DEPARTMENT OF ENERGY, MINES AND RESOURCES, OTTAWA

MINES BRANCH

TENTATIVE DESIGN GUIDE FOR MINE WASTE EMBANKMENTS IN CANADA

PREPARED FOR THE
MINES BRANCH MINING RESEARCH CENTRE

MARCH 1972

TECHNICAL BULLETIN TB 145

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TENTATIVE DESIGN GUIDE FOR MINE WASTE EMBANKMENTS IN CANADA

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Keywords: WASTE EMBANKMENTS, STABILITY, RECLAMATION, DESIGN.

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FOREWORD

Early in 1968 the Canadian Advisory Committee on Rock Mechanics established a sub-committee to review the requirements for improved practices with respect to waste embankments. The members of the sub-committee were: C.O. Brawner, Golder, Brawner & Associates Ltd.; K. Davies, Cominco Ltd.; L. Dwarkin, Kaiser Resources Ltd.; G. Godfrey, Rio Algom Mining Ltd.; R. Harris, Cassiar Asbestos Ltd.; K. McRorie, Wright Engineers Ltd.; W. Robinson, B.C. Dept. of Mines. After gathering information from industry, through a questionnaire and otherwise, they reported their findings and recommendations.

The CACRM's report to me in 1969 showed that, whereas research on several aspects were required for good embankment design, much applicable technology was already available that could be applied by mine engineers for the design, construction and maintenance of waste embankments. Accordingly, the Mines Branch felt that the drafting of a design guide would be of assistance to the provincial mines inspectors and to the mining companies. The Mines Ministers' Conference of 1970 and the National Advisory Committee on Mining and Metallurgical Research confirmed this view.

Accordingly, the Mines Branch commissioned the drafting of a report, which was done by consultants in a very short time. After circulating the draft report for comment, it was edited and the Design Guide prepared as a public document.

Many individuals and organizations have assisted in creating this volume - too many to provide appropriate aknowledgements for each. Perhaps it is sufficient to say that the consultants are to be commended for having done a very good job and to thank the final reviewers: Mr. J.M. Fletcher, Canadian Johns-Manville Co. Ltd.; Mr. B. Hoare, Bert Hoare Consulting Engineers; Prof. L. Juteau, Ecole Polytechnique; and Dr. N. Morgenstern, University of Alberta.

As mining economics are changing and new information is being produced by research, it is planned to revise this guide within a few years. For this reason the present editing has been kept to a minimum. We will also be looking for practical criticisms from those in industry who try to follow these recommendations.

John Convey,
Director, Mines Branch

AVANT-PROPOS

Au début de l'année 1968, le comité consultatif canadien de la mécanique des roches a établi un sous-comité pour réviser les exigences pour les practiques améliorées en ce qui concerne les terrils. Furent membres du sous-comité: G.O. Brawner, "Golder, Brawner & Associates Ltd."; K. Davies, "Cominco Ltd."; L. Dwarkin, "Kaiser Resources Ltd."; G. Godfrey, "Rio Algom Mining Ltd."; R. Harris, "Cassiar Asbestos Ltd."; K. McRorie, "Wright Engineers Ltd."; W. Robinson, "B.C. Dept. of Mines". Aprés avoir rassemblé les renseignements de l'industrie, à l'aide d'un questionnaire et par d'autres moyens, ils ont rendu publiques leurs découvertes et leurs recommandations.

Le rapport du comité consultatif canadien de la mécanique des roches qui m'a été envoyé en 1969, a indiqué que, tandis que la recherche sur plusieurs aspects était nécessaire pour un bon plan du terril, beaucoup de technologie applicable était déjà disponible pouvant être appliquée par les ingénieurs des mines pour le plan, la construction et l'entretien des terrils. La Direction des mines a donc senti que la préparation du guide de dessin pourrait aider à la fois les inspecteurs provinciaux des mines et les compagnies miniéres. La Conférence des ministres des mines en 1970 et le Comité consultatif national de recherches miniéres et métallurgiques ont confirmé ce point de vue.

La Direction des mines a donc autorisé la préparation d'un rapport, qui a été fait en peu de temps par les consultants. Aprés avoir fait circuler le rapport provisoire pour obtenir des commentaires, on l'a édité et on a préparé le Guide de dessin comme document public.

Plusieurs individus et organisations ont aidé à la préparation de ce volume - un trop grand mombre pour pouvoir remercier chacun de facon appropriée. Peut-être, est-il suffisant de dire queles consultants sont dignes de louanges par leur excellent travail et remercier: M.J.M. Fletcher, "Canadian Johns-Manville Co. Ltd."; M.B. Hoare, "Bert Hoare Consulting Engineers"; Prof. L. Juteau, Ecole Polytechnique; et Dr. N. Morgenstern, "University of Alberta," pour leur révision définitive.

Comme l'économie miniére change et que la recherche nous donne continuellment de nouveaux renseignements, on propose de réviser ce guide d'ici quelques années. C'est pourquoi l'édition actuelle a éte distribuée de façon restreinte. Nous attendons aussi les critiques pratiques de ceux dans l'industrie qui essayent de suivre ces recommandations.

John Convey,
Directeur,
Direction des Mines

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INTRODUCTION

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SECTION 1

INTRODUCTION

SCOPE

Purpose of the Guide

The mining and processing of mineral ores generally results in the production of large quantities of solid wastes. In many cases, the processing of these ores involves grinding and the addition of water and chemicals in the ore treatment plant, with a large portion of the resulting waste leaving the plant in the form of a slurry. Usually this slurry is ponded for sedimentation of the solids, free water accumulated in the pond being pumped back to the plant, or being allowed to discharge from the pond to an adjacent water course. Other wastes, particularly overburden, native rock excavated from the orebody and coarse wastes removed from the ore during its processing are stored in waste piles. With the increase in mining activity in Canada, and the trend towards the mining of grades of ore lower than those worked in the past, waste embankments are increasing in volume and height. Problems have been encountered with many of them and, in 1969, the Canadian Advisory Committee on Rock Mechanics recommended that:

"A design guide be developed for use by mining engineers and government officials charged with the responsibility of operation and inspection of mining projects. The prime purpose of the design guide should be to outline the general aspects relating to stability, the more common types of problems which may develop and investigations necessary to evaluate each of these problems. It is recommended that a portion of the design guide be explicitly detailed to outline site investigation details, design requirements and specifications, techniques of construction, procedures of inspection and the approach to evaluate stability of existing facilities."

This Guide has been prepared as a result of that recommendation.

For the user of the Guide, it is emphasized that considerable judgment is necessary in assessing the values of parameters appropriate for use in the planning and design of mine waste embankments. Consequently, it is recommended that the investigations and design of mine waste embankments be made under the direct supervision of engineers who are competent in this field, which includes mine waste disposal techniques, geology, soil mechanics, hydrology and hydraulics.

Particular emphasis has been placed in the Guide on the adequacy of waste embankments in retaining the solid and liquid wastes held by them. Their adequacy in preventing pollution of the environment by wastes has not been treated specifically, except insofar as such a requirement may affect their design. Research is continuing on finding the ways in which this source of water pollution can be eliminated.

Mine Waste Embankments

Varying classes of materials must be wasted from mining operations. These can include soils from strip and placer mining operations, excavated rock, ground ore in slurry form and fine wet materials such as filter cake from coal treatment plants. Their characteristics will vary considerably according to their geological origin, particle size, processing and water content. The most practicable methods of disposal will depend on the characteristics and volumes of the waste materials and on the availability of storage sites. Some wastes may be used for back filling mined out spaces; many are dumped in areas adjacent to the mine. Generally, coarse dry materials are dumped in self-supporting piles while slurries are ponded behind retaining embankments. Fine wet wastes may require retention by an embankment or may be incorporated in a pile of self-supporting material depending on their characteristics. A typical system of mine waste disposal is illustrated by Figure 1-1.

As used in this Guide, "waste embankments" include all deposits of mine waste materials placed on the ground surface, including any embankments necessary to support them. However, the term does not include deposits placed underground in backfilling mined out spaces.

"Tailings embankments" are defined as embankments required to retain slurries of ore and liquid, or to retain liquid alone.

"Waste piles" include all waste embankments not intended to retain liquid, including any embankments necessary to support them. Ore storage piles, though not containing "waste" materials, could be treated as waste piles for design purposes.

SUMMARY

Basic Functions

The prime function of both mine waste piles and mine tailings ponds is to store solids. However, tailings ponds usually must provide temporary storage of a certain minimum volume of water for clarification prior to reclaim for plant use or discharge to adjacent streams. Where tailings pond effluent would be a serious pollutant, tailings embankments may have to be designed to retain as much water as practicable, reliance being placed on water reclaim and evaporation to prevent its release to surface or sub-surface water courses. If this is not practicable, it may be necessary to treat the water prior to its release from the pond.

Two extreme approaches are therefore possible in the design of mine tailings embankments - to make the embankment relatively impervious, or to make it relatively pervious. Whether one of these, or an intermediate approach is taken, the embankment must be adequately stable and necessary provisions must be made to control seepage through and under the embankment, and to control surface run-off into the pond.

In some cases, it may be desirable to promote drainage of water from tailings ponds in order to consolidate the deposits in them. This could be for the later excavation of fine coal tailings or for permanent stabilization of fine tailings which might otherwise be susceptible to liquefaction under earthquake, or other shocks.

Problems Encountered

There have been many serious waste embankment failures and stability against sliding of embankment slopes is a major consideration in the design of waste piles and tailings embankments. Such sliding failures can be caused by weak foundations, placement of the waste materials at slopes that are too steep (or of too great a height) and high piezometric water levels within the embankments or their foundations. Breaching of tailings embankments can occur as a result of over-topping by water in the pond, or by piping of fine materials under the action of seepage through the embankment or its foundation. A common problem has been the piping of tailings into decant and other culverts installed under tailings embankments.

Burning of coal waste piles is a hazard because of its generation of noxious gases. Other chemical changes, such as the oxidation of pyrite minerals, can cause acidic run-off from waste piles.

Factors Affecting Stability

The resistance to sliding along potential failure surfaces within the embankment and its foundation is a prime factor affecting the stability of an embankment. This resistance is governed by the shear strength of the materials, both cohesive and frictional, and the pore water pressures at the failure surface. The shear strength of the materials can be reduced by weathering and by softening by water; it can be increased by compaction and, sometimes, by chemical cementing of the waste materials. Water pressures will vary from point to point within the embankment and its foundation, depending on the source of seepage water and the relative permeability of the various materials in the embankment.

Cracking of embankments caused by differential settlements can reduce the shearing resistance along potential failure surfaces. Where such cracking occurs in a tailings embankment, excess seepage may develop leading, ultimately, to piping.

Basic Considerations

The design of mine tailings embankments should consider the economics of alternative sites, types of embankment and methods of waste disposal; these will be interdependent. The availability of construction materials, the quantities and characteristics of the wastes, climate, topography and the nature of the foundations at alternative sites will all be factors. The consequences of failure should be considered in establishing the factor of safety for which the embankment is to be designed. Deformations under earthquake shocks, and the possibilities of liquefaction of the tailings or embankment materials, should be considered for embankments located in seismically active areas.

Mineralogical Properties of Materials

Notes on the mineralogical properties of mine wastes are given in Section 3. In general, the strip mining of coal seams results in the production of large quantities of broken shales, siltstones and sandstones. These wastes are usually dumped in waste piles; the weathering of the shales and siltstones may reduce their shearing strength and cause stability problems. Fine wastes from the coal washing plant can be ponded for settlement, or filtered and deposited in a more-or-less dry condition.

In the mining of industrial minerals, the proportion of waste to ore is relatively low in comparison to that in metal mining. Severe dusting often occurs on asbestos waste piles. The escape of brines from potash tailings ponds can be a problem with these wastes.

Finely ground wastes that have a high content of siliceous materials generally have poor binding qualities and are very susceptible to erosion by wind and water. High sulphide tailings will often oxidize, cementing to form a surface crust; however, they may ignite if the sulphide content is very high and they often produce acidic run-off which inhibits plant growth. The treatment of gold ores with sodium cyanide can result in poisonous tailings effluents which must be treated before release from settling ponds. Arsenic may also be produced by the roasting of gold bearing ores. Tailings from uranium mining can contain dangerous radioactive materials.

Geotechnical Properties of Materials

Clays, where present in the foundation, can affect embankment settlement and stability. However, they may consolidate under the weight of fill placed over them and gain in shear strength as the embankment rises. Saturation and swelling of uncompacted clays in waste piles can reduce their shear strength substantially.

Silts can behave either as cohesive or as frictional materials, depending on the conditions of density, gradation and moisture content. Sands and gravels are relatively incompressible; their shear strength is primarily frictional and their behaviour can be predicted fairly reliably when their gradation and density are known. However, loose, fine, saturated sands are often susceptible to liquefaction.

The properties of glacial tills and other soil mixtures are dependent to a great extent on their densities and on the proportions and characteristics of the fines they contain. Their shear strengths may have both cohesive and frictional components. Dense tills are usually strong, relatively impermeable and incompressible. However, in foundations geological variations can be important.

Organic soils, such as peat, are generally very compressible and have low shear strength. Removal from embankment foundations might be necessary.

Rockfill composed of sound angular rock fragments will be strong and pervious, even without compaction. However, substantial settlements can occur in uncompacted rockfill. Weathering of sedimentary rocks, such as shales and mudstones, can seriously reduce the shear strength of rockfills containing a large proportion of these types of materials.

Most mine tailings materials, particularly from siliceous rock types, will behave like sands, except for the very fine "slimes" fractions which will behave like silts or clays.

Sound bedrock generally has more than adequate compressive and shear strength to support mine waste embankments. However, extensive seams of soft materials, such as fault gouge, can affect embankment stability.

General Site Investigations

Published climatic data should be consulted for the evaluation of evaporation and runoff at embankment sites. Estimating methods utilizing such published data are given in Appendices A and B. Their accuracy can be substantially improved by even a few years of evaporation, precipitation and stream flow measurements at the actual embankment sites. For major embankments where runoff could be a decisive factor in promoting failure, it would be advisable to install the gauges necessary to obtain such records.

Topographic maps are necessary for the planning and design of mine waste embankments. Aerial photographs are useful for interpreting the general geology of the site and in locating potential sources of construction materials.

Soils and Waste Material Investigations

A geological appraisal of the site should be made followed by a preliminary foundation investigation based on this appraisal. The investigation should be reviewed and modified as required when initial test hole data is obtained. High embankments, or those founded on clays or silts, may justify extensive foundation drilling, sampling and testing. For low embankments, or at sites where bedrock is obviously at shallow depth and the surface soils are competent, an investigation consisting of auger holes and test pits may suffice.

Soil samples should be classified and tested in the laboratory to determine their gradation, in-place density and moisture content. Where clays are present in the foundation, consolidation and shear strength tests may be necessary to determine their compressibility and shear strength characteristics. Field permeability tests may also be required to determine the approximate porosity of foundation materials.

Foundation water levels should be measured in all exploratory holes; they are particularly important where the foundation contains several strata of greatly different permeability characteristics. All exploratory drill holes should be accurately logged and their locations and elevations should be recorded on topographic maps and sections.

For the design of mine tailings embankments, the specific gravity and gradation of the tailings should be determined to assess their suitability in embankment construction and to guide the design of embankment drains and filters. Permeability, consolidation and strength testing also may be required on representative samples of the tailings.

Design

As a general guide, the procedure for the design of tailings embankments usually will involve:

making a field reconnaissance of the general site area,

making preliminary layouts and comparing the areal extent, height and economics of alternative ponds capable of storing the intended mine output of waste solids,

selecting the most promising site; making detailed site inspections; and establishing a programme of foundation and materials investigations (including waste materials),

estimating seepage, evaporation and runoff for the pond and assessing the range of pond water level variations, with both minimum and maximum water reclaim from the pond to the mill,

studying alternative embankment types, tailings disposal methods and water control measures for the selected site and selecting the most economical overall system, making seepage stability and, if necessary, settlement analyses and establishing the final embankment design,

Preparing construction drawings and specifications which should cover also, the waste disposal procedures to be followed.

Except for the evaluation of probable seepage, evaporation and pond level variations, design procedures for waste piles will be similar.

For small-volume embankments, a tailings dam constructed entirely of borrow or dry waste materials may provide the most economical system, allowing simplicity and economy in tailings disposal.

Larger embankments usually require at least a "starter dam" of borrow materials, to provide sufficient freeboard to prevent over-topping of the crest when the pond is initially put into service and to provide storage for water clarification and reclaim.

