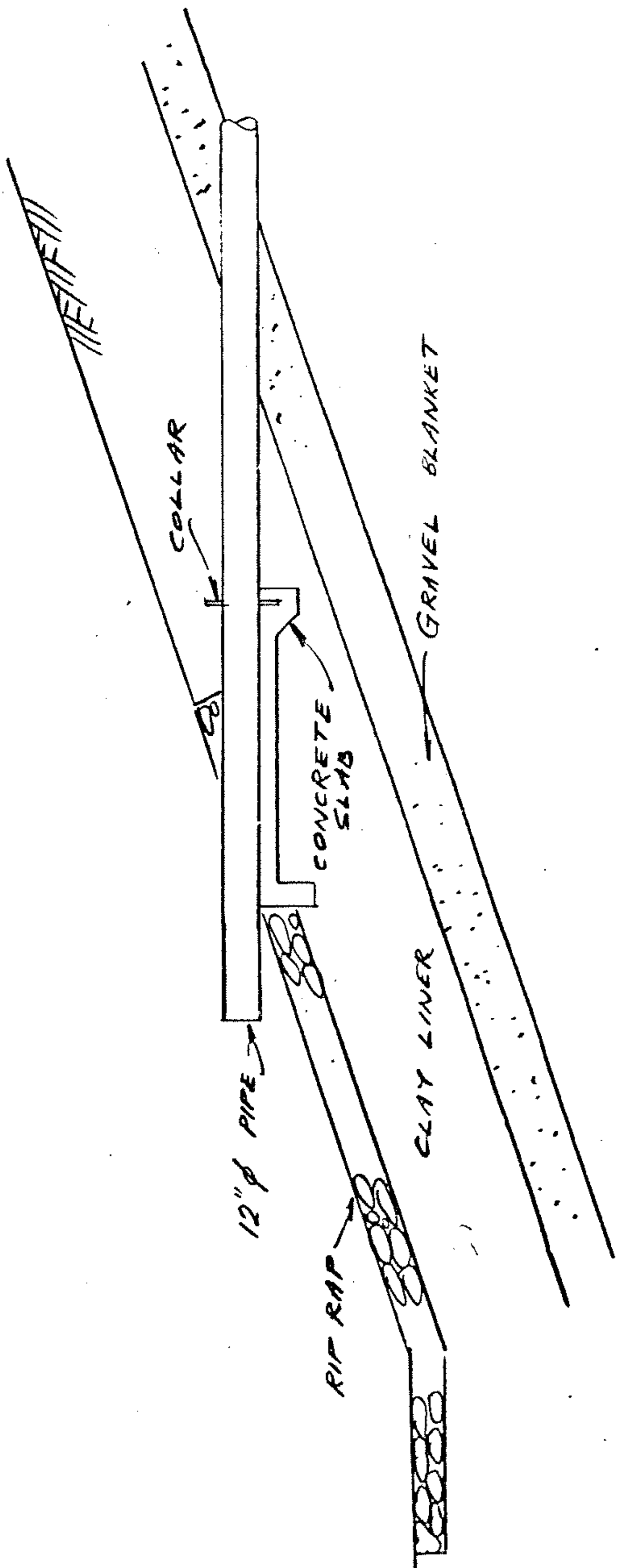


Figure 3.15 : <u>INLET PIPE AS BUILT</u>

(Case History B)



CASE HISTORY C

An aerobic cell for a sewage treatment facility was built on a terrace adjacent to a river valley wall in Central Alberta. The cell sustained minor damage due to beaver tunnelling which was readily repaired. Severe damage occured when a slide of the river valley wall affected the cell embankment.

C. 1. Description of the Cell

The cell measured approximately 150 m x 210 m and could hold a maximum depth of 2.5 m of waster water. The general layout, is given in Figure 3.16

C. 1.1 Site Description

Soils in the region of the cell include silty sand, silt and clay typical of glacial outwash deposits. The layering is complex, but the predominant material is silty sand. Organic topsoil and peat occured over portions of the site.

The slope adjacent to the cell is approximately 23 m high with an average slope of between 10 to 15 degrees. The slope contains complex layering of silty sand, silt and clay over bedrock. Bedrock consists of fine grained weathered bentonitic siltstone and sandstone. The average thickness of the material overlying the bedrock is 5 to 9 m as shown in Figure 3.17.

Water levels in the area were not accurately determined prior to construction. The complex layering meant that perched water tables were present.

However with the limited information available, it can be inferred that the average level of the ground water table under the cell was about 4 m above the impermiable till basement.

C. 1.2 Construction and Operation of the Cell

The original geotechnical report recommended that the site be stripped of all organic material, and that a 0.5 m thick clay liner be installed. The liner material was not specified except at "an impervious clay". After failure of the cell, additional site investigation showed that the liner had been omitted, and that some organic material was present under the embankment.

The cell was never fully utilised. About four years after construction water levels reached to about the half way mark. Some leakages was noted, an a beaver nest was discovered in the west dyke, and the pond level was lowered for successful repair work.

C. 2. Description of Failure

About five years after construction a 120 m long crack developed in the crest of the north dyke, and the ground dropped up to 3.7 m. The extent of the slide is shown in Figure 3.16.

Seepage was noted at the base of the slide, and the slide appeared to be confined to the soil above the bedrock.