Tailings sands (produced by spigotting, cycloning or sluicing) that contain less than 10-15 percent of fines (-200 mesh) are usually suitable for embankment construction. However, unless they are compacted or kept unsaturated in the embankment, such sands may be susceptible to liquefaction. Embankments of tailings sands can be built by the "upstream" method, in which case the starter dyke will usually be located at the downstream toe - and should be pervious relative to the sands. The principal disadvantages of this method are the limited heights to which such embankments can be built because of possible shear failures through the underlying slimes and the susceptibility of the loose and saturated sands in the embankment to liquefaction.

Embankments built by the "downstream" method usually will have the starter dyke at the upstream toe - and this dyke should be relatively impervious. The method usually requires cyclones for the production of sand-fill material, although sluicing could also be used. The cyclones often require maintenance and frequent replacement. Also, substantial variations in sand gradation can be caused by fluctuations in operating conditions in the tailings pipelines to the cyclones. The principal disadvantage of the downstream method is the large volume of sand required for construction of the embankment cross-section. With finely ground tailings, it may be difficult to keep the embankment crest above the pond surface because of the low available volume of sand suitable for placement in the embankment. However, this difficulty can often be solved by constructing part of the section with borrow or dry waste materials. The embankment cross-section and construction schedule can then be arranged to provide adequate freeboard.

Methods of seepage, stability and settlement analysis for tailings embankments, and for waste piles, are described in Section 5. Also indicated are the extent of site and material investigations and of design analyses that could reasonably be required for waste embankments. Minimum factors of safety for various conditions, and limiting crest widths, are suggested.

Construction and Operation

The design of mine waste embankments is particularly dependent on the methods utilized for waste disposal, as these establish the condition and distribution of the materials in the embankments. For tailings embankments, the methods and locations of slurry disposal and water reclaim are major factors to be considered. By locating disposal points near the embankment crest and the reclaim water intake on the far side of the pond, the pond water can often be maintained at a location well back from the upstream face of the embankment. This will reduce seepage through the embankment, lower the level of the phreatic surface within the embankment and allow greater freedom in selection of the type of embankment. Embankments of tailings sands alone are more likely to be stable under these conditions than they are with the pond located close to the upstream face of the embankment.

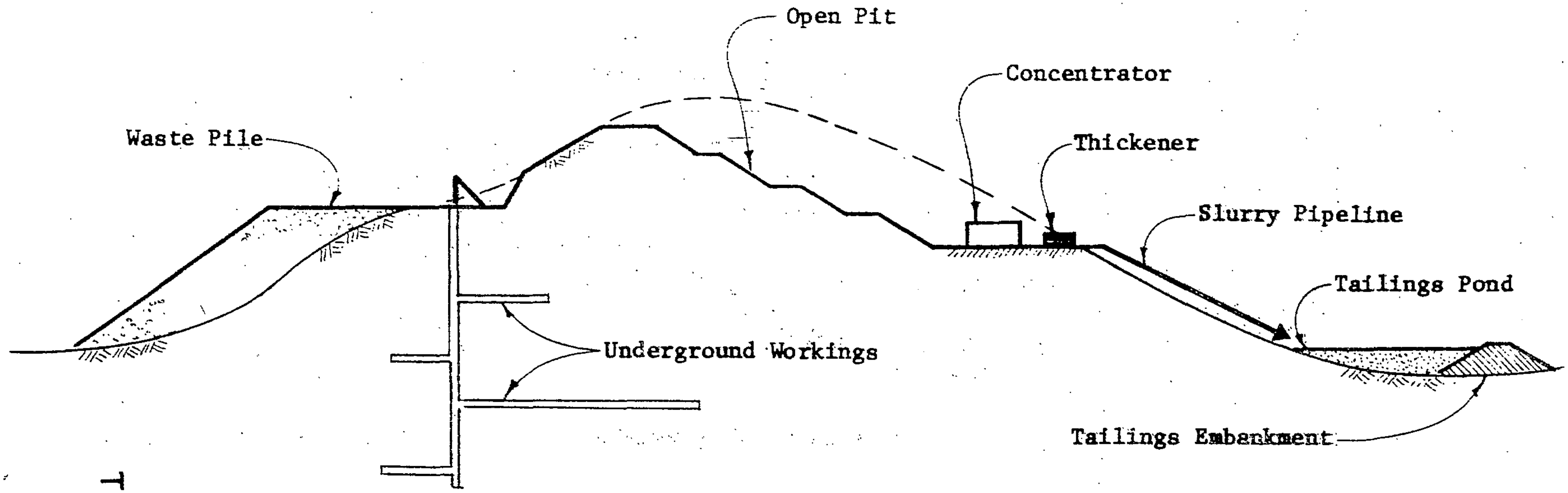
Barge-pump reclaim systems are likely to be more economical than decant or siphon systems when the pond is located distant from the embankment, because of the culvert lengths involved. Decant culverts should be conservatively designed, because of the danger of piping into collapsed sections and open joints and along the outside of the culvert - a common type of failure in the past. Siphons have several serious operating disadvantages.

Mine waste embankments are usually raised to full height over a period of many years and, during this time, many factors can develop to influence the stability of the embankments. High embankments should be instrumented to monitor movements in the embankment and its foundation, and to measure changed in piezometric levels and seepage flows. Suitable instruments are described in Section 6. Data obtained by these instruments, and construction and waste disposal procedures, should be recorded and periodically reviewed to ensure the safety of the embankment throughout its life.

Maintenance and Reclamation

The objectives of maintenance programmes for mine waste embankments are described in Section 7. These should include periodic inspections and reviews of recorded measurements and of waste materials and disposal methods. Careful attention should be given to ensuring effective drainage and effective seepage and runoff control. Methods of improving the stability of existing waste embankments such as slope flattening, the addition of berms and slope height reduction are described, as are measures for controlling piping and surface erosion and for controlling fires in coal waste embankments.

Descriptions of waste pile reclamation by landscaping, seeding with grass and reforestation are included also.



TYPICAL MINE WASTE DISPOSAL SYSTEM

FIGURE 1-1

SECTION 2

FUNCTIONS, PROBLEMS AND GENERAL CONSIDERATIONS

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FIGURES

SECTION 2

FUNCTIONS, PROBLEMS AND GENERAL CONSIDERATIONS

FUNCTIONS OF MINE WASTE EMBANKMENTS

Solids Storage

The prime purpose of waste embankments is the storage of solids. In the case of tailings embankments, a secondary requirement may be the temporary storage of liquids (usually water) for clarification or pollution prevention purposes. The approach to be taken to final disposal of the liquid included with slurried wastes may be influenced by requirements as to the final disposal of the solids. For example, fines from coal preparation plants are sometimes recovered for sale from settling ponds. In this case, there would be an advantage in promoting rapid drainage of water from the pond in order to reduce the water content of the fines prior to their recovery. In other cases, slurries may contain potentially dangerous liquids which may be so damaging to the environment that a basic requirement will be to reduce their escape from the pond to a minimum.

Sedimentation and Consolidation of Solids

Usually, needs for the reclaim of water from tailings ponds for re-use in the ore treatment plant, or for discharge of tailings effluent into adjacent streams, will require that a tailings embankment retain a certain minimum volume of liquid for sedimentation of the suspended solids. Non-colloidal tailings solids will settle rapidly from the slurry in this pool leaving a relatively clear liquid for reclaim or discharge.

Initially, particles settling from the liquid in the pond will accumulate in a very loose state, deposits near the surface having a high void ratio and a high water content. These loose deposits will consolidate with time as liquid is expelled from the voids between the particles, the void ratio at any point and the water content being influenced by: the effective vertical pressure of the overlying material, the permeability of the surrounding deposit, the distance that the liquid must travel to drain from the deposit, and the time during which vertical pressure has been applied.

The amount of liquid expelled from the deposits during the consolidation process will be much smaller than the amount of free liquid produced during the initial sedimentation stage. Sedimentation occurs rapidly; consolidation is a slow process with some tailings, particularly those that are finely ground or have a high clay content or low specific gravity. Even after consolidation under their own weight for many years, such deposits may have very high void ratios and water contents. This condition, which is a factor in the long-term stability of tailings deposits, will remain indefinitely, unless additional measures are taken to promote further drainage of liquid from the deposit.

PRINCIPAL REQUIREMENTS

Stability

There are few situations in which instability of mine waste embankments can be tolerated. Instability of waste embankments located in isolated areas may not involve risks to adjacent property or installations at the time they are formed but may become a threat, due to nearby development, at some time after their abandonment. Such unstable waste piles could endanger persons working on or near them during operation. An added danger in the case of the failure of tailings embankments would be the loss of liquid and unconsolidated solids from the pond, with consequent possibilities of flood waves and pollution in adjacent streams.

Runoff Control

There will always be some catchment area contributing runoff into the area of the mine waste embankment. This may vary from a minimum encompassing the area of the waste pile (or tailings pond) itself to a substantial area incorporating the drainage area of streams entering the valley across which a tailings embankment is constructed. Substantial runoff volumes and flows can result from heavy precipitation or snow melt over relatively small catchment areas.

The effects of runoff can include: surface erosion of waste piles, with downstream pollution; saturation of waste piles, with a danger of decreased resistance to sliding failures; and overtopping of tailings embankment, with the threat of a complete failure. The potentially serious consequences of a failure to effectively control runoff into the area of mine waste embankments make such control an important consideration in the design of these embankments.

TYPES OF PROBLEMS ENCOUNTERED

Instability

1. Rotational Slides

Earth slopes commonly fail in the form of a rotational slide, as illustrated by Figure 2-1. In this type of failure, the surface along which movement occurs closely approximates that of a horizontal cylinder or sphere. This is characteristic of failures of slopes composed of materials whose shear strength includes both cohesive and frictional components. Slopes of cohesionless materials may also exhibit this form of instability when the foundation materials under the slope are instrumental in bringing about failure.

The first sign of rotational slipping is usually a tension crack at the top of the failure surface, sometimes accompanied by slumping of the material on the slope side of the crack. Bulging at the toe of the slope, or heaving of soft foundation materials close to the toe, may also occur. The rate of development of rotational slides is unpredictable. In some instances the rate of movement may be sufficiently slow to allow promptly executed remedial works to arrest the movement. In others, large rotational movements occur so rapidly that insufficient time is available to complete remedial works.

2. Surface Slides

The sliding of shallow surface layers is a form of instability characteristic of slopes of dry cohesionless materials formed by dumping at the natural angle of repose of the material. It is a special case of rotational sliding where the radius of the failure surface is large and the failure surface almost planar.

3. Slides on Planes of Weakness

Planar failure surfaces, and other modifications of the cylindrical surfaces occuring with rotational sliding, can be caused by surfaces of weakness beneath the slope. Such surfaces can include: buried slopes which at one time were exposed to weathering; snow covered surfaces over which additional waste material has been dumped; layers of fine waste materials included in a pile of coarser waste; and foundation strata of low shear strength. With these conditions, failure surfaces may follow the surfaces of weakness producing non-circular or wedge-shaped slides as illustrated on Figure 2-2.

4. Creep

Movement of a mass of material forming a slope at a slow and steady rate down and parallel to the slope is known as creep, particularly where the material moves as a consequence of deformation rather than along a defined failure surface. The siting of a waste pile on a natural clay slope increases the forces tending to cause downhill movement of the clay and may initiate creep. However, changes in pore water pressures may be the more explicit cause of such deformation. The apparent creep of a slope may thus develop into a slide along a defined failure surface.

5. Mud Flow

A mud flow occurs in the form of a rapidly moving stream of water-borne soil having the consistency of mud. It is usually caused by the saturation of soil masses by heavy rainfall or springs and commonly occurs at the downstream toe of tailings embankments and waste piles constructed of relatively fine and impervious material. Dumping over-the-side on steep slopes can produce particularly critical conditions.

6. Progressive Sliding

Where a slope is interrupted by a minor slide, as may occur as a result of the emergence of seepage on the slope, subsequent sliding of progressively increasing size may occur repetitively above the first slide, the material from each slide being removed by the seepage water at the toe of the slide area. Similar slides may occur where the material from a minor slide is excavated without restoration of the original geometry of the slope. The form taken by progressive sliding is illustrated on Figure 2-3.

7. Flow Slides

Some soils, such as very fine-grained uniform sand and cohesionless silt or rock flour adopt a loose unstable structure when deposited through water. The permeability of such soils may be relatively low. If such a soil is disturbed, its structure may collapse, the soil particles attempting to find a new structure by which they occupy a smaller volume. If, at the same time, such a soil is saturated it can only decrease its volume through drainage of water from the voids between the soil particles. If the soil has a low permeability, this drainage may occur too slowly to prevent a transfer of weight from the soil skeleton to the pore water. This results in the soil particles becoming temporarily suspended in the water and the soil mass suddenly acquires the property of a viscous liquid. This transformation is known as liquefaction.

Large volumes of liquefied soil can flow through relatively narrow openings and can travel for considerable distances before stopping. Loose soils deposited at void ratios above a certain critical value are most susceptible. Their liquefaction can be caused by vibration, resulting from earthquake shocks or nearby blasting, by large strains resulting from additional loading by superimposed materials, or by a rapid rise or fall of the water table within the soil mass. Mine tailings deposited in ponds often satisfy the known criteria identifying soils susceptible to liquefaction. A number of serious tailings embankment failures have been attributed to liquefaction of the tailings and/or the fine sand embankment used to retain them.

Runoff and Seepage

1. Overtopping

Overtopping of an earthen embankment by water usually results in serious damage to the embankment, and can result in a complete and disastrous failure of the entire embankment. Probably, it has been the most common single cause of dam failures.

The most important factors affecting the extent of damage to the embankment are:

the rate of flow over the crest,

the duration of flow over the crest,

the erodibility of the materials in the embankment and its foundation.

The rate of flow, and the length of the embankment crest, establish the depth and velocity of initial flow over the crest. With a relatively low flow and a long embankment, this initial depth and velocity may, theoretically, be low. However, unless the crest of the embankment is absolutely level and is composed of a material strongly resistant to erosion, the flow will concentrate and rapidly erode a gully in the crest. As the gully develops, the flow becomes more concentrated with subsequent rapid increases in flow velocities. With sustained high flows through such a breach in an embankment of erodible materials, a major failure can occur very rapidly.

Tailings embankments have an advantage over dams retaining water alone in that the volume of water in the pond may be relatively small; consequently the pond may empty before disastrous damage occurs to the embankment. On the other hand, tailing embankments are often constructed of highly erodible materials, namely the cohesionless sands of the tailings themselves. Breaching of the embankment may allow not only water to escape from the pond but, also, the unconsolidated deposits of solids accumulated in the pond which may liquefy and flow through the breach as a viscous fluid.

2. Surface Erosion

In areas of high runoff, particularly where the slopes are long and composed of erodible materials, erosion gullies may form on the slopes of waste embankments. A common location is the contact line between embankment and foundation, where runoff from adjacent hillsides will concentrate. If allowed to develop deeply, these gullies may promote slides of adjacent waste material on the slopes of the embankments. If sufficiently deep that they reach the water table within the waste embankment, they will promote the concentration of seepage flows into the gullies, with consequent increased erosion of material in the gully. The material eroded adds to pollution of the environment downhill of the waste embankment.

3. Sub-Surface Erosion

A common form of sub-surface erosion is the "piping" which sometimes occurs on the downstream slopes of earth dams and tailings embankments. It is caused by seepage through the embankment or its foundation, the seepage flows usually concentrating in layers or lenses of materials more pervious than the average. If the exit velocity of the seepage water from the soil is sufficiently high, this water will move grains of soil from the embankment or foundation surface. With continued movement of the soil, a hole or gully will develop at the seepage exit point. In cohesive materials, or even in wet cohesionless materials, the material may arch over the developing hole and progressive erosion will develop a "pipe" extending back towards the source of the seepage. The development of this pipe decreases the length of the seepage path from source to pipe-head, with consequent acceleration of the rates of seepage and erosion. If the source of the seepage is a pond of water, this water will eventually break through into the pipe and unrestrained flow will occur from the pond through or under the embankment.

This type of failure can occur very rapidly if the development of the piping is not noticed. It does not necessarily occur at exposed ground or embankment surfaces. It often occurs beneath the ground surface by erosion of fine materials into the voids of coarser materials. (For example, by the movement of grains of tailings sand into the open voids of a rockfill embankment). It is promoted by any situation which tends to concentrate seepage along a confined path, such as along the smooth surfaces of culvert or decant pipes passing through the embankment. Numerous failures of this type have occurred with tailings embankments by erosion of tailings solids into, through and around decant lines installed under the embankment.