C. 3. Causes of Failure

The immediate cause of the failure of the cell was the activation of the slope. Some possible causes of this activation are

- The surcharge effect of the dyke embankment at the head of the slope.
- Oversteepening of the slope, caused by roadway construction cutting into the toe of the slope.
- Increased water levels in the slope, due either to seepage from the cell or natural ground water surface fluctuations.

The first two factors would have reduced the factor of safety of the slope. However the slide was not triggered at the time these actions were taken, and so it is most probable that increasing water levels in the slope were responsible for a decrease in the factor of safety with time.

C. 4. Seepage in the Slope

Observations at the cell site showed that the water table had risen from approximately 4 m to 9 m above the impermeable till. This rise was due to the formation of a mound caused by infiltration from the cell. As a consequence of this raised water table under the cell, the water level in the slope increased sufficiently to cause failure. Water levels in the slope at the time of failure are not known.

It is interesting to note that even if the originally specified liner had been incorporated in the cell, the mound development beneath the cell would have reduced the factor of safety of the slope to unacceptable levels. An analysis, by others, for this case has been summarised in Appendix A.

C. 5. Conclusion

To have prevented the slope failure, seepage from the cell should not have been allowed to reach the slope. An imperm able liner, with an effective permiability of less than 10^{-8} cm/s, such as a plastic membrane could have been used, or seepage could have been prevented from reaching the slope by a deep continuous drain in front of the cell embankment, or the use of a cutoff between the top of the slope and the reservoir.

Figure 3.16 : General Layout
(Case History C)