The development of piping is illustrated on Figure 2-3.

Burning of Coal Wastes

Coal waste piles may be ignited accidentally by external agencies, such as the tipping of hot ashes or by lighting open fires on the piles. The most common cause of burning, however, is spontaneous combustion of carbonaceous materials, often aggravated by the presence of pyrite.

Burning may cause the formation of voids within a waste pile, which may result in local collapsing and adverse change in the stress distribution within the pile. Not all the results of burning have an adverse effect on stability however; burning may cause an increase in the shear strength of the waste and, where temperatures are sufficiently high, fusing may occur.

The chief hazards associated with the burning of coal wastes are due to the generation of noxious gases. These include carbon monoxide, carbon dioxide, sulphur dioxide and occasionally hydrogen sulphide; each can be dangerous if breathed at certain concentrations which may be found at waste pile fires. The rate of evolution of the gases may be accelerated by disturbing a burning pile, for example by excavating spoil or by re-shaping.

Carbon monoxide is the most dangerous and the most insidious of the noxious gases, as it may be present in potentially lethal concentrations and cannot be detected by smell, taste, or irritation. The sulphur gases are not likely to be present in high concentrations and they are readily detectable by smell, taste, and irritation long before the lethal levels are reached therefore the sulphur gases, although adding to the overall toxicity, are mainly a nuisance rather than a threat to life.

Combustion in a waste pile may lead to the formation of cavities below the surface, leaving a crust on the surface of the fire area which appears to be solid but which would not support the weight of a person or machine. Similarly, the ashes over a supposedly burned out portion of the heap may appear harmless but often cover red hot embers to a considerable depth.

Steam when in contact with red hot carbon forms water-gas, a mixture of carbon monoxide and hydrogen. A mixture of air and water-gas in which the proportions of the latter are between 5 and 72 per cent is explosive and these conditions could occur on a burning waste pile. Incidents have occurred where persons have been burned by hot material scattered by an explosion on or in a waste pile.

When working on a burning waste pile every effort should be made to prevent the formation of a cloud of coal dust in the vicinity of the fire, as a mixture of coal dust and air, if ignited, may explode with great violence.

FACTORS AFFECTING STABILITY

Foundations

Instability of mine waste embankments is not necessarily confined to sliding on surfaces located wholly within the waste embankment itself. In many cases, the foundation materials beneath the embankments are substantially stronger than the wastes deposited on them. However, even in foundations predominantly of sound rock, or of well consolidated soils, strata of relatively weak materials may occur. Such strata could include thin seams of soft fault gouge material in strata of otherwise sound rock, or strata of soft clay occurring between deposits of relatively firm sands and gravels. The low shear strength, and in some cases the low permeability, of such strata can result in sliding along surfaces located partially within the foundations or abutments of mine waste embankments.

The principal foundation characteristics affecting the stability of waste embankments are:

shear strength,

compressibility,

permeability.

The shear strength determines directly the resistance to sliding along potential failure surfaces passing through the foundation. Compression of the foundation can cause appreciable settlement of the overlying materials, sometimes causing cracks in tailings embankments which lead to excessive seepage and even failure by piping. The permeability of the foundation affects seepage through it, sometimes leading to reductions in stability from an increase in water pressures beneath or in the embankment, or from saturation of the waste material with a resulting increase in its weight and reduction in its shear strength. Foundation permeability is particularly important for tailings embankments during the early years of pond operation, when excessive seepage under the embankment may lead directly to piping failures. Examples of some foundation conditions affecting stability are illustrated on Figures 2-4 and 2-5.

Loads

The total driving force tending to create instability is supplied by gravity acting on the mass of the waste materials; it is proportional to the specific gravity and volume of solids and liquids in the waste materials. Superimposed plant or equipment live loads are not generally significant in relation to the weight of these materials.

Slopes

The steeper the slope of a waste embankment, the higher are the shearing stresses that develop on any potential failure surface. Consequently, higher shearing strengths must be mobilized along these surfaces to resist sliding.

Characteristics of Materials

The principal material characteristics affecting stability are:

grain size and distribution (gradation),

density,

permeability,

shear strength,

moisture content,

plasticity.

Materials vary widely in grain size and gradation, ranging from fine grained, impervious, compressible clays to coarse grained, permeable, incompressible gravels and rock fills. The grain size, shape and distribution all affect a material's resistance to shearing, as does its in-place density A relatively small increase in density can substantially increase a soil's shear strength and reduce its permeability.

The permeability of soil or waste materials, and the relative permeabilities of different materials in a deposit, affect seepage flows and water levels in that deposit. These in turn can affect the shearing resistance of the deposit. Of particular importance in determining the overall permeability of soil or waste deposits are the voids and stratifications which may exist in the deposit. These can cause substantial directional variations in permeability. The influences of open fissures and stratification often make it impossible to accurately predetermine seepage flows and water tables. Under these conditions, estimates can be made only of the probable range of seepage flows and of the approximate locations of water tables.

Usually, the shear strength of a soil will have two components, a cohesive and a frictional component. The cohesive component of a soil's strength is not affected by the pressure of the pore water in the voids of the soil. However, softening by absorption of water can reduce cohesion values. The frictional component of a soil's shear strength, however, is a function of the effective pressure between the grains.

Granular soils (those containing only small percentages of clay or silt sized particles) generally have very little cohesion, their shear strength consisting almost entirely of the frictional component.

Some soils exhibit a reduction in shear strength with increasing strain. Such soils may produce sudden slides if stressed to a point beyond the "peak" shear strength. Beyond this point, the strength available to resist sliding suddenly drops to a lower "residual" shear strength, while the activating forces remain constant.

The consolidation of soft, saturated, impervious soils, such as clays and silts, can affect embankment stability. As these soils are compressed under the applied loads, water is squeezed from the voids. During this process (consolidation), the water in the soil voids is under pressure. The increase in shear stress with little immediate increase in shear resistance, can affect the stability of embankments of fine waste materials, particularly if the rate of waste placement is faster than the rate of pore pressure dissipation.

The water content of an undisturbed, natural, soil when compared to the Atterberg limits of that soil, provides a key indicator to many of the soil properties. Soils whose natural water contents are close to the liquid limit are usually highly compressible. Soils whose natural water contents are higher than the liquid limit are usually sensitive to remoulding and may liquefy, under relatively small strains, when loaded.

The placement water content of soils used for embankment construction may greatly affect the characteristics of the embankment (density, shear strength, compressibility, permeability). For every soil there exists an optimum water content at which maximum densities can be achieved using a minimum compactive effort.

Changes in Material Characteristics

1. Weathering

Physical and chemical changes in waste materials following their deposition on embankments can affect the stability of these embankments. A common physical change is the breakdown of shales and siltstones wasted during coal mining. This can alter the gradation of these materials from basically coarse broken rock to soils, with consequent changes in shear strength and permeability. The extent of breakdown depends on the parent rock, and the effects of air, water, frost and handling between mining and placing on the pile. Particles of coals and sandstones which have strongly cemented bonds do not change appreciably on exposure, whereas weakly cemented coals and sandstones break down readily. Shales and silstones break down to an extent related to the grain size of the parent rock. The finer the grain, the quicker the degradation, although a fine-grained rock from one region or horizon may be more resistant than a coarser-grained rock from another. Some wastes break down to an extremely flaky material.

Degradation of the shales and siltstones can start within a few weeks of placement on the surface of the pile, the first effect being the degradation of the larger particles into fine gravel and sand-sized particles. At depths below a few inches degradation occurs more slowly or not at all. A reduction in particle size will tend to cause a decrease in permeability. Consequently some wastes will in time acquire a relatively impervious skin. This effect will not be so pronounced for the more stable wastes, as the permeability change for such material is not likely to be large.

Shales in which the constituents contain a large proportion of clay minerals can degenerate into plastic soils of very low shear strength.

2. Softening

Some soils and waste materials will soften or swell when in contact with water, often resulting in a radical reduction in shear strength. These materials can include certain types of clays and clay shales. If a deposit of such a clay is extensively fissured, water penetrating into the fissures can seriously weaken the whole deposit, in that its shear strength will be governed by the strength of the softened material adjacent to the fissures. Remoulding of a clay can produce the same effect.

Materials at the surface or toe of a waste pile, particularly where there is ponding, are most likely to exhibit softening or swelling. This may cause local reduction of stability resulting in local sliding, and hence sudden changes in stress within the adjacent intact pile or its foundation. Materials which have softened or swollen may constitute surfaces of weakness within any waste pile built over them. Chemical solutions seeping through a pile or its foundations may react with the waste and bring about changes in its shear strength and other properties, with resultant adverse effect on stability.

3. Chemical Changes

Pyrite, and rocks containing pyrite, are often found in wastes from iron and coal mines. Such materials oxidize continuously while lying on the surface of a waste pile. The acidic products of pyrite oxidation can cause chemical changes in other waste materials, affecting their shear strength. Water draining from the pile may be contaminated with acidic or neutral salts.

Changes in Water Level

1. Within the Embankment

Changes in the level of the water table in a waste embankment will change the pore pressures and consequently the resistance of the pile to sliding. Increases in level can be caused by surface water seeping into a waste pile, springs located under the pile and not effectively drained, seepage water from settlement ponds constructed on the pile, blockage of drainage culverts beneath or around the waste pile and changes in the characteristics of the waste materials placed in the pile.

In tailings embankments, increases in the level of the water table can be caused by blocking of drainage and filter layers within or below the embankment, freezing of surface layers of material on the downstream slope of the embankment and changes in methods being used to construct the embankment.

Alteration of the permeability of foundation materials below waste embankments caused by strains induced by mining subsidence can also affect the level of the water table.

2. Pond Drawdown

Seepage from a pond through its retaining embankment will buildup pore pressures within the embankment, these pressures being highest near the upstream face and reaching constant values if the water level in the pond does not vary. The more impermeable the embankment materials, the longer is the time required for the seepage pattern and pore pressures to react to changes in pond level and reach a new steady state.

If the pond water level drops, there will be, until the new steady state seepage pattern is established, continuing pore pressures in the embankment fill. The additional shearing stresses caused by the drawdown in the upstream slopes may cause failure. In general, the magnitude of the additional forces introduced by pond drawdown will vary directly with the rate of drawdown and inversely with the permeability of the embankment materials.

Disturbance of Waste Embankments

1. General

Disturbances of waste embankments will often reduce their resistance to sliding failures. Such disturbances may be caused by: vibrations from earthquakes, blasting, pile driving or machines in the vicinity of the pile; mining subsidence or other foundation settlements; impact loading from dumping or from material slipping from one part of the pile to another; or by localized sliding causing sudden changes in stress within the pile or its foundation. Particularly where waste materials are saturated, sudden disturbances which cause rapidly applied shearing strain in the pile or its foundation may cause increases in pore pressures, resulting in a reduction in stability. Where disturbances produce large shear strains, the mobilized shearing resistance of the material may pass through its peak value to a lower residual value along potential failure surfaces. If disturbances induce tensile strains within an embankment or its foundation, open cracks may develop along which no shearing resistance can be mobilized.

2. Differential Settlement

The shape of the foundation, or the nature of the foundation materials, may cause differential settlements to occur in an embankment which can affect its stability. Particularly in relatively brittle fills, such differential settlements can cause extensive open cracks in the embankment and its foundation. In the presence of seepage, full hydrostatic pressures can develop in such cracks.

Examples of foundation conditions causing differential settlement are shown on Figure 2-6.

GENERAL CONSIDERATIONS

Economics

The design of waste embankments cannot be separated from considerations of the costs of alternative methods of disposing of the waste.

Construction procedures long regarded as standard in producing stable highway, dam and other embankments may represent a substantial item of cost when applied to mine waste embankments. However, the increasing size of waste embankments, and the increasing number and seriousness of failures, makes it important that stabilization procedures, such as compaction and seepage control, be used to the extent to which they are necessary. Traditional waste embankment construction procedures should also be critically reviewed. Procedures such as the raising of the downstream slopes of tailings embankments with flashboards may have produced adequate embankments with relatively small quantities of coarse grained tailings but may be entirely inadequate with large quantities of finely ground materials. Spigotting, though a simple procedure with large tailings quantities, may be inadequate in producing a stable embankment when the materials are finely ground.

Waste Quantities

Together with the topography and geology of sites available for the disposal of waste materials, the overall quantity of waste will establish the extent and height of waste embankments. A lower but more extensive waste pile may ensure a greater degree of stability at some sites but may be less economical than one of greater height and more limited extent. The required rate of disposal may affect the method of disposal, also, and consequently the design of the embankment.

Sources of Materials

A fundamental consideration in the design of any earth embankment is that of the sources of material from which the embankment can be built. Because of the relatively large quantities of fill involved, it is desirable to locate borrow pits close to the embankment. The cost of hauling borrow materials more than one or two miles is usually prohibitive. In the case of embankments required to retain mine wastes, the low costs of waste materials available for use as fill will often dictate that these materials be used to the maximum possible extent for embankment construction and that more costly borrow materials be kept to a minimum.

Waste Material Characteristics

Apart from the effects of their physical and chemical characteristics on stability, the handling characteristics of the materials are important in the design of mine waste embankments. Some fine wastes may be so wet when dumped that they cannot be mechanically compacted and must be retained by coarser wastes or by compacted embankments. Even after two stage cycloning, the sand from some fine tailings may have such a high content of fine material that it must be allowed to dry before it can be spread and compacted in the embankment.

Tailings Disposal and Reclaim Procedures

Particularly in the design of tailings embankments where substantial cost savings can be made by the use of tailings sands as fill material, the manner of deposition of tailings in the pond, and the type of water reclaim facilities used, can substantially affect the design of the embankment.

In embankments composed predominantly of tailings sands, the location of the water table within the embankment may well be critical in determining its stability. A high water table may cause sliding or piping failures; with low water tables, the embankment may be amply stable.

The principal factors governing the location of the water table will be the elevation and location of the pond water surface, in relation to the downstream face of the embankment, and the distribution of permeability of the materials between the pond and this downstream face. With disposal of the tailings along the upstream edge of the crest of the embankment, the tailings deposits in the pond will slope downwards into the pond away from this crest and there will be a decrease in material size and permeability with increasing distance from the crest. If the free water in the pond can then be controlled so that it is always located a considerable distance upstream of the crest of the embankment (over the more impermeable tailings fines), the seepage from the pond to the downstream face of the embankment may be relatively small and the water table low. However, if because of high runoff, or other reasons, the free water in the pond approaches the crest of the embankment, seepage through the coarser and more permeable tailings near the downstream slope of the embankment will cause the water table to rise, reducing the stability of this slope. This effect is illustrated on Figure 2-7.

When the tailings are deposited into the pond at a considerable distance from the embankment, the fine fractions will tend to settle against the upstream face of the embankment. This will also tend to be the lowest point in the pond, where free water will accumulate. Although this may shorten the required length of water decant or reclaim pump lines, the result will almost certainly be a high water table under the downstream slope of the dam, unless special materials are used in the embankment to lower it.

Therefore, disposal and pond level control procedures which will ensure that the free water in the pond is always kept a considerable distance from the downstream slope of the dam may mean that a homogeneous embankment of tailings sand will be amply stable. In contrast, procedures which allow the free water to approach the downstream face may require, essentially, the construction of a dam capable of retaining water alone. For adequate stability, this may require the use of relatively expensive borrow materials in the embankment. Such an arrangement is shown on Figure 2-8.

Climate and Hydrology

1. Affecting Tailings Embankments

The extent to which the climate of the area in which tailings embankments are located will affect the design of these embankments will depend principally on the effects of precipitation, evaporation and freezing on pond levels and on the materials in the tailings embankment. For tailings ponds which are required to retain all of the runoff from the pond catchment area, the combined effects of runoff, evaporation, seepage out of the pond, and disposal and reclaim from the pond will be cumulative; pond water levels may rise or fall, not only season by season, but also year by year depending on whether there is a net long-term gain or loss of water in the pond. Long-term rises in pond water level may threaten overtopping of the tailings embankment at some time during its construction, or after its abandonment, if provisions are not made to pass runoff around the embankment.

The importance of evaporation in determining whether there will be a net long-term gain or loss in pond water storage will depend on the climate and on the relative areas of the pond catchment and water surface. In arid areas, and where the pond water surface occupies a large proportion of the total pond catchment area, evaporation losses may more than balance the runoff into the pond. Water levels will then be principally a function of disposal into and reclaim from the pond. Where the pond catchment area is relatively large, runoff also will be a principal factor influencing pond water level variations.