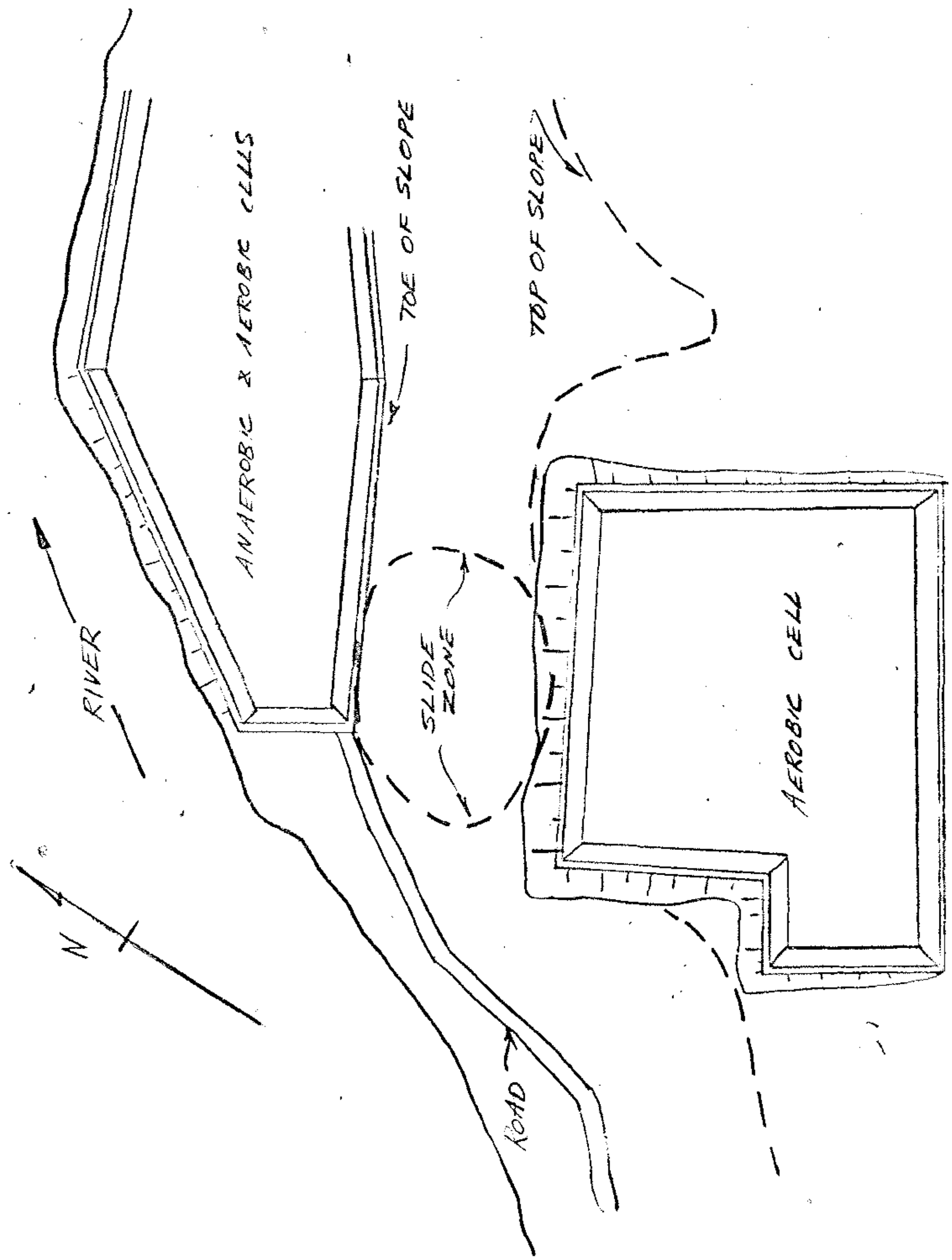
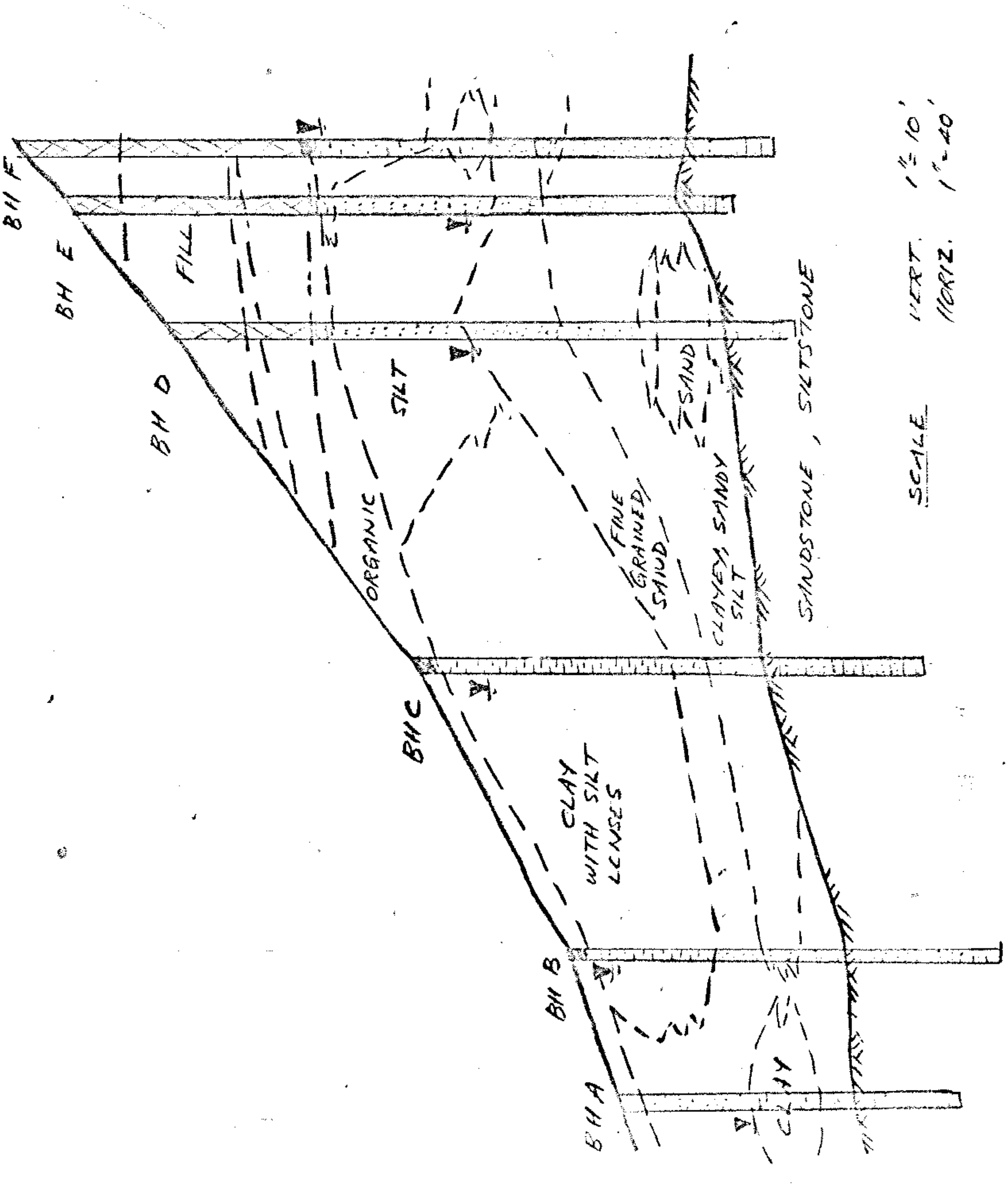


Figure 3.17 : TYPICAL SECTION OF SLOPE

(Case History C)



CASE HISTORY D - Baldwin Hills Reservoir

The Baldwin Hills Reservoir in Los Angeles failed in December 1963. It has been the subject of numerous investigations including State of California (1964) and has been reported in the technical literature by Jansen et al (1967), Leps (1972) and Casagrande et al (1972) amongst others. Readers are refered to the above articles for details of reservoir construction, ground movements, geology and site investigation results, as well as descriptions of the events leading to failure. The following is a brief summary of the above papers.

D.1 Reservoir Construction

The underlying soils in the Baldwin Hill area were known to be dispersive and susceptible to piping. The philosophy of the design was therefore to prevent any seepage from the reservoir reaching those soils. To achieve this an impervious lining and an underdrain system were designed.

The cross-section of the reservoir is shown in Figure 3.18.The porous asphaltic paving was to protect the earth liner, which was made by blending and compacting fine grained soils excavated on the site. A 4" cemented pea gravel drain beneath the liner was to act as a collector for all seepage. Tile drains were then to carry the water out of the reservoir area. The pea-gravel drain was capped with a ¼" porous sand gunite layer to prevent the compacted earth from infiltrating, and underlain by a ¼" asphaltic membrane to prevent all water from reaching the foundation soils.

D.2 Geology of the Area

The Baldwin Hills are a combined anticlinal and fault block structural and topographical uplift. A geological section through the area is shown in Figure 3.19.Although seismic activity is present in the area there has been no reliable instances of surface faulting or creep along the faults of the zone in recent times.