Freezing can affect tailings embankment design in several ways. Spigotting or cycloning operations may be impracticable during the winter, thus preventing raising of the embankment crest during this season. Meanwhile, with continued disposal of tailings into the pond, the pond level will continue to rise. Particularly with embankments constructed by spigotting, the freeboard available at the end of the winter for storage of the spring, snow-melt runoff may be very small, involving a real danger of overtopping or piping failures. This seasonal variation in disposal procedures may also affect the distribution of tailings materials in the pond, winter dumping of tailings at points distant from the embankment sometimes causing the fine "slimes" fractions to settle near the face of the embankment. Subsequent raising of the embankment crest over these slimes may then lead to instability of the embankment.

Snow layers incorporated in the embankment, or the freezing of saturated materials on the downstream face, can also affect its stability. Freezing of the downstream face, which is aided by high pond water levels, can cause instability by blocking natural drainage, thereby raising the water table in the embankment. Freezing of the pond water surface can also cause difficulties with water reclaim, thus affecting pond levels.

2. Affecting Waste Piles

The principal climatic factor influencing the design of mine waste piles is the magnitude of precipitation. Runoff from rainfall and snow melt affects the degree to which waste materials may become saturated, or water pressures develop in the pile, and the degree of surface erosion which may occur. Freezing of fine wet wastes during winter placement may have some subsequent effect on stability when the frozen materials thaw; snow layers buried in the waste pile can cause surfaces of weakness along which sliding may occur.

Topography

A major influence of topography on the design of a waste embankment is its effect on establishing the height and extent of the embankment and the method of waste disposal. In some cases a high escarpment will allow dumping of wastes over a high face to form a pile of relatively limited extent. With coarse relatively pervious wastes, the pile may be stable at the angle of repose adopted by the dumped material. With fine wastes which can become saturated by seepage or runoff water, or with those whose material characteristics can change by weathering or softening, such a pile may later become unstable. On slopes approximately equal to the natural angle of repose of the waste material, the pile may first accumulate at the top of the slope and then suddenly fail, sending a substantial volume of material sliding down the slope. In such cases, it may be necessary to construct a retaining embankment at the toe of the slope, or to build up the waste pile from the bottom rather than by dumping from the top.

With tailings embankments, an important influence on the design may be the relationship between the volume of fill required in the retaining embankment and the volume available in the pond for storage of the tailings. A long embankment requiring a large volume of fill will usually mean that the embankment crest cannot be kept much above the level of the rising pond and there will be a tendency for free water always to be close to the downstream face of the embankment. A short embankment having a low volume relative to the pond capacity may allow the embankment crest to be kept always well above the pond surface, thus keeping the free water well back from the downstream face, reducing seepage, improving stability and reducing the possibility of overtopping.

Geology

Geological features of principal interest in the design of waste embankments are those affecting the shear strength, compressibility and permeability of the embankment and its foundations. These include soft strata or seams in the foundation and sources of seepage or hydrostatic pressure.

In most instances, the properties of the surficial soil deposits control the design, as even the softest rocks are normally more than adequate to support the embankment. However, in some cases, weak or highly pervious zones within the bedrock may have a major effect on the embankment design.

Earthquake

Oscillations of the foundations during earthquakes can cause substantial distortions in earth embankments, sometimes amounting to failure. The oscillations can occur in any direction and can last for several minutes. The characteristics of the oscillations, the duration of the earthquake, and the geometry and properties of the embankment and foundation materials are the major factors affecting the deformations which occur.

Movements of embankment materials along definite failure surfaces may occur, under certain conditions, during earthquake shocks. A succession of slides of limited displacement which could occur in an embankment of cohesive materials is indicated on Figure 2-9. The type of sliding to be expected in dry cohesionless materials is indicated on the same figure. These types of movement ignore the possibility of liquefaction of the embankment or foundation materials, which would cause lateral movement or spreading of the whole embankment; and of vertical cracking, which would be caused by differential shearing of the embankment.

It can be seen that the type of deformations to be expected during earthquake shocks will lower the embankment crest. With tailings embankments, such a lowering could cause overtopping by the water in the pond. Overtopping could also be caused by waves generated in the pond by slumping of the upstream face of the embankment, or by slides of unstable natural deposits (or of tailings) on the perimeter of the pond. Faulting or sudden settlement may also cause wave action. Transverse cracking of the embankment could lead to piping failures.

In considering the design of embankments to resist earthquake motions, two basic possibilities have to be considered:

liquefaction of the embankment or foundation materials,

embankment slumping not associated with liquefaction.

Slumping and sliding can be analyzed with a reasonable degree of certainty; liquefaction cannot. The best that can be accomplished is to identify a particular material as being susceptible to liquefaction. Whether this is sufficient to justify preventative measures, such as compaction or drainage of tailings sands, should involve an assessment of earthquake probabilities for the area as well as consideration of the consequences that a failure induced by such liquefaction would have. In assessing earthquake possibilities at a site, the distances to known epicentres should be considered. This may require a tectonic analysis of the project area to define the nearest active faults.

Consequences of Failure

Many factors involved in the design of embankments cannot be assessed with certainty. Such factors can include various characteristics of the materials involved, the magnitude of runoff flows into a tailings pond during its life and the characteristics of earthquakes which could affect the embankment. Ultimately the degree of risk attached to the adoption of a particular design must be assessed. This involves consideration not only of the degree of confidence which can be placed in the design parameters used but also of the possible consequences of one or more types of failure to which the embankment could be prone. For embankments threatening loss of life, serious property damage or environmental pollution, the design should be conservative. Where the consequences of failure would be slight, a greater risk of failure could be acceptable.

The potential seriousness of a failure is related to many factors other than the size of an embankment. A low embankment located above and close to inhabited buildings could pose a greater danger than a high embankment in a remote location. However, generally, there is a relationship to the amount and rate of energy that would be released by a failure. The amount of energy released can be related to the size of the embankment and its elevation above its surroundings. Often, the potentially most dangerous types of failures involve soils that undergo a sudden release of energy without warning: examples are soils subject to liquefaction and soils having a low ratio of residual to peak shear strength. Such soils can occur in waste embankments and their foundations.

When assessing the possible consequences of a tailings embankment failure, the designer should consider that the embankment will always exist and may still retain semi-fluid tailings hundred of years into the future.

INVESTIGATION AND DESIGN PROGRAMMES

Extent

The extent to which any programme of investigations and design analyses of mine waste embankments should be taken will depend primarily on the consequences of failure, the size of the embankment and the general site conditions. The complexity of the conditions will govern the extent of investigations but these investigations should be sufficient to permit definition and assessment of all adverse conditions. In-so-far that uncertainties in design parameters will usually be more critical for higher embankments, and the consequences of failure more serious, higher embankments generally will require a more extensive programme than will lower embankments.

Generally, investigation programmes should be taken to the point where it is known, with reasonable certainty, that the parameters adopted in the design analyses are conservative. The embankment designer is in the best position to assess this point and, therefore, should exert the strongest influence in determining the actual extent of the programme. However, for most high embankments (those in the order of 100 feet or more in height), it could reasonably be expected that the programme would include the following:

Site and Material Investigations

1. Engineering Site Inspections

A detailed inspection of the ground surface at the site should be made, concentrating upon: the delineation of old underground mine workings; the relative location of any streams, dwellings or other nearby installations; the waste and foundation material characteristics relevant to stability of the embankment; available sources of construction materials; the requirements for further foundation, borrow material and waste material investigations; and the types of embankments suitable to the general site conditions and mining operation.

2. Topographic Mapping

Topographic maps should be made of the embankment site, potential sources of borrow materials and, for tailings embankments, the pond site.

3. Geological Site Inspections

If it is indicated to be necessary by the engineering site inspections, geological inspections of the site should be made concentrating upon: the location of rock outcrops in relation to the embankment; the probable location of bedrock in the embankment foundation and abutments; the dip and strike of stratified rock exposures; the nature of exposed bedrock relevant to the possibility of weak or pervious seams in the foundation; and the requirements for further investigations of the foundation bedrock, if any.

4. Foundation and Construction Soils Investigations

Embankment foundations and potential sources of borrow materials should be investigated by test pits and/or drill holes, and by laboratory testing, to determine their characteristics in relation to embankment stability, settlement and seepage. In some cases, the determination of bedrock location by geophysical methods may be advantageous.

5. Waste Material Investigations

Proposed waste materials should be sampled (producing sample batches in the laboratory for this purpose if necessary) and tested in the laboratory to assess those of their characteristics significant to design of the embankment. If practicable and important to the design of the embankment, field and/or laboratory tests should be made to determine the gradation, permeability and handling properties of waste materials under the disposal conditions proposed for the embankment. Such tests could include trial embankments, cycloning tests and sluicing tests.

Design Analyses

1. Economic Comparisons

Alternative preliminary waste embankment designs should be made and the capital and operating costs of waste disposal over the life of the mine estimated for the alternatives studied.

2. Seepage Analyses

Seepage analyses should be made of tailings embankments to determine the probable location of the water table and the measures required for seepage control.

3. Stability Analyses

Static stability analyses should be made. Earthquake deformations, and the possibility of liquefaction due to earthquake shocks, should be considered if the embankment is located in a seismically active area (Zones 1, 2 and 3 as defined by the National Building Code).

4. Settlement Analyses

If the foundation soils investigations indicate that there are strata of substantial compressibility in the embankment foundation, settlement analyses should be made to determine: the expected settlement of the embankment; the possible extent of embankment cracking due to this settlement; and the amount of settlement of any drainage or decant culverts to be installed under the embankment.

5. Hydrological Analyses

For tailings embankments, analyses should be made to determine the probable influences of evaporation and runoff on pond water levels. Initially these should be based on available records of evaporation, precipitation and stream flows in areas near the embankment site and on information on the proposed rates of disposal into, and reclaim and seepage from, the pond. The results of these initial analyses will usually indicate the necessity for further climatic and hydrological investigations, if any.

Construction Supervision

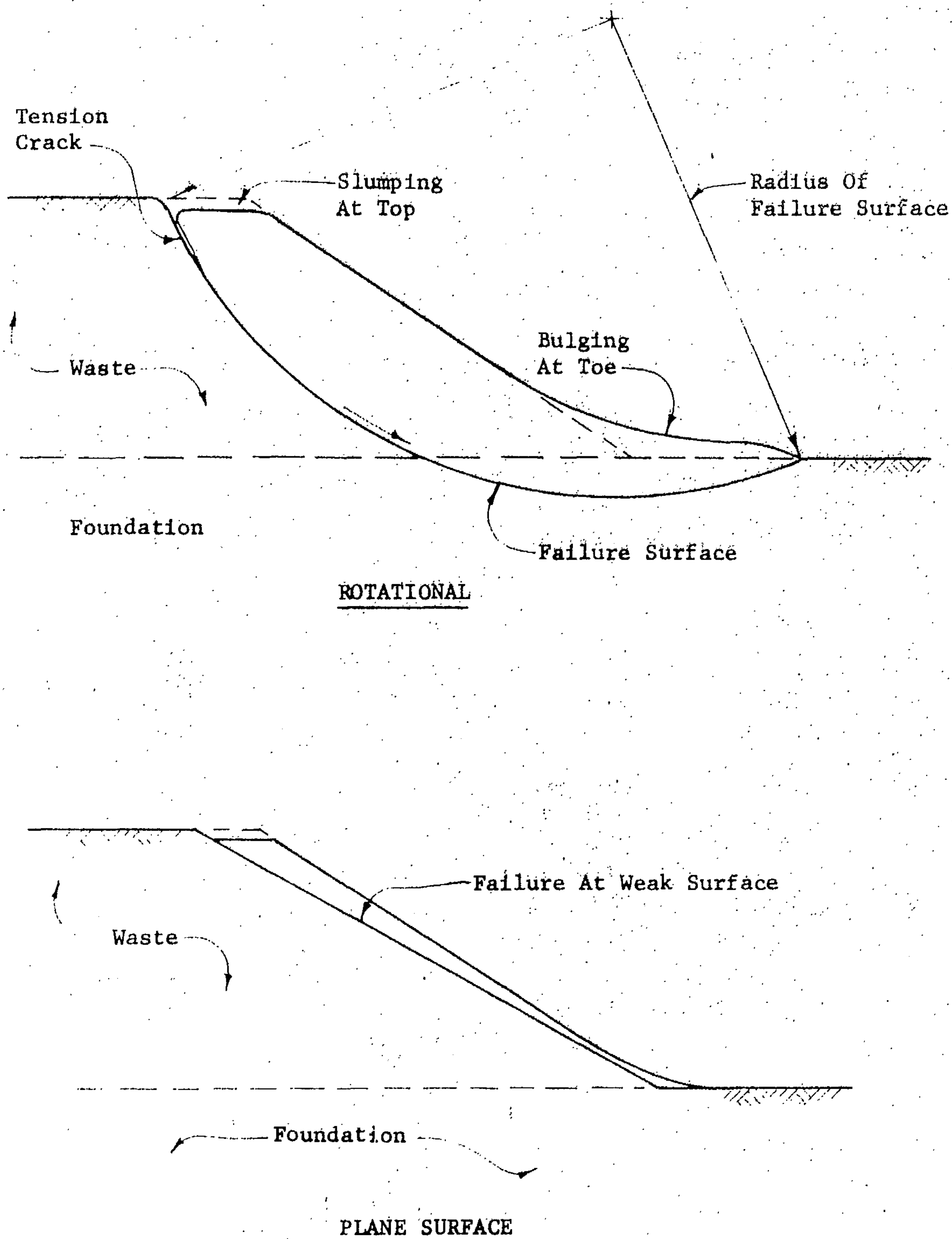
1. Inspections

For tailings embankments, foundation preparation and fill placement should be supervised continuously. For all high waste embankments, there should also be periodic site inspections by a compentent person to review the general status of the embankment.

2. Instrumentation

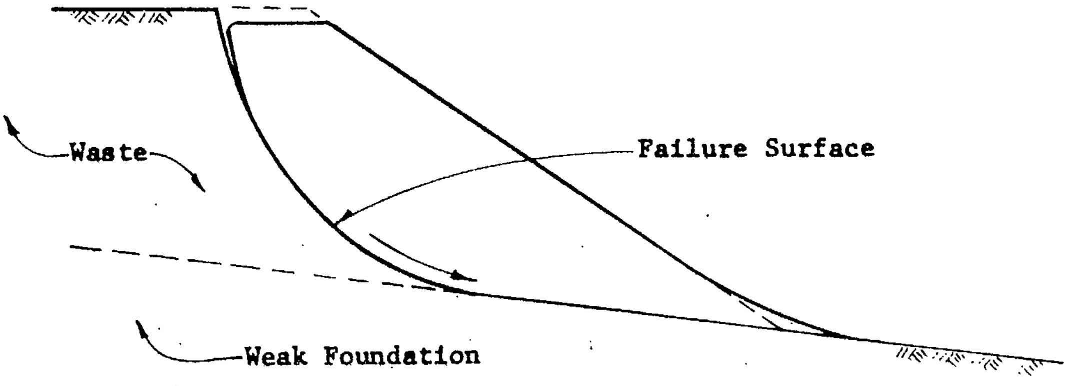
If the seepage and stability analyses indicate that this would be advisable, seepage levels and flows, and embankment movements, should be monitored by suitable instrumentation. Records should be kept of measurements made by the various instruments, and of any changes in construction and waste disposal procedures (and of waste material characteristics) which may change the distribution and properties of the materials in the embankment.

For lower embankments, particularly those in the order of 25 feet or less in height, some of these investigations will usually not be necessary. Where such low embankments do not consitute a serious danger, or site conditions are particularly favourable, initial engineering site inspections, stability analyses utilizing assumed material characteristics and periodic inspections during construction may be adequate to ensure a stable embankment.