D.3 Subsidence at Baldwin Hills

The anticlinal feature shown in Figure 3.19 has been the site of oil accumulation and was developed as an oil field from 1924. Associated with the extraction of oil was the development of a bowl of subsidence in the region. This bowl is shown in Figure 3.20. The location of the reservoir is shown on this Figure. The vertical differential movement over the lifetime of the reservoir was as much as 6½".

Horizontal tensile straining developed with the formation of the subsidence bowl. The magnitude of the tensile straining differs with different authors from 0.02% (Leps 1972) to 0.07% (Casgrande et al 1972) during reservoir operation. This tensile strain appears to have been concentrated in the faults, causing them to open by as much as ¼" to ½".

D.4 Principal Events Leading to Failure

The integrity of the reservoir depended primarily on the ¼" thick asphaltic membrane. During construction, difficulty was experienced in preventing cracks and horizontal displacements.

During first filling excessive leakage was noted, and upon draining the reservoir compression cracks and bulges, as well as settlement on the south side of the outlet tower, were observed. Repairs were made to the earth liner at this stage. It is probable that at this time the asphaltic membrane and relatively brittle pea-gravel drain were ruptured by differential vertical movement along the faults, produced by the combined effect of the water load and the greater compressibility of the soils on the west side of the faults.

The pea-gravel drain then acted as a collector to channel the seepage water into the foundations soils. The wetting of these soils caused additional differential settlement and the development of cavities and pipes beneath the reservoir. This progressive erosion led eventually to the collapse of one or more of these cavities which in turn breached the earth liner allowing full hydrostatic pressure to develop in the faults.

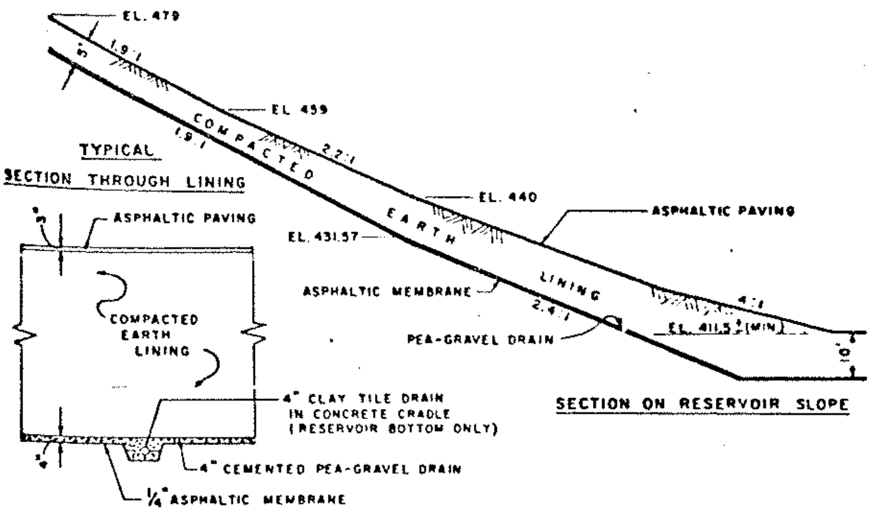
Once the earth liner was breached, the volume of water flow was substantially increased and erosion proceeded apace, either through stoping from an existing cavity, or flow through a tension crack until a continuous pipe was formed through the entire abutment.

D.5 Comments

The movements to which the reservoir were subjected are not uncommon, and would not harm most reservoirs and dams. However, the pea-gravel drain in this dam was brittle, and the differential movements of the faults caused distinct breaks in the drain, and the attached underlying asphaltic membrane, Once the membrane was broken, failure was inevitable. A suitable design would have had to allow sufficient flexibility to accommodate the movements which occured.

Figure 3.18 : <u>TYPICAL SECTION</u>

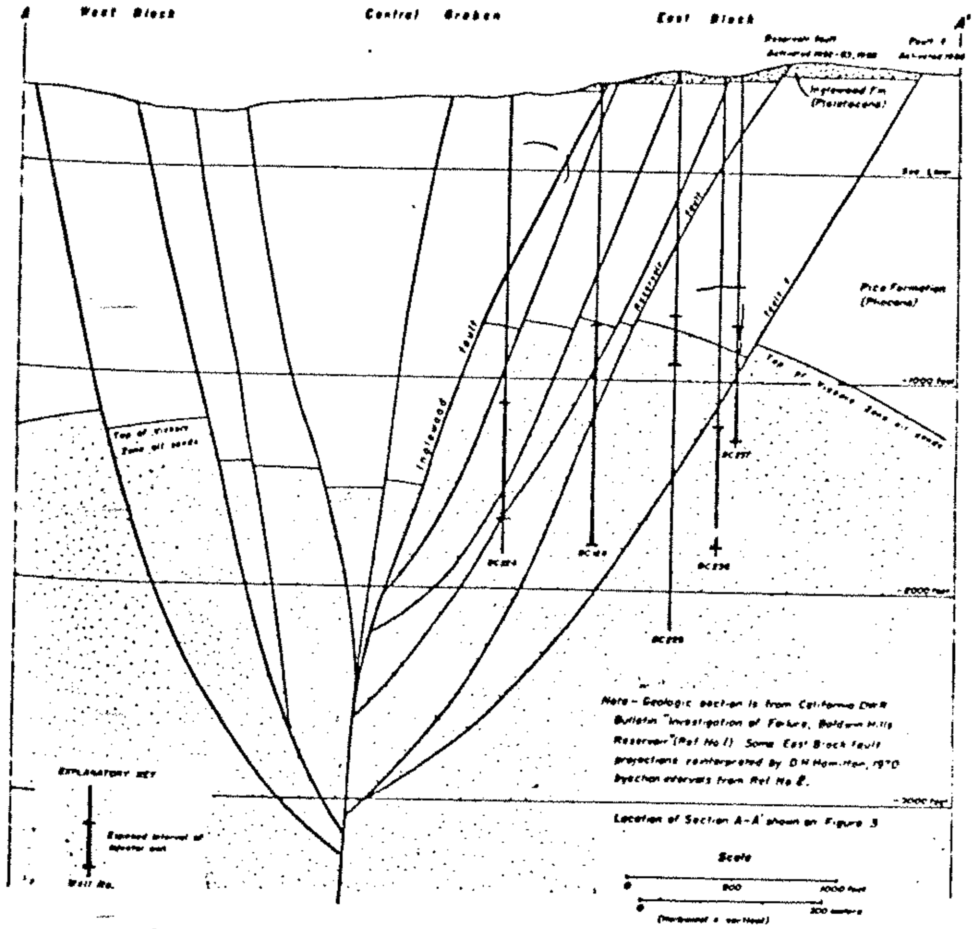
(Case History D)



(from Jansen et al 1967)

Figure 3.19 : GEOLOGICAL CROSS-SECTION OF BALDWIN HILLS REGION

(Case History D)

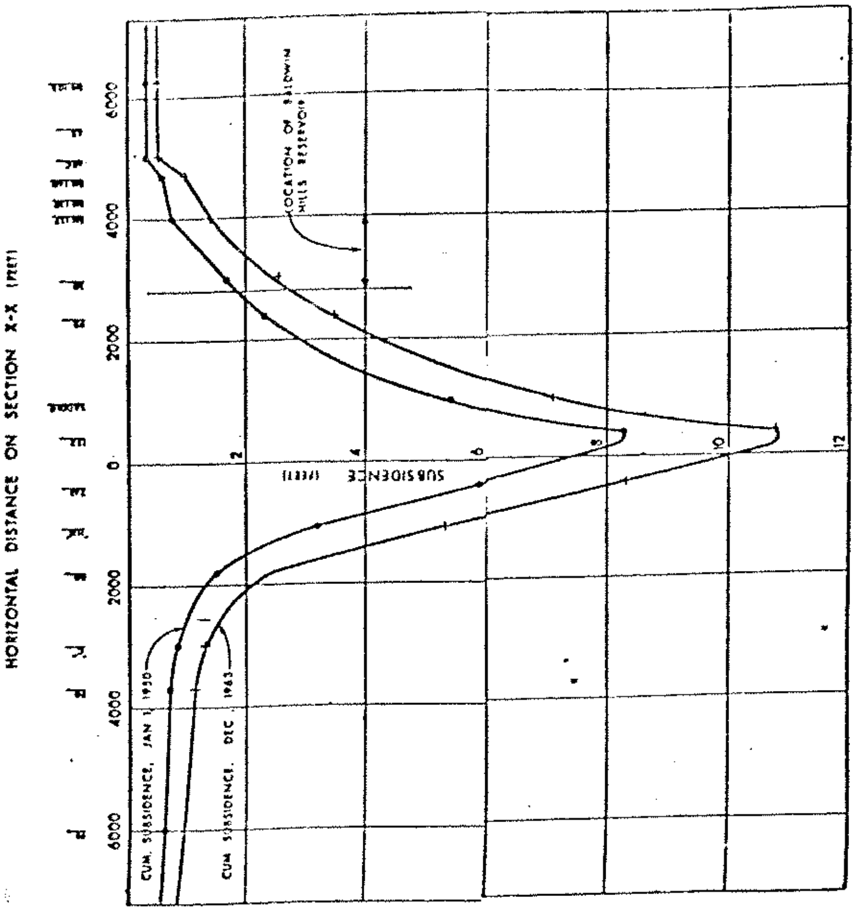


Geologic structure section through the Baldwin Hills-Inglewood area and the Inglewood oil field.