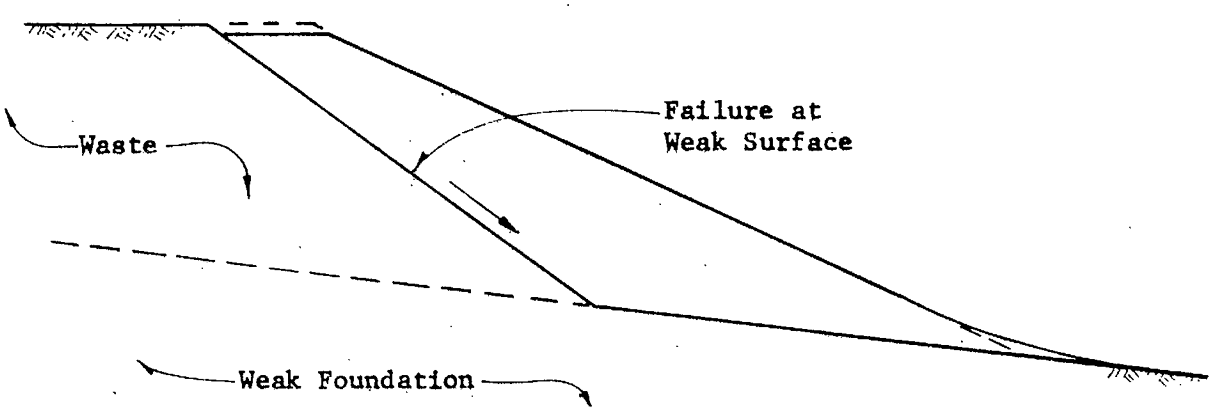


ROTATIONAL AND SURFACE SLIDES

FIGURE 2-1

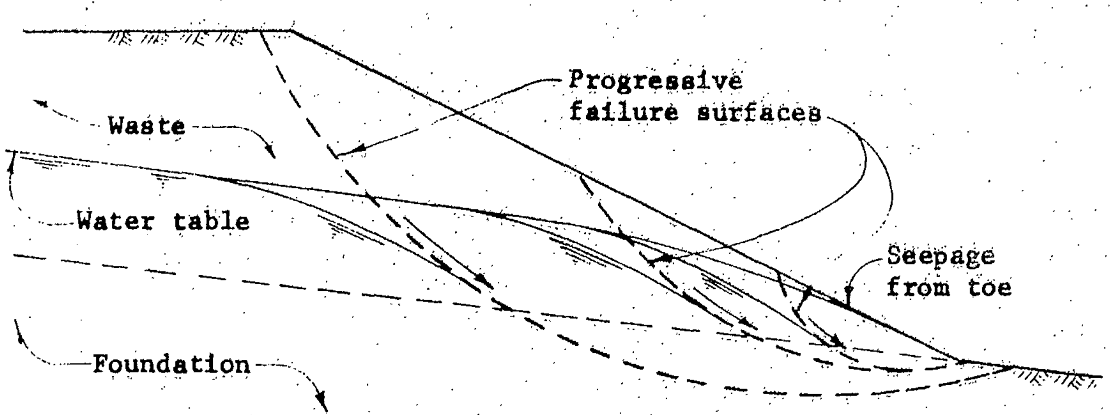


NON-CIRCULAR

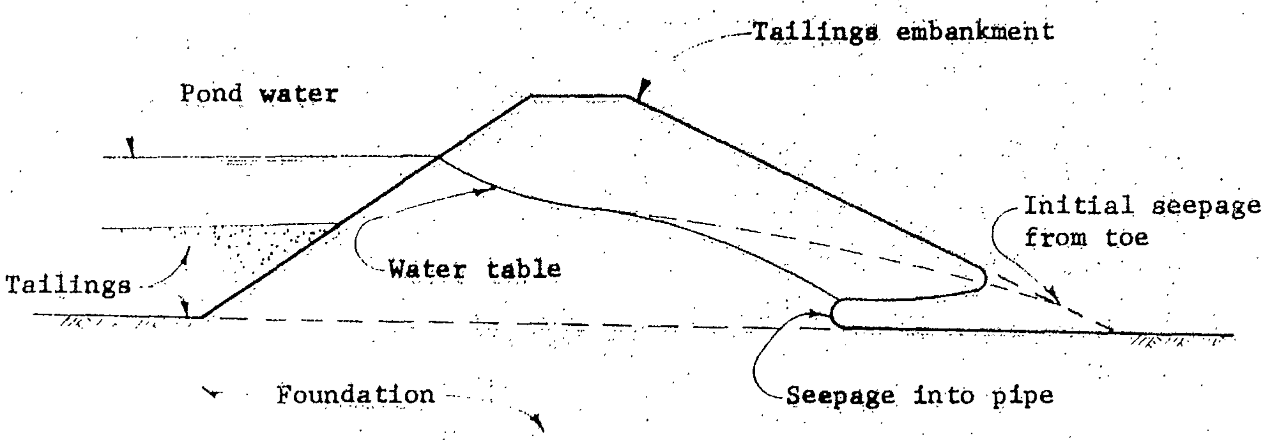


WEDGE

NON-CIRCULAR AND
WEDGE SLIDES
FIGURE 2-2

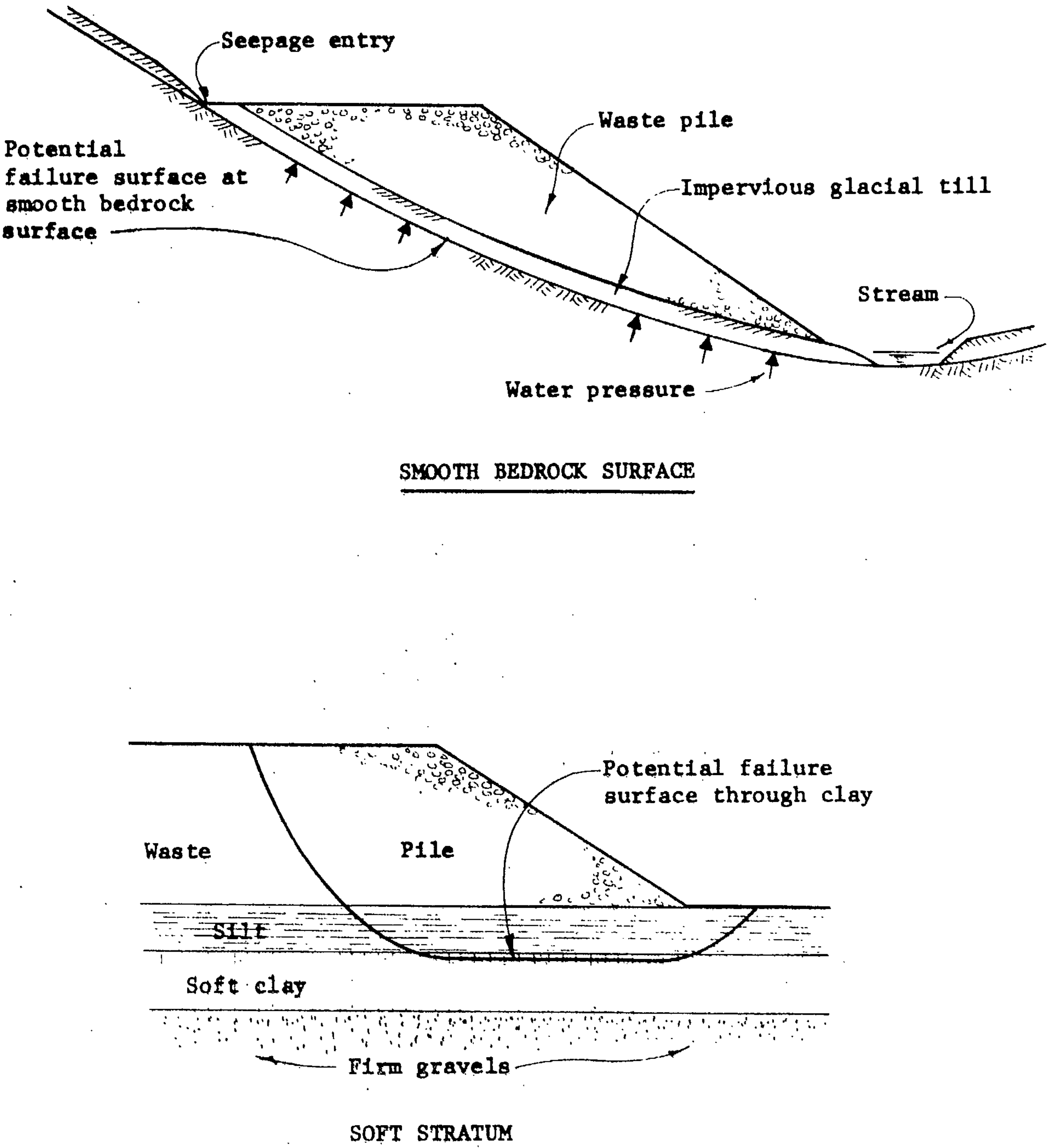


PROGRESSIVE



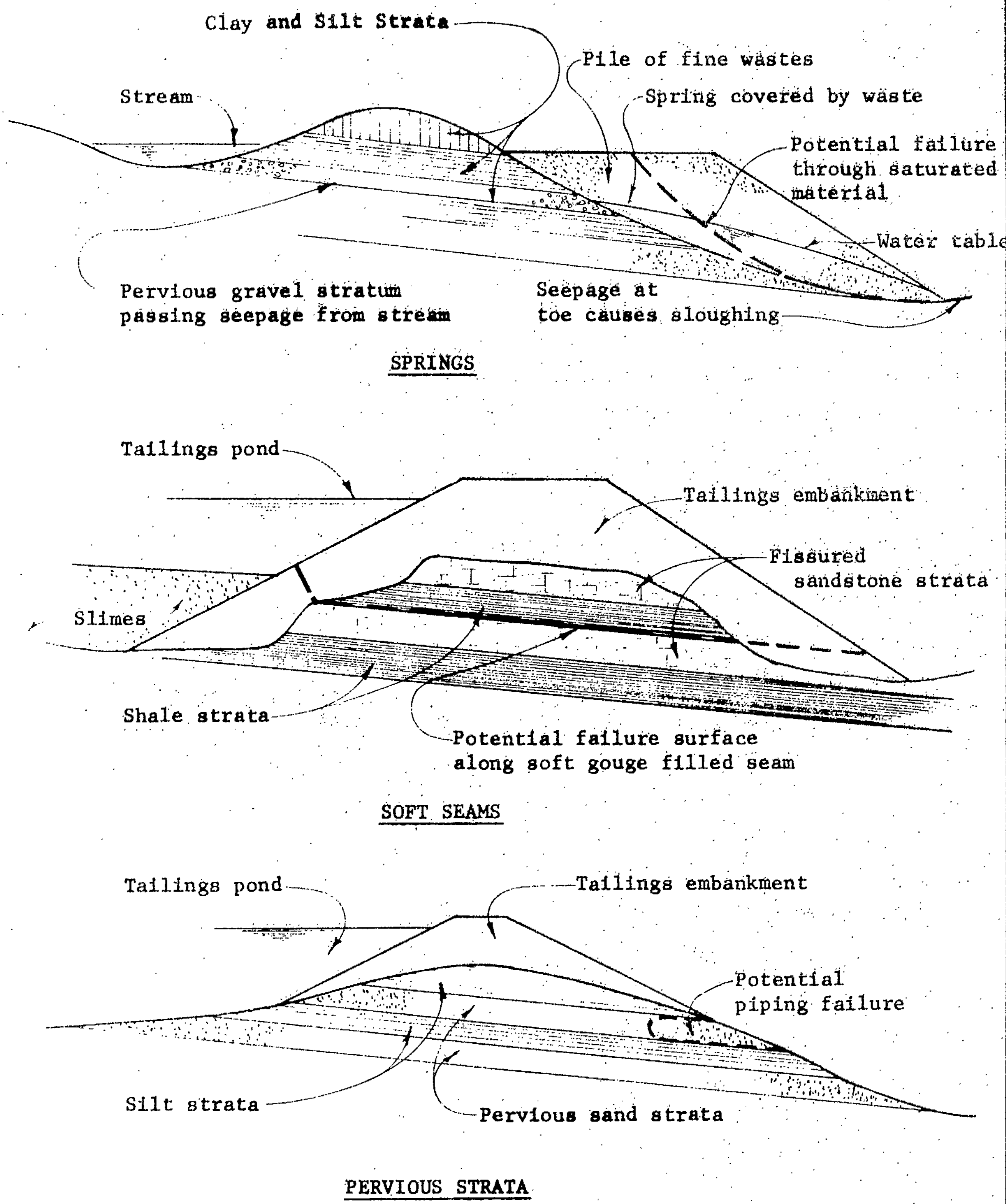
PIPING

PROGRESSIVE SLIDES
AND PIPING
FIGURE 2-3



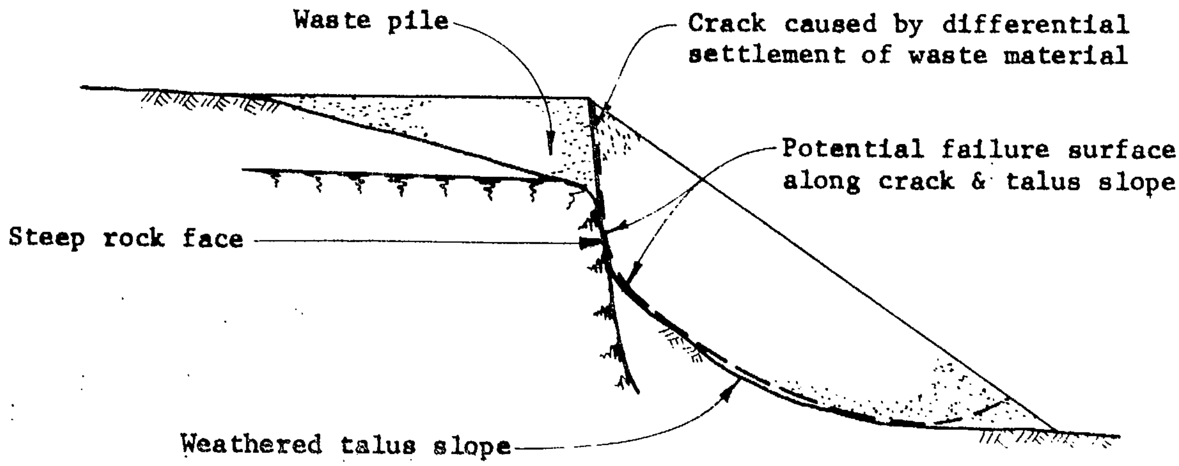
FOUNDATION CONDITIONS AFFECTING STABILITY

FIGURE 2-4

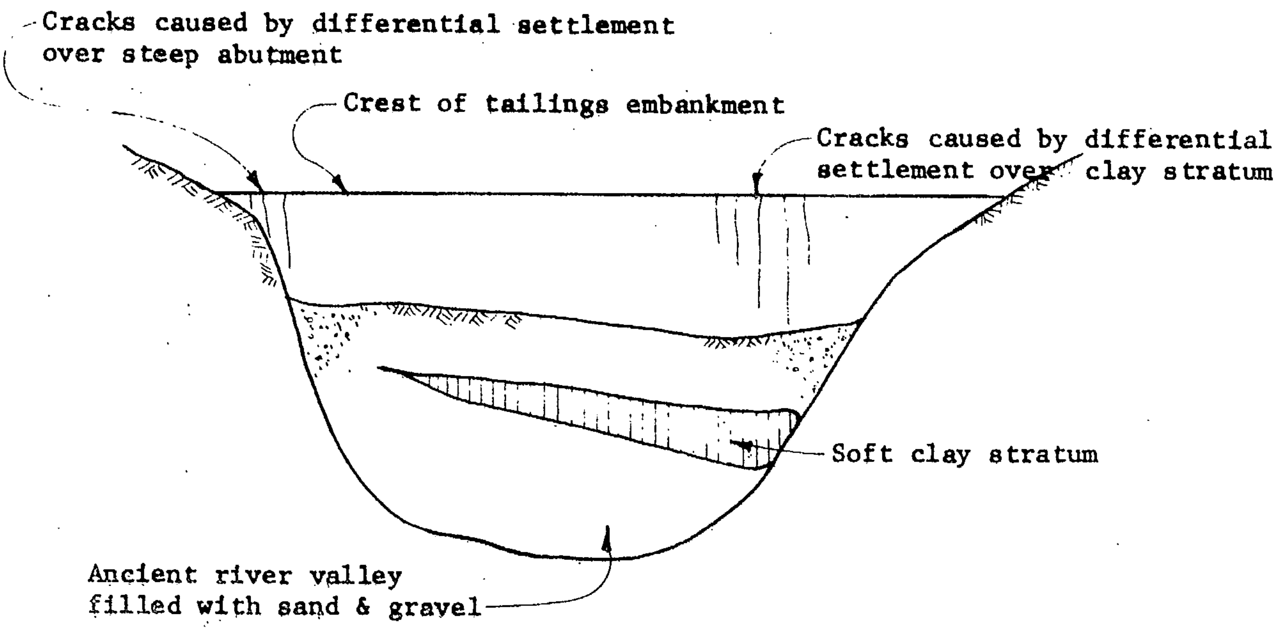


FOUNDATION CONDITIONS AFFECTING STABILITY

FIGURE 2-5



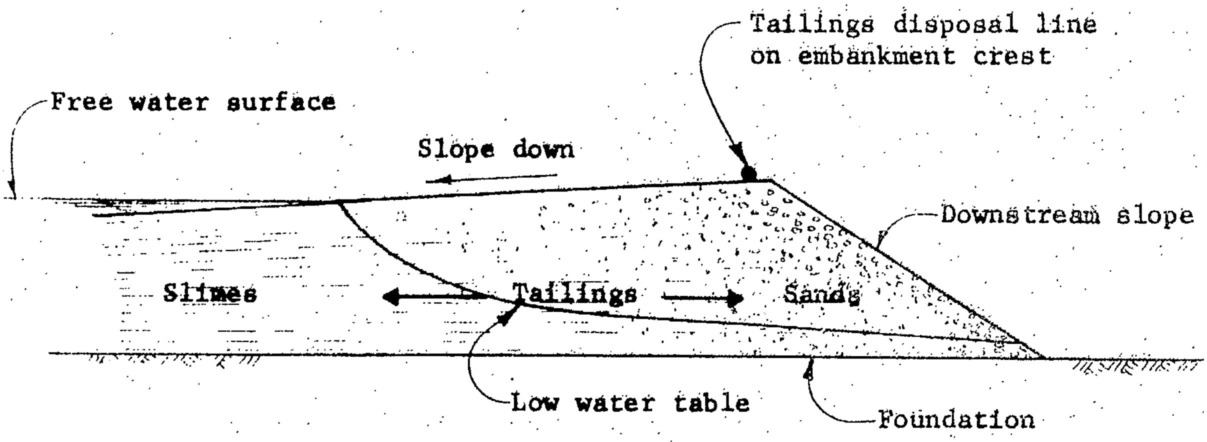
OVER STEEP FOUNDATION SURFACE



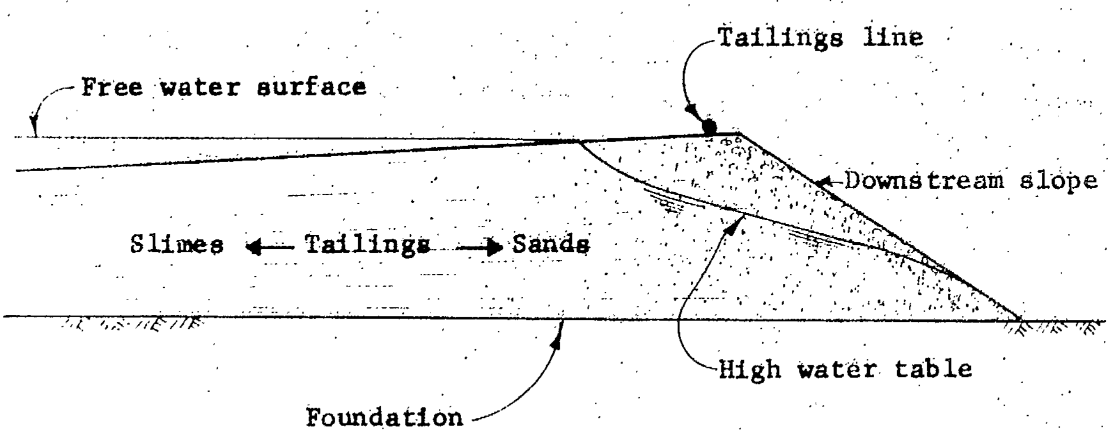
OVER SOFT FOUNDATION STRATUM

DIFFERENTIAL SETTLEMENT AFFECTING STABILITY

FIGURE 2-6

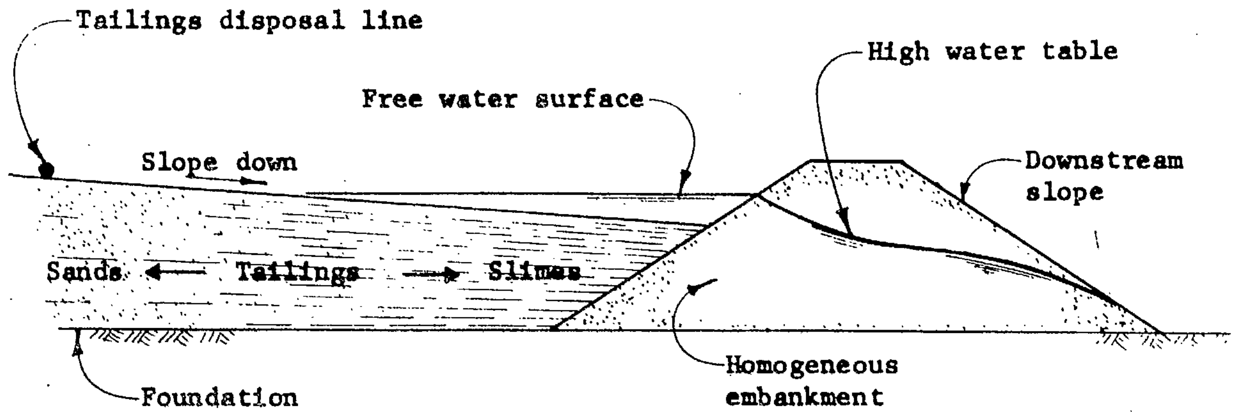


POND DISTANT FROM CREST

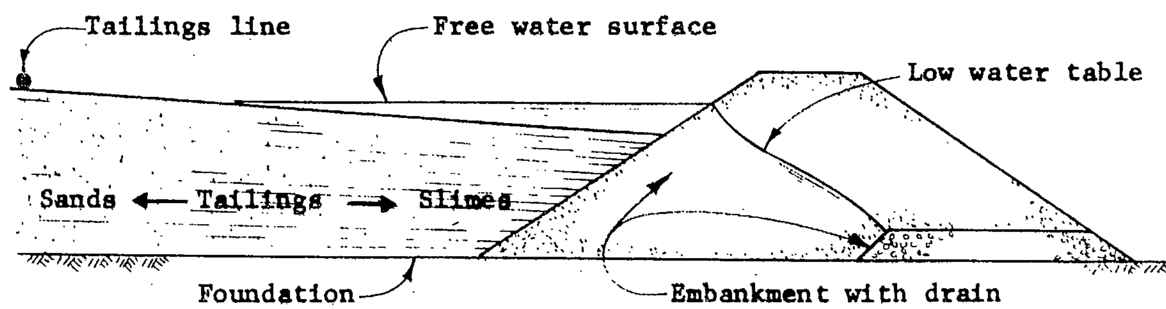


POND NEAR TO CREST

TAILING EMBANKMENTS
EFFECT OF POND LOCATION
ON WATER TABLE
FIGURE 2-7



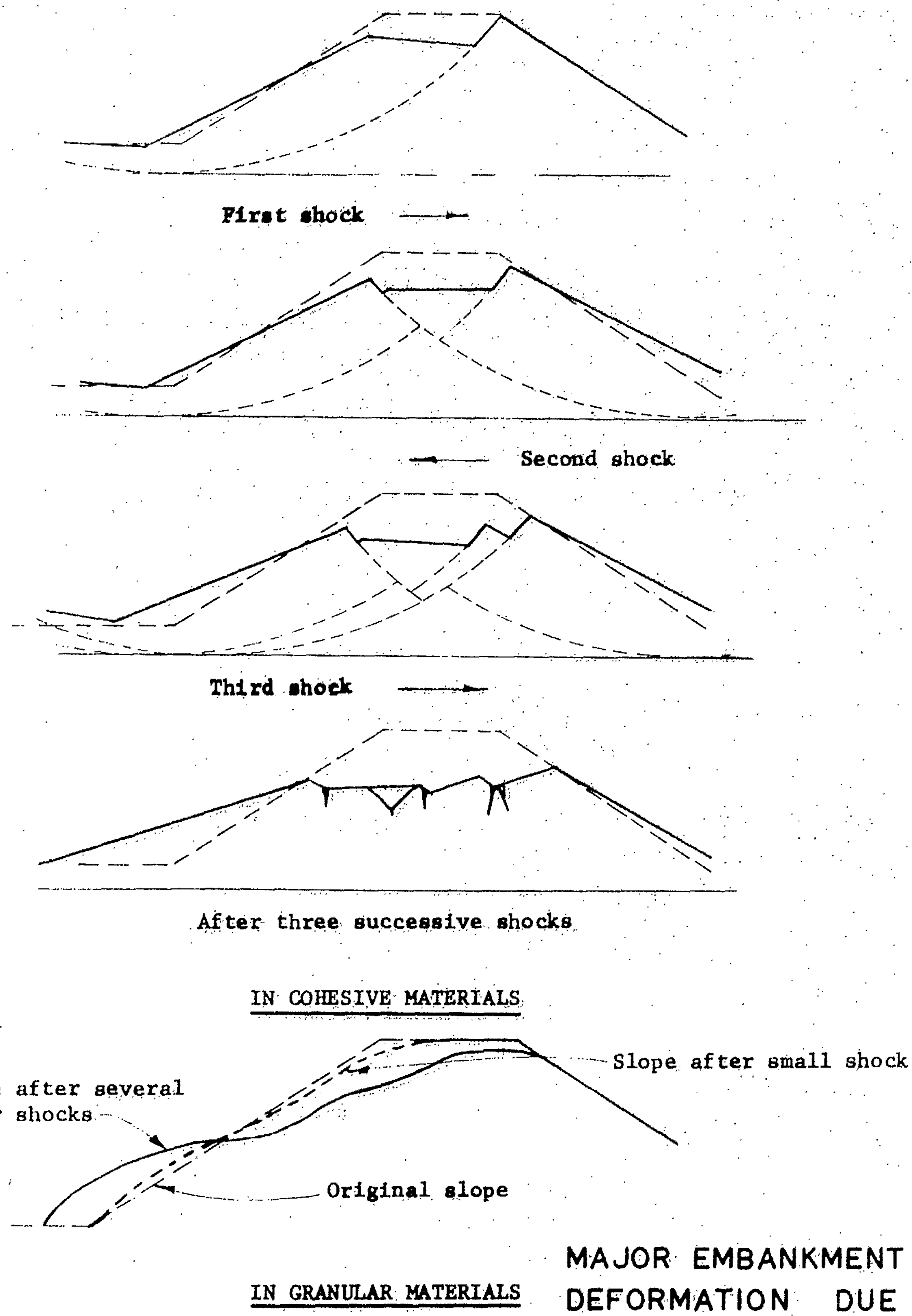
IN HOMOGENEOUS EMBANKMENT



IN ZONED EMBANKMENT

TAILING EMBANKMENTS
EFFECT OF DISPOSAL AT POINTS
DISTANT FROM EMBANKMENT
ON WATER TABLE

FIGURE 2-8



MAJOR EMBANKMENT DEFORMATION DUE TO EARTHQUAKE

FIGURE 2-9

After Ambraseys

SECTION 3

PROPERTIES OF MATERIALS

CONTENTS | PAGE

FIGURES

SECTION 3

PROPERTIES OF MATERIALS

MINERALOGICAL

General

There has been little quantitative data assembled on the physical and chemical properties of various classes of wastes from different mining operations, as they may affect the design of waste embankments. However, it is known that some types of wastes have particular characteristics which can affect the design requirements and stability of embankments in which they are stored. Some qualitative notes are given in this Section on characteristics of wastes from three classes of mining. The three classes of mining are:

coal,

industrial minerals.

Those mined in Canada as principal products include:

arsenious oxide; asbestos; barite; feldspar; gypsum; magnesitic dolomite and brucite; nepheline syenite; potash; pyrite, pyrrhotite (for production of sulphur and sulphuric acid); quartz; salt; soapstone, talc; pyrophylite; sodium sulphate and sulphur.

metals

Those mined in Canada as principal products include:

cobalt; columbium; copper; gold; iron (also produced as a by-product from pyrite and pyrrhotite); lead; magnesium; mercury; molybdenum; nickel; silver; tungsten; uranium; and zinc.

Those produced in Canada as by-products include:

antimony; bismuth; cadmium; platinum and tin.

Physical Properties

1. Grain Size

There is no general or "normal" grind for tailings from different ores. The size and gradation of the tailings wasted from the treatment plant will depend on the grain size of the native rock and the "grind" necessary to liberate the marketable minerals. Some grinds usually kept as coarse as possible are those of asbestos and potash, premium prices being obtained for coarse products with these minerals.

The grain size of the tailings has an important influence on the slope adopted by the surface of the tailings deposits in the pond and, consequently, on the height of embankment required to store a particular volume of solids. Generally, the coarser the tailings the steeper will be the slope adopted. It can range from about 60 per cent for tailings coarser than 3 m.m. grain-size (+6 mesh) to less than 1 per cent for tailings finer than 0.07 m.m. (-200 mesh). (A second factor affecting the slope assumed by a given tailings material is the ratio of water to solids at the time of discharge).

2. Grain Shape

The shear strength of a waste material is affected by the shape of its particles. Slabby rock pieces such as those resulting from excavation of serpentine and hard shales will tend to slide over each other. They will thus tend to have unstable angles of repose and be susceptible to sliding when subjected to the shock loads resulting from dumping.

In tailings, the shape of the grain will depend on the nature of the native rock and its manner of fracturing during grinding. Minerals such as halite and pyrite will tend to produce cubic particles; quartz and feldspar particles will tend to be angular. Peridotite, olivine, serpentine, sericite, mica and talc will tend to produce plate-shaped particles, generally resulting in lower shear strengths.