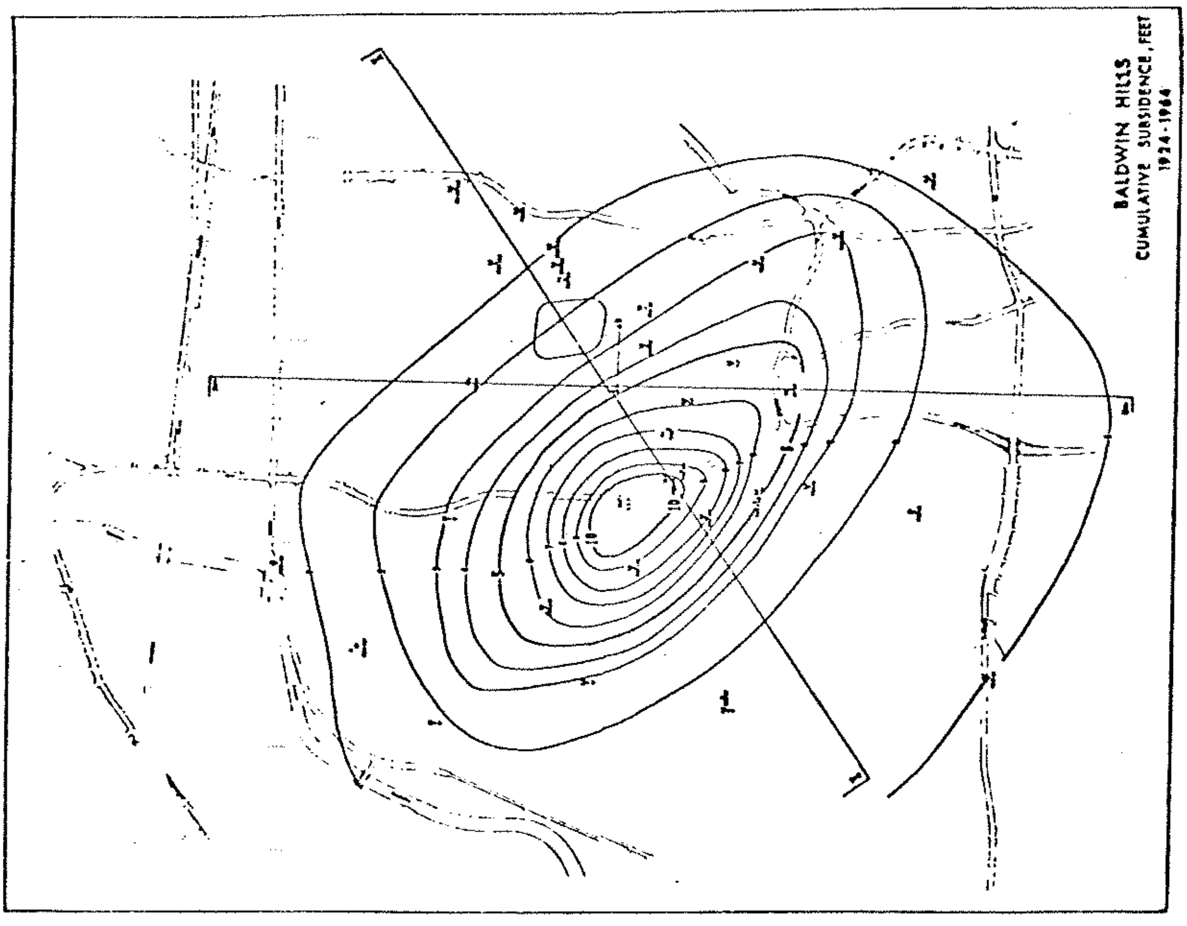
(from Leps 1972)

Figure 3.20 : SUBSIDENCE BOWL, BALDWIN HILLS

(Case History D)



BALDWIN HILLS
SECTION X-X THRU SUBSIDENCE BOWL
SECTION ON LINE N. 56° E.
THRU CENTER OF BALDWIN HILLS RESERVOIR



CHAPTER 4 - CONCLUSIONS

Lined reservoirs are now commonly used as engineering structures. The methods of design and construction are well established using known soil mechanics and construction principles, obtained from other water retaining structures.

The major determining factors in the performance of a lined reservoir are the type of liner material used, and the differing environments to which the liner will be subjected. These factors are obviously interrelated. If the environment in which the liner will have to perform, and the loadings which will be applied to it, can be accurately specified, then the liner choice and design should follow readily.

The selection of the liner type for the particular job is therefore of paramount importance. Of the four case histories examined, three can be attributed directly to misunderstanding of the behaviour of the critical liner parameter under the conditions to which they were exposed, be it durability, permeability or rigidity.

All liners must be expected to leak to some extent. Tears, construction errors and deterioration of the liner all lead to increased permeability. Care must be taken to determine where the leakage will go, and what effect it will have on the structure, and particularly with pollutants, on the environment. Adequately designed subdrain systems can facilitate monitoring and control of leakage.

Finally, it is believed that there is a tendancy amongst engineers to regard reservoirs as relatively minor structures. As a consequence of this attitude insufficient attention may be paid to detailing. It is the attention to detail in a design which ultimately determines the soundness on the whole. The failure of reservoirs containing even small volumes of some pollutants could have very wide reaching reprecussions.

APPENDIX A: MODEL FOR LAGOON AT THE TOP OF A SLOPE

A lagoon located at the top of a slope underlain by an impermeable stratum can be modelled as shown in Figure A1.

A mound will develop beneath the lagoon causing an increased water level in the adjacent slope. Closed form solutions are available for the steady state mound formed by infiltration and for the flow between differential water levels on an inclined surface (Harr 1962). These solutions may be coupled using an infinitely small, fully penetrating slot to satisfy the boundary conditions of flow and pressure between the two flow zones.