3. Thixotropic Characteristics

If a sample of a very fine soil fraction is thoroughly kneaded and is then allowed to stand without further disturbance, it acquires cohesive strength, first at a relatively rapid rate and then more and more slowly. If the sample is again kneaded at unaltered water content, its cohesion decreases considerably but, if it is once more allowed to stand, its cohesion is completely regained. This effect is known as "thixotropy". The softening and subsequent recovery seem to be due to the destruction and subsequent rehabilitation of the molecular structure of the adsorbed layers.

These thixotropic effects occur with materials such as bentonite, clays, fine gauge materials, potash "clays" (the water insoluble portion of the ore) and basic rock types, to a greater extent than they do with siliceous ores. In particular, they can occur with finely ground tailings from shales, peridotite, olivine, serpentine, sericite and talcy minerals.

4. Cementing

Well graded soil or tailings materials will tend to bind together better than gap graded mixtures, particularly when there is just sufficient quantity of the fine fractions to fill the voids between the coarser particles.

Some mixtures containing minerals such as clays, bentonite and finely ground shales have better binding characteristics than others. Soluble minerals such as halite form actual cements, brine cooling and evaporation from the voids forming crystal bridges between the grains. Oxidizing sulphides will cement to other minerals. Such processes can result in very rigid stable deposits.

5. Settling Rate

Sedimentation of tailings in ponds occurs in more-or-less two distinct stages. Settling of the coarser particles usually occurs within several days; settling of the fine colloidal material suspended in the remaining "free" water may never occur naturally. Characteristics of native rocks, and of ore processing methods, which increase the percentages of fine and colloidal materials in the tailings will slow both stages. Low specific gravity materials will also be slow in settling. Coal fines, and tailings from the naturally fine grained and platy minerals such as talc, chlorite and mica will generally have slow sedimentation rates. Flocculating aids may be required for such minerals, to clarify the water for reclaim or release from the pond.

6. Oil Coating

Tailings resulting from the extraction of oil from tar sands can be oil-wet after processing. This oil coating of individual grains could affect the frictional component of the tailings' shear strength.

Chemical Characteristics

1. Oxidation

Pyrite, and rocks containing pyrite, are often found in wastes from iron and coal mines. Such materials oxidize continuously while lying on the surface of a waste pile. Oxidation of pyritic minerals within the pile is determined by the rate of penetration of air into the pile and this depends upon the particle size distribution, the degree of compaction, and the degree of saturation with water. The products of pyrite oxidation are extremely acidic and are water soluble, but these acidic products may be neutralized rapidly by other alkaline minerals in the waste.

The effects of these neutralizing reactions are:

the pH value often remains near 7.0 and seldom falls below 3.0,

iron oxides are precipitated, giving rise to ochreous staining of the waste,

calcium sulphate crystals form inside fracture planes in the other rocks, and sodium sulphate crystals form as efflorescence in dry weather on the surface of the pile, the shales in proximity to the pyrite may be converted into plastic clays by the action of sulphuric acid,

any water draining from the pile may be contaminated with acidic or neutral salts.

These chemical changes may result in acid and sulphate attack on concrete, pollution of drainage water, and reduced fertility of the weathered waste materials.

The sulphate content of highly weathered unburnt coal waste is normally sufficiently high to require special precautions in the design of concrete structures which may be in contact with the waste or with water from the pile. This problem will be intensified if the waste has a low pH value.