For the model 1 shown in Figure A1 it has been assumed that there is no change in the water level at the edge of the lagoon away from the slope.
The flow in the slope is given by:

$$q_1 = k_s \, \alpha \, h^* i$$

where α was assumed constant for the problem

k_s is the soil permeability
i is the impermiable strata gradient
h^* is the depth of water in the fully penetrating slot

The flow under the lagoon is:

$$q_2 = k_s \frac{(h_1^2 - h^{*2})}{2L_2} + \frac{eL2}{2}$$

where e = infiltration per unit area

The infiltration e will depend on the average hydraulic gradient causing infiltration ($\bar{i}$) and the effective permeability of the clay liner, or the liner-subsoil combination ($\bar{k}$)

$$e = \bar{k} \, \bar{i}$$

For continuity

$$q_1 = q_2$$

i.e. $$\alpha \, k_s \, h^* \, i = \frac{k_s}{2L_2} (h_1^2 - h^{*2}) + \frac{\bar{k}\bar{i}L_2}{2}$$

This may be solved for h^*

$$h^* = \frac{-b + \sqrt{b^2 - 4_c}}{2}$$

where $$b = 2L_2 \, \alpha \, i$$

$$c = -(h_1^2 + \frac{\bar{k}}{k_s} \, \bar{i} \, L_2^2)$$

Figure A2 shows the relationship between h^* and the permeability ratio $\bar{k}/k_s$ for liners 0.5m and 2.5m thick when the depth of water in the lagoon is 2.5m and h_1 is taken as 3.6m.

For Model 2, Figure A1, it was assumed that the water levels at both boundaries of the infiltration zone are changed by an equal amount Δh^*

Solving as above gives

$$\Delta h^* = \frac{\bar{k}}{k_s} \frac{\bar{i}\ L_2{}^2}{c_2}$$

where $c_2 = 2h_o - 2h_1 + \alpha\ i2L_2$

The relationship between $h^* = h_o + \Delta h^*$ and the permeability ratio $\bar{k}/k_s$ are shown for the same cases as Model 1.

It can be seen that the effect of the assumption of water level rise away from the slope has minor effect on the height of water in the slope.

The factor of safety of an infinite slope of cohesionless material with seepage parallel to the slope is given by:

$$F = (1 - r_u) \frac{\tan \phi}{\tan \alpha}$$

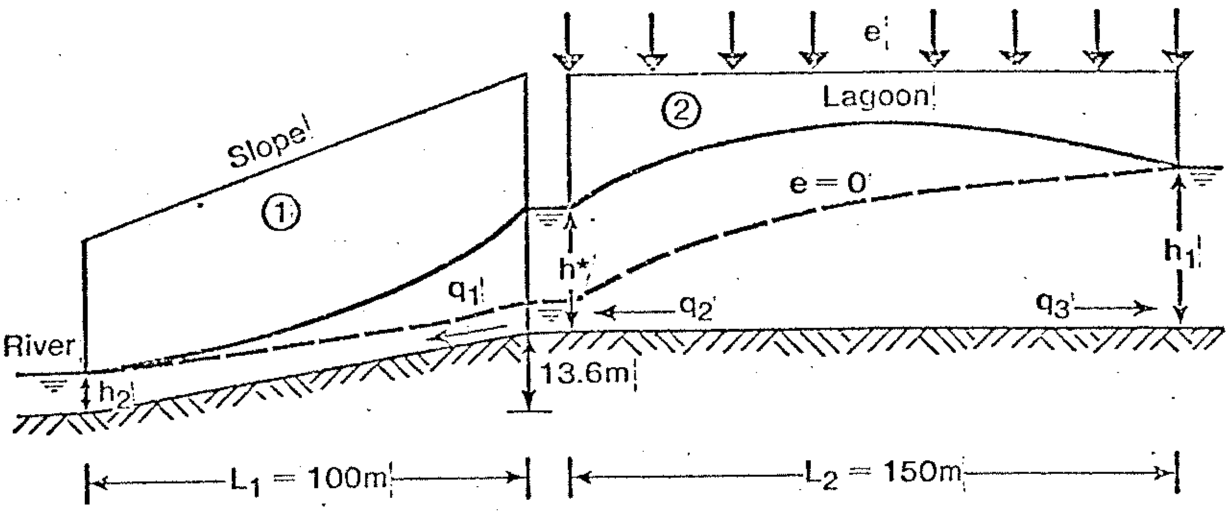
where
α = surface slope
ϕ = angle of interval friction of the soil
r_u = pore pressure ratio = $\frac{u}{\gamma d}$
u = pore water pressure
d = depth to failure surface
γ = soil density

For a particular slope the factor of safety decreases as the water level in the slope increases.

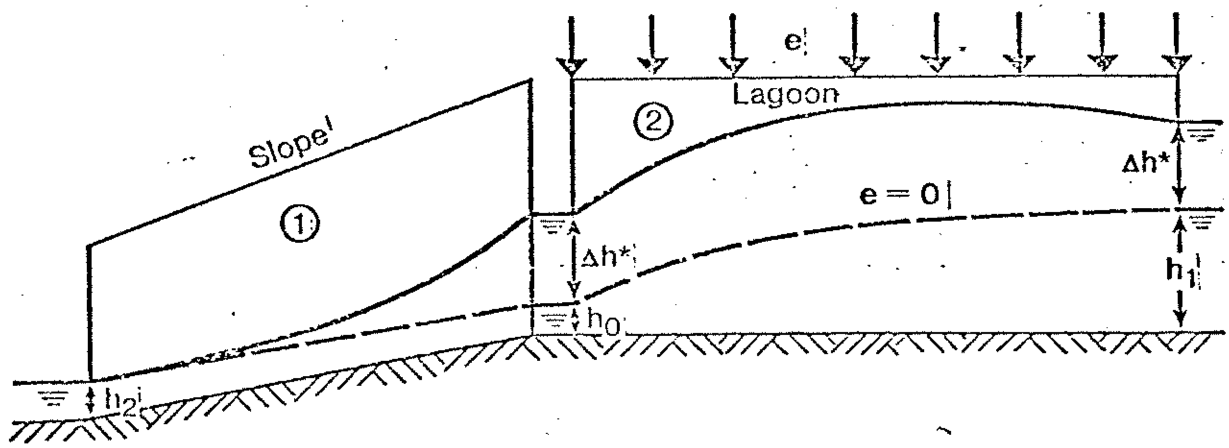
As an example, if a factor of safety of greater than 1.5 is acceptable in a 10° slope of sand with $\phi = 25^\circ$ and $\gamma = 1.75\ gm/cm^3$, then a water level in the slope of greater than or equal to three quarters of the depth to the failure surface will cause an unsafe slope. The effect of perched water tables in such a slope is of major importance.