Drainage water flowing from a coal waste pile may contain concentrations up to several thousand parts per million (p.p.m.) of ferrous, ferric and aluminum sulphates, and of calcium and magnesium sulphates derived from the action of sulphuric acid on carbonate minerals and clays. Concentrations of manganese salts, up to 10 and sometimes 100 p.p.m., may also be derived from the latter source. Many of the iron bearing waters are relatively clear when they emerge from the pile, but rapidly become ochreous on exposure to the atmosphere. In some coalfields, waste pile seepage waters may contain sodium chloride in concentrations of a few hundred p.p.m. which has probably been derived from the waste materials, but the concentration may rise to thousands of p.p.m. when tailings ponds are located on the waste pile. Some of these waters may cause pollution if they are discharged directly into a stream or on to adjacent land.

Fresh coal waste is normally neutral or slightly alkaline in reaction, but as weathering proceeds it may become increasingly acidic. High acidity, and high local concentrations of salts of manganese, iron and particularly aluminum, may increase the toxicity of the surface of the weathered pile and inhibit or prevent the growth of vegetation.

Other minerals such as pyrrhotite will oxidize in waste piles, producing soluble salts and sulphuric acid. Some minor amounts of arsenopyrite, texahedrite and tenanite also occur in some tailings; these minerals will oxidize.

These oxidation processes tend to produce acidic and toxic conditions in tailings pond effluent and in water running off, or seeping through, waste piles. They may contribute somewhat to increased stability, also, by producing some cementing of fine waste materials.

2. <u>Toxicity</u>

The processing of some minerals results in tailings effluents which are toxic to persons, animals and plants. These tailings can include gold ore cyanidization plant residues, arsenopyrite residue, high sulphide tailings (which produce acidic effluents), potash plant brines (which kill plants) and uranium plant tailings.

For such tailings, retaining embankments will usually have to be designed to limit seepage losses to a practicable minimum.

Coal Wastes

In coal mining there will generally be several separate types of waste. Strip mining usually produces a volume of waste shales, siltstones and sandstones amounting to several times that of the raw coal mined. The tendency for these shales and siltstones to degrade by weathering in the waste piles has been described previously.

Cleaning of the raw coal in the treatment plant often results in about 15 - 20 per cent waste, the bulk consisting of coarse material, usually containing a high percentage of shale particles, and the remainder consisting of fine coal particles and clay. These coarse and fine wastes can be stored separately or together. Sometimes, the coarse materials are used to construct settling pond embankments for the fine wastes. If the percentage of fine waste is high and/or the mine output large, this procedure can require very large ponds, because of the relatively slow sedimentation rate of the fine and low specific gravity materials. In other cases, the slurry containing the fine wastes is filtered in the plant and the resulting filter cake material hauled away to separate waste piles, or combined with the coarse wastes. The filtered material is still relatively wet when it leaves the plant and is a sticky and soft material to handle in haulage units and on waste piles.

It is important to recognize the differences in permeability, shear strength and handling characteristics of these different types of coal wastes when designing embankments for them. The presence of clays and clay shales in these wastes can substantially reduce their shear strength. With fine clay-like slimes in the slurries, flocculating agents may be required to clarify pond effluents.

Industrial Mineral Wastes

1. General

In most cases, the proportion of waste to ore in the mining of industrial minerals is low in comparison to that in metal mining, since the unit value of the mineral is too low to justify the mining and treatment of low grade ores. However, in most respects, the disposal of industrial mineral wastes is similar to that of metallic mineral wastes. Some specific problems encountered are described below.

2. Asbestos Wastes

Coarse and fine wastes originating from asbestos mills can be dumped separately but are usually conveyed to one combined waste pile. The presence of clays, and some types of shales and serpentine, which will soften on exposure to the atmosphere and to water can affect the stability of such waste piles.

Probably, the main hazard from asbestos wastes comes from dust on the waste piles. The ore is dried prior to separation of the asbestos; the fine waste materials are usually warm, or hot, and severe dusting occurs at the discharge point on the pile. Water is usually sprayed onto the waste, either on the conveyor or on the pile, to reduce dust losses but wetting is not always complete and some dusting still occurs. Winds will also blow dust from the pile if the fine wastes are not kept wet, or stabilized by a "binder" chemical. Apart from the pollution aspects of this dusting, the very fine asbestos fibres in the dust can cause illness in persons inhaling it. A too liberal use of water on the pile could also affect its stability.

3. Potash Wastes

Potash mill tailings usually range in grain size from a maximum of about 5 m.m. (4 mesh) down to very fine salt (NaCl). They generally contain minor amounts of potash (KCl) and varying quantities of finely divided insoluble "clays" (usually consisting of about equal amounts of carbonates and silicates). These tailings are often pumped to settling ponds in a saturated salt-potash brine. The relatively coarse salt portions of the waste settle and drain quickly, setting in place to form rigid and brittle, but partially soluble, deposits. However, the "clay" materials, which cannot normally be thickened by settling to more than 25-30 per cent solids (by weight), settle slowly and entrain air, often producing a frothy scum on the surface of the settling pond. Clear brine can be decanted from the settling pond if sufficient settling area is provided and baffles are used at the decant intake to hold back the scum.

Problems have been encountered in potash waste disposal in using a single settling pond in which a large pool of brine is continually held in the pond. Being partially soluble, the settled waste deposits will develop solution channels which allow brine to escape from the pond onto surrounding lands. These channels may develop at cracks caused by contraction or differential settlement of the relatively rigid deposits, or at points where unsaturated brines percolate through the deposits. (Runoff water can cause such unsaturated conditions). The escape of this brine onto surrounding lands, or into the sub-soil, can contaminate surface or ground water aquifers.

Because of this danger of the escape of brine from the main settling pond, it will often be necessary to use natural basins to retain potash wastes, to construct retaining embankments of borrow materials, or to use separate ponds for final settling of the "clay" portion of the waste. With such separate ponds, the bulk of the brine is decanted from the primary pond to supplementary ponds for the sedimentation of the "clay" fraction, thus limiting the volume of brine which may escape from the primary pond. These supplementary ponds are retained by relatively impervious embankments.

The necessity to limit the seepage of brines into the sub-soil usually requires that the soils underlying potash waste ponds be of low permeability, or that impervious membranes be laid down over permeable soils prior to the disposal of tailings over them.

Metallic Mineral Wastes

1. General

Metallic minerals containing less than one percent of marketable minerals are presently being mined. The extraction and concentration of these low grade ores produce large volumes of waste. Some particular problems associated with the disposal of wastes from metal mining are described below.

2. Low Sulphide - High Siliceous Tailings

Tailings low in sulphides, but high in quartz and silicates, often result from the mining of copper-porphyry ores. Such tailings, low in clay minerals, have poor binding properties (low cohesion). They can usually be used in the construction of retaining embankments but are very erodible when dry. This makes them susceptible to erosion by wind and water. If such erosion constitutes a serious problem, deposits of such tailings may have to be stabilized by seeding a layer of topsoil spread over them.

3. High Sulphide Tailings

The mining of massive sulphide ore bodies (for example, of copper, lead, iron and zinc) sometimes results in tailings having a high sulphide content (particularly if iron sulphide is not removed). In concentrating these ores, the heavier sulphides (generally pyrite and pyrrhotite but also some minor amounts of others) are more finely ground than the lighter siliceous fractions. After deposition of the tailings, these finely ground sulphides will partially oxidize and cement to form a surface crust in dormant areas of the tailings pond; (areas not receiving tailings). Such a crust will reduce surface erosion by runoff from the tailings deposits. However, if it forms on the downstream face of the retaining embankments, it could, because of its lower permeability, raise the water table within the embankment and so reduce its stability.

If the sulphide content of the tailings is very high, they can, under favourable conditions of moisture and air, spontaneously ignite and burn. Usually, such burning can be controlled by smothering the burning materials with more tailings pulp but the permeability and stability of the deposits can be affected.

4. Gold

In the extraction of gold, ores are often treated with sodium cyanide, a poison. The extent of the cyanide treatment will depend on the manner in which the gold occurs in the ore, varying from complete treatment of all the ore to treatment of only a small portion from which it is difficult to separate the gold. The tailings effluent may have a sodium cyanide content (in solution) varying from a maximum of about 3 pounds per ton (1500 m.g. per litre) to minor amounts of about 0.01 pounds per ton (5 m.g. per litre). Although much of the cyanide is present in the tailings as complex compounds, and some degrades or is absorbed in the settling ponds, the effluent can be a hazard to persons and animals. Where water quality standards require, the effluent can be treated with chlorine and caustic to produce sodium cyanate and so reduce the free cyanide to an acceptable concentration.

Gold bearing arsenopyrite usually requires roasting to remove the arsenic and thus allow leaching of the gold. The arsenic produced from this roasting is highly toxic and must be collected and stored, as it is not usually saleable. It is usually stored in weather proof housing on the surface, or underground in suitable dry stopes.

The danger of the release of toxic effluent from gold extraction plants usually requires that tailings embankments be conservatively designed to ensure the minimum practicable seepage from the pond and that effluents from these ponds be monitored and, if necessary, treated to prevent the release of toxic solutions.

5. Uranium

Uranium mines using the acid leach process produce effluent wastes with very low pH and having a high content of metal in solution. These acidic effluents must be neutralized (usually with lime) to a pH of 7.0 to reduce the toxic components to an acceptable level before disposal of the tailings to the settling pond.

Tailings from uranium mining usually contain small amounts of radium 226 and strontium 90, dangerous radioactive materials. The toxic nature of these effluents makes any embankment failure which would release them a potentially serious matter.

GEOTECHNICAL

General

The general influences of material properties on the stability of mine waste embankments have been described in Section 2. In this Section these properties are described in more detail, typical ranges of values being given for those properties significant to design, so that their general differences can be gauged. It can be seen from the included data that characteristic values can vary over wide ranges and that they can overlap from one soil classification to another. For this reason, it is important that values representative of the actual waste, fill and foundation materials be determined by test for the preparation of final embankment designs. Suitable testing procedures are described in Section 4.

Soils and Fine Wastes

1. Grain Size Distribution

Grain size distribution (gradation) is the schedule of percentages of a material falling in various size categories in a generally accepted series of sizes ranging from 100 mm to less than 1 micron (1/1000 mm). Materials are often classified, on the basis of their average grain size, into four general categories: gravel (2 mm to 60 mm), sand (0.06 mm to 2 mm), silt (0.002 mm to .06 mm) and clay (less than 0.002 mm). Soils can be "well graded" (containing relatively constant proportions of the various size classes in the total mixture) or "gap graded" (in which there are only small amounts of one or more size classes in relation to other sizes). The Coefficient of Uniformity (U) is commonly used to describe the gradation of materials. This coefficient is defined as the ratio of the 60 per cent size of the material to the 10 per cent size of the material:

$$\text{i.e. } U = \frac{D_{60}}{D_{10}}.$$

Materials having U values of 3 or less are considered poorly graded. A well graded material, such as glacial till, can have a U value of 8 or greater.

The wide variation in the gradation of individual soil types encountered in foundations and fills can be seen from Figure 3-1. In general, the geotechnical properties of a given soil are greatly influenced by the characteristics of the finest fraction of that soil.

Tailings have a wide variation of grain size distribution, depending on mill operation, but usually range from coarse sand to colloidal material (see Figure 3-2). Generally, the particles are angular, especially when derived from hard rock sources.

The usual range of grain size distribution for coal waste is shown on Figure 3-3. Coal waste can be expected to become progressively finer after being placed in storage because of the effects of weathering.

2. Plasticity

The plasticity characteristics of materials are described by the liquid limit, plastic limit and shrinkage limit; these limits, which are expressed as water contents, are known as the Atterberg limits. In general, the plasticity of a soil is not significant where the proportion of fines (-200 mesh) is low.

Soils may be classified according to their plasticity. This (Casagrande System) method uses the relationship between liquid limit and plasticity index, the latter being the difference between liquid limit and plastic limit, (see Figure 3-4).

Plasticity characteristics of various soils are tabulated on Figure 3-5. The plasticity of clayey soils is influenced by the amount of clay and by the composition of the clay fraction (Scott, 1963).

Typical plasticity data for tailings from several South American mining operations which had embankment failures are tabulated on Figure 3-6; the range of values for coal waste is shown on Figure 3-7.

3. Water Content

Water content is one of the most important index properties for a given soil. It is defined as the ratio of the weight of water to the weight of dry soil, expressed as a percentage. The natural water content of a soil, or better - the liquidity index (a measure of the difference between the water content and the plastic limit), provides a qualitative assessment of the soils compressibility, sensitivity and preconsolidation.

The water contents of soils commonly range from 5 per cent to 35 per cent. However, for soil types that contain a high percentage of clay or organic material, water contents in excess of 100 per cent may occur. In general, the water content depends on void ratio, particle size, clay mineral and organic content, and ground water conditions.

Water content data for tailings from the South American mines are shown on Figure 3-6.

The water content of coal wastes generally varies from 5 per cent to 20 per cent, depending on the method of coal preparation. Large variations may also occur during the day-to-day operations of a coal processing plant.

4. Density

The unit weight or density of a material is defined as the weight of material divided by its total volume, (solids, liquids and voids). The in-place density of soils will be a function of their manner of deposition, gradation and loading history. Fine soils deposited by wind or water and not subsequently subjected to superimposed loading will be relatively loose; if subsequently consolidated by the weight of overlying soil deposits or ice, or compacted mechanically, their densities will be materially increased.

In general, when measuring the degree of compaction of a soil, the dry density (the density of the soil with the water removed while the volume remains constant) is used. For a given compactive effort, the maximum dry density is obtained at a particular "optimum water content", (see Figure 3-8).

Typical ranges of density of various soil types are shown on Figure 3-9. The dry density of tailings varies from 70 lbs/cu. ft. to 120 lbs/cu. ft. The dry density of coal waste tailings obtained from different deposits ranges from 80 lbs/cu. ft. to 130 lbs/cu. ft.

5. Specific Gravity

The specific gravity of a rock or soil is the unit weight of the particles which form the sample, expressed as a multiple of the unit weight of water. The average specific gravity of granite rock is about 2.65, with a range from 2.4 to 3.6 depending on the nature of the mineral constituents. Specific gravity may be used to determine other important soil properties such as void ratio, porosity and degree of saturation (described in Appendix C).

The specific gravity of tailings particles may range from about 2.5 to 3.5, depending on the mineralogical composition.

The specific gravity of coal waste may range from less than 1.8 to 2.7, depending on the amount of coal, shale and sandstone present. The specific gravity of finely divided coal wastes is likely to be in the range of 1.5 to 2.5.

6. Permeability

The permeability of a homogeneous soil is a function of the size and number of the voids between the soil particles. These are dependent on the size and shape of the particles, the gradation of the soil and its density. In the analysis of seepage through soils, the soil permeability is defined by a "coefficient of permeability", (K), expressed as the velocity of flow through a unit area of the total soil (voids plus solids) under a unit hydraulic gradient (pressure head divided by the length of the seepage path). Knowing this coefficient, the actual hydraulic gradient and the total area through which the seepage flow passes, the rate of seepage flow through a homogeneous soil can be calculated.

Seepage through natural soil or rock deposits is dependent not only on the coefficient of permeability of the homogeneous material but also on local variations such as fissures, joints, lenses of open-work talus or gravel etc. The voids in a homogeneous soil, without fissures, can be measured in fractions of a millimetre, with consequent low coefficients of permeability. The dimensions of open fissures which can exist in natural soil or rock deposits, and in constructed embankments, can often amount to several inches. The seepage flow through such fissures can exceed by hundreds of times the flow through the homogeneous soil or rock itself. Where potential seepage is important, such as with tailings embankments retaining water, the possible existence of such fissures should be considered. They often occur in the foundation; at the contact surfaces between the embankment fill and the underlying foundation and abutments; within the fill itself, in the form of seggregated seams of stony material between compaction layers; and at contacts between pipes and walls incorporated in the fill.

The coefficients of permeability for various soils and tailings are indicated on Figure 3-10. Typical relationships between permeability and density for various types of soils and tailings are illustrated on Figure 3-11.

The influences on permeability of soil gradation, and of the nature and amount of the fines contained in the soil, are illustrated on Figure 3-12. Soils may be classified according to their coefficients of permeability, as shown on Figure 3-13.