Once an acceptable water level in the slope has been selected, reference to the previous equations plotted for specific values in Figure A2 enables the selection of a suitable permeability ratio k/k_s and hence the selection of a suitable liner type.

FIGURE A1 : Models for Lagoon above a slope

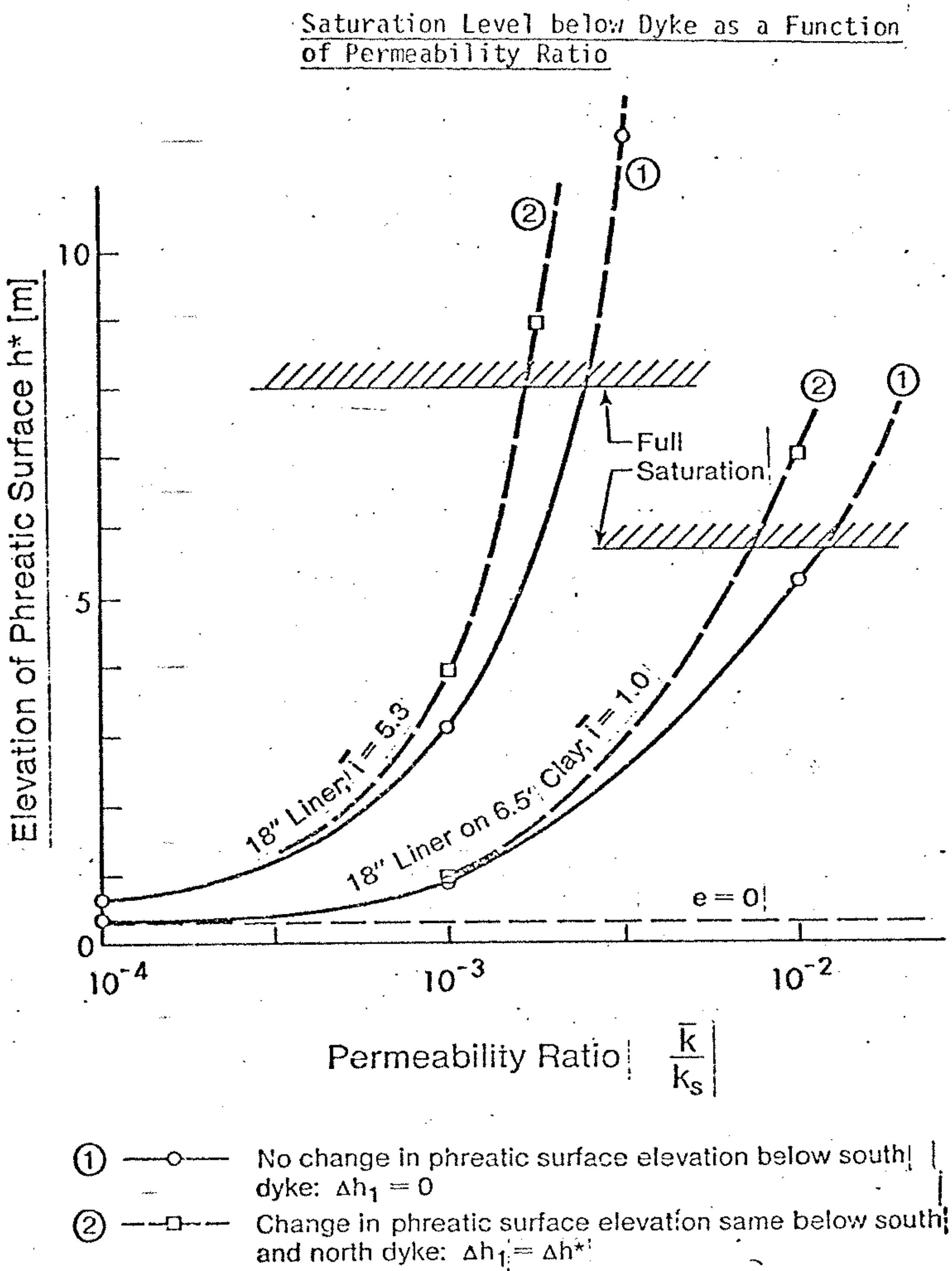


a) No change in water level at edge of lagoon away from slope.



b) Equal rises in water level on both edges of the lagoon.

FIGURE A2 : Phreatic Surface in Slope as a function of liner permeability.



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