The permeability of coal wastes depends primarily on their gradation and degree of compaction. Typical values of the coefficient of permeability of coarse wastes are in the range of 10^{-2} to 10^{-5} cm. per second. In deposits of fine waste transported as a slurry, the coarser particles tend to settle out in thin layers interfingered with layers of clayey materials and, as a result, the horizontal permeability tends to be much higher than the vertical permeability. The coefficient of permeability of slurry deposits varies from about 10^{-3} to 10^{-6} cm. per second in the horizontal direction and from 10^{-5} to 10^{-7} cm. per second in the vertical direction. Tests at the Van Stone tailings dam indicated the horizontal permeability of the tailings to be 5-10 times the vertical permeability.

7. Shear Strength

The shear strength of a soil is made up of two components, cohesion and internal friction.

$$s = c' + \bar{p} \tan \phi',$$

where s = shear strength,
c' = effective cohesion,
$\bar{p}$ = effective normal stress = $(p-u)$,
p = total stress normal to the plane of failure,
u = hydrostatic pressure on the failure plane,
ϕ' = effective angle of internal friction.

The cohesion is derived from flexible bonds between fine soil particles and is virtually independent of intergranular pressure. Cohesive soils have a high value of c' and a low value of ϕ', while cohesionless soils have a high value of ϕ' and no cohesion. Typical values of c' and ϕ' for various soil types are tabulated on Figure 3-14.

Because the frictional component of the shear strength of any given soil depends on the effective pressure, the shear strength increases with depth, (higher confining pressure). However, in determining the effective stress that contributes to the soil's shear strength, one must be certain that a proper allowance for pore water pressure is made. (Effective stress equals total stress minus pore water pressure).

Tailings usually have the characteristics of a cohesionless soil ($c' = 0$), with average effective friction angles (ϕ') that range between 30 and 36 degrees, depending on the density of the soil and the angularity of the soil particles. The shear strength of tailings is also dependent on the method of operation of the tailings facility. Cycles of wetting and drying induce fluctuations in pore water pressure and, consequently, fluctuations in shear strength.

The shear strength of coal wastes is generally low. The effective cohesion and the effective angle of internal friction vary between wide limits. C' varies usually from 0 to 1000 lbs/sq. ft., and ϕ' varies from 22 degrees to 34 degrees.

8. Compressibility

The compressibility of a soil is defined as the volume change per unit volume under an applied unit increment of stress. Soils are relatively compressible because of their high void content. The rate at which a soil settles, when loaded, depends on the permeability of the soil and the drainage conditions. The compressibility of a soil is determined by the relationship between void ratio and load-carrying capacity, and is obtained by consolidation tests (Terzaghi and Peck, 1967). Figure 3-15 shows typical curves of this relationship.

The consolidation characteristics of most tailings are similar to those of a normally consolidated soil (consolidated by its own weight only). A consolidation curve for the tailings at the East Geduld mine is shown on Figure 3-16. A gradation curve for these tailings is shown on Figure 3-2. Consolidation curves for a very loose and low density tailings material, in an undisturbed and in a remoulded condition, are shown on Figure 3-17.

Of primary interest in relation to the disposal of coal wastes is the difference in the rate of consolidation of finely divided coal and of tailings in settling ponds. Generally, the tailings consolidate at a relatively slow rate and the finely divided coal at a more rapid rate.

Rockfill

The properties of rockfill depend primarily on particle shape, size distribution, mineralogy of particles, relative density and confining pressure. Typical values of specific gravity and density are tabulated on Figure 3-5 and 3-9. The average coefficient of permeability of a well graded rockfill normally will range between 10^{-1} cm. per second and 10 cm. per second. However, the placement of rockfills in high lifts by end-dumping can result in segregation and, consequently, can produce local permeabilities that are many times greater than the average values for the fill.

Some relationships, determined by laboratory tests on crushed rocks, between the angle of internal friction and particle size, relative density and confining pressure are shown on Figures 3-18, 3-19 and 3-20. Typical values of the angle of internal friction of rockfill materials lie above 35 degrees.

Sedimentary Rocks

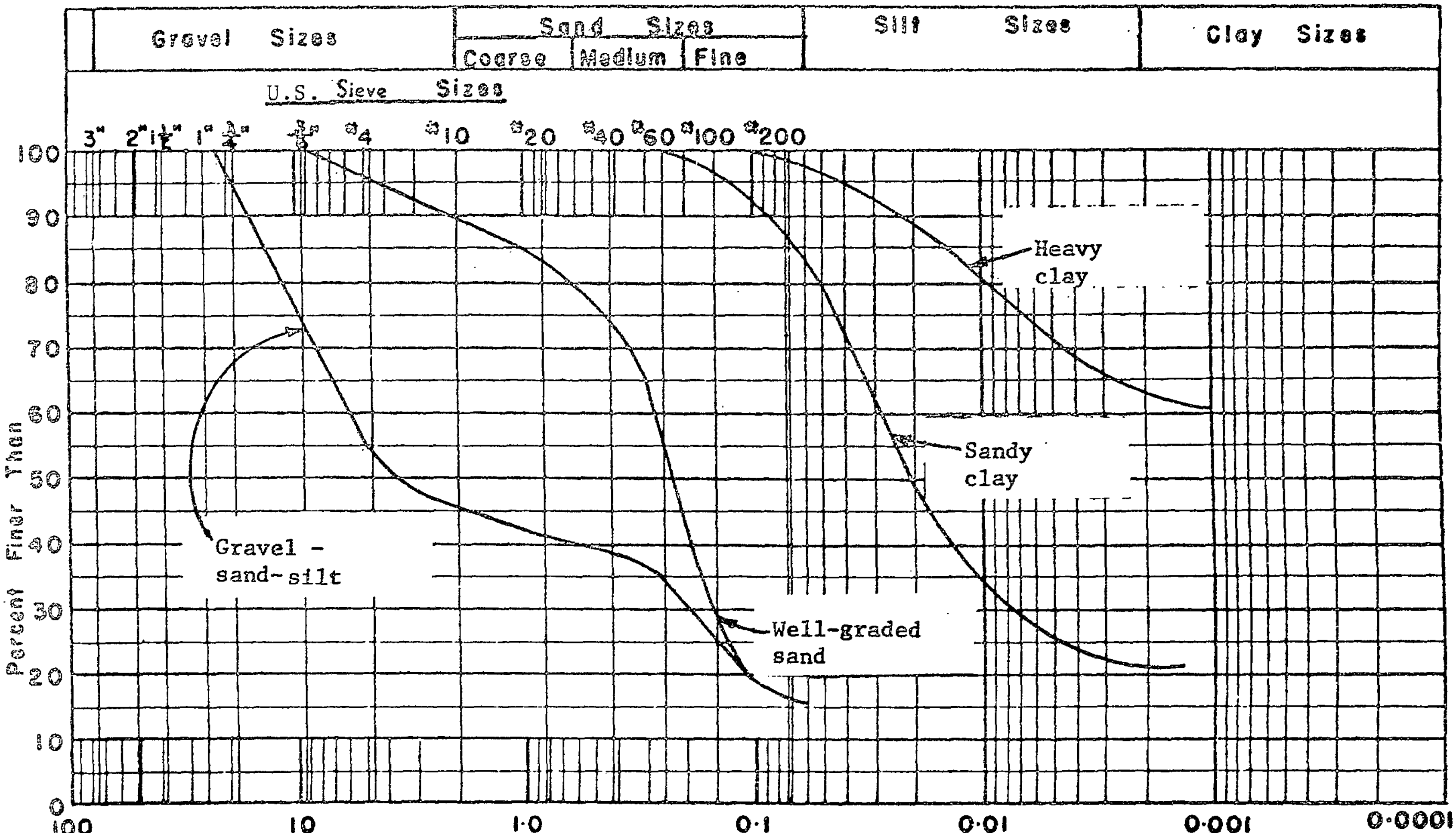
The behaviour of sedimentary rocks is affected by inherent stratification and jointing. Some frequently encountered in embankment foundations are conglomerates, limestones, sandstones, siltstones and shales.

Typical values of some important mechanical properties of sedimentary rocks are tabulated on Figure 3-21. These properties, insitu, are greatly influenced by structural discontinuities such as joints, faults and foldings. However, they are generally high in comparison to the stresses under mine waste embankments.

For tailings embankments, special attention should be given to limestone foundations, because of the effect of weathering on strength and permeability. Hard limestones, however, make a good foundation. Sandstones can be relatively pervious. The friable nature of the more porous sedimentary rocks should be considered if these rocks are to be used as fill material.

Igneous Rocks

Granite, diorite and basalt are some of the igneous rocks frequently encountered in foundations; some of their important properties are tabulated on Figure 3-21. Generally, they are adequate foundation materials except that basalt formations, in particular, can be very pervious because of heavy jointing.



Soil	Liquid Limit %	Plastic Limit %	Plasticity Index %	Specific Gravity
Heavy clay	75	23	52	2.77
Sandy clay	40	20	20	2.72
Sand		Non-plastic		2.70
Gravel - Sand-Silt		Non-plastic		2.68

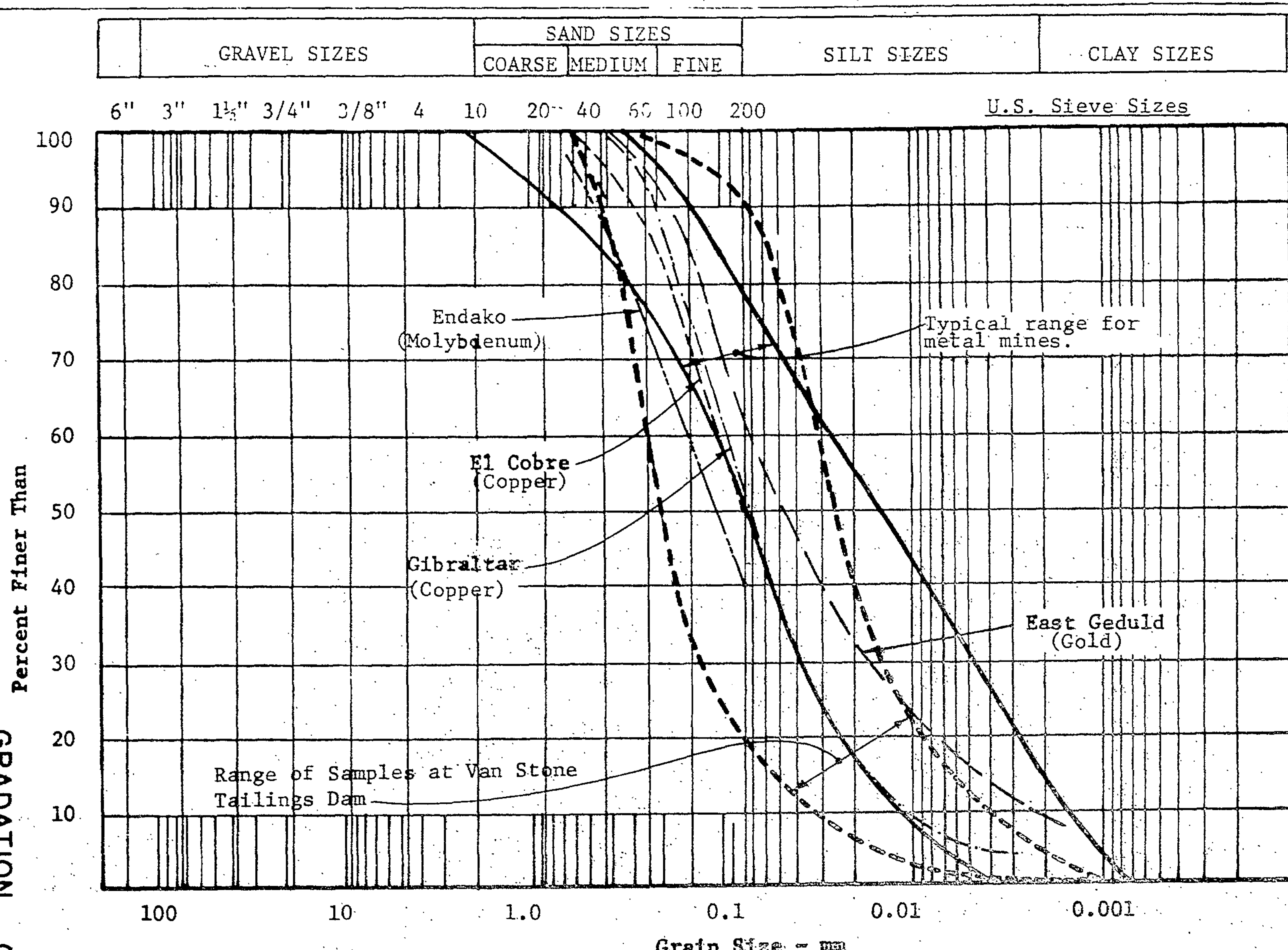
Particle Size Distribution

M.I.T. classification

(After Lambe 1969)

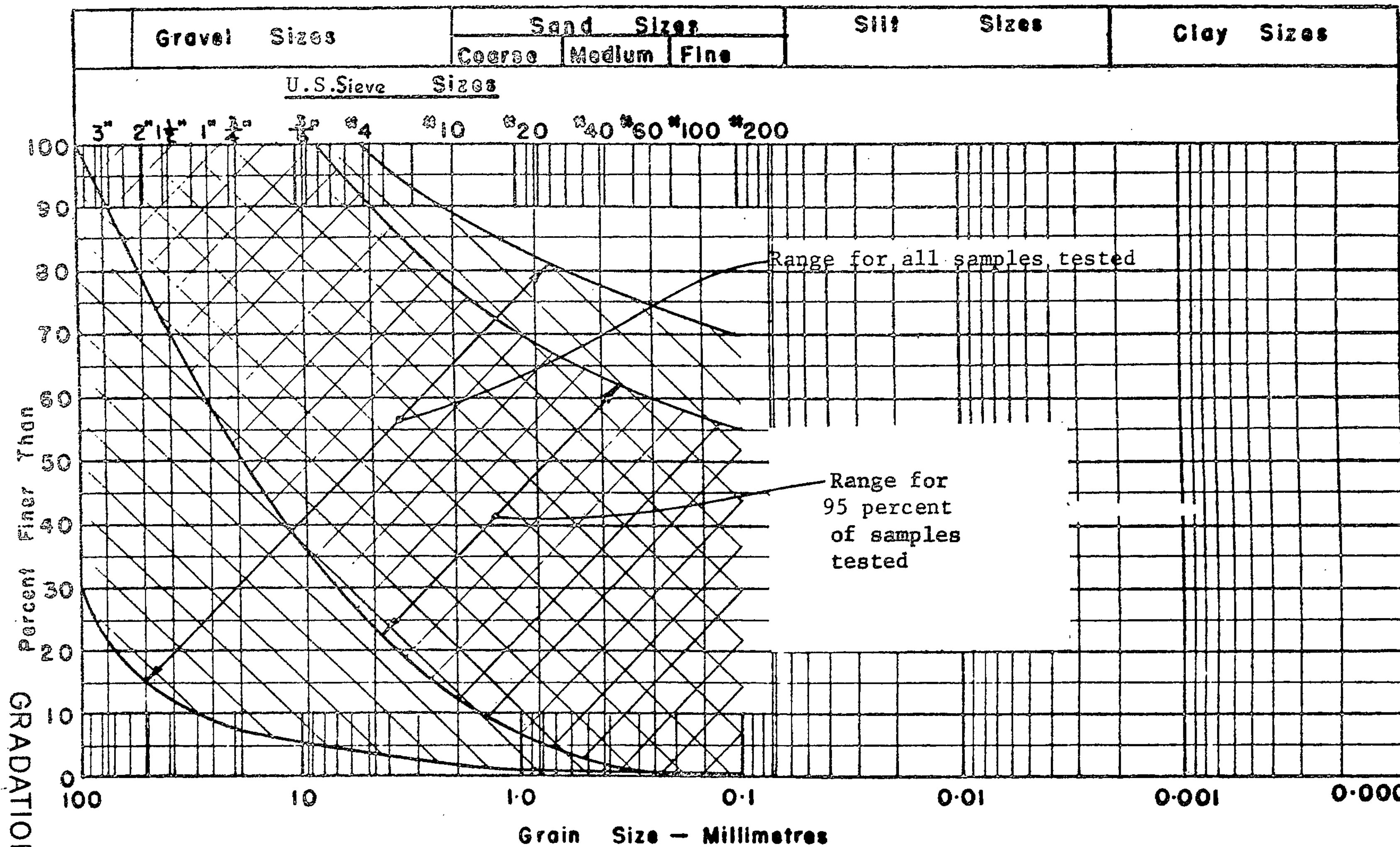
FIGURE 3-1

TYPICAL SOIL GRADATION CURVES



GRADATION OF TYPICAL TAILINGS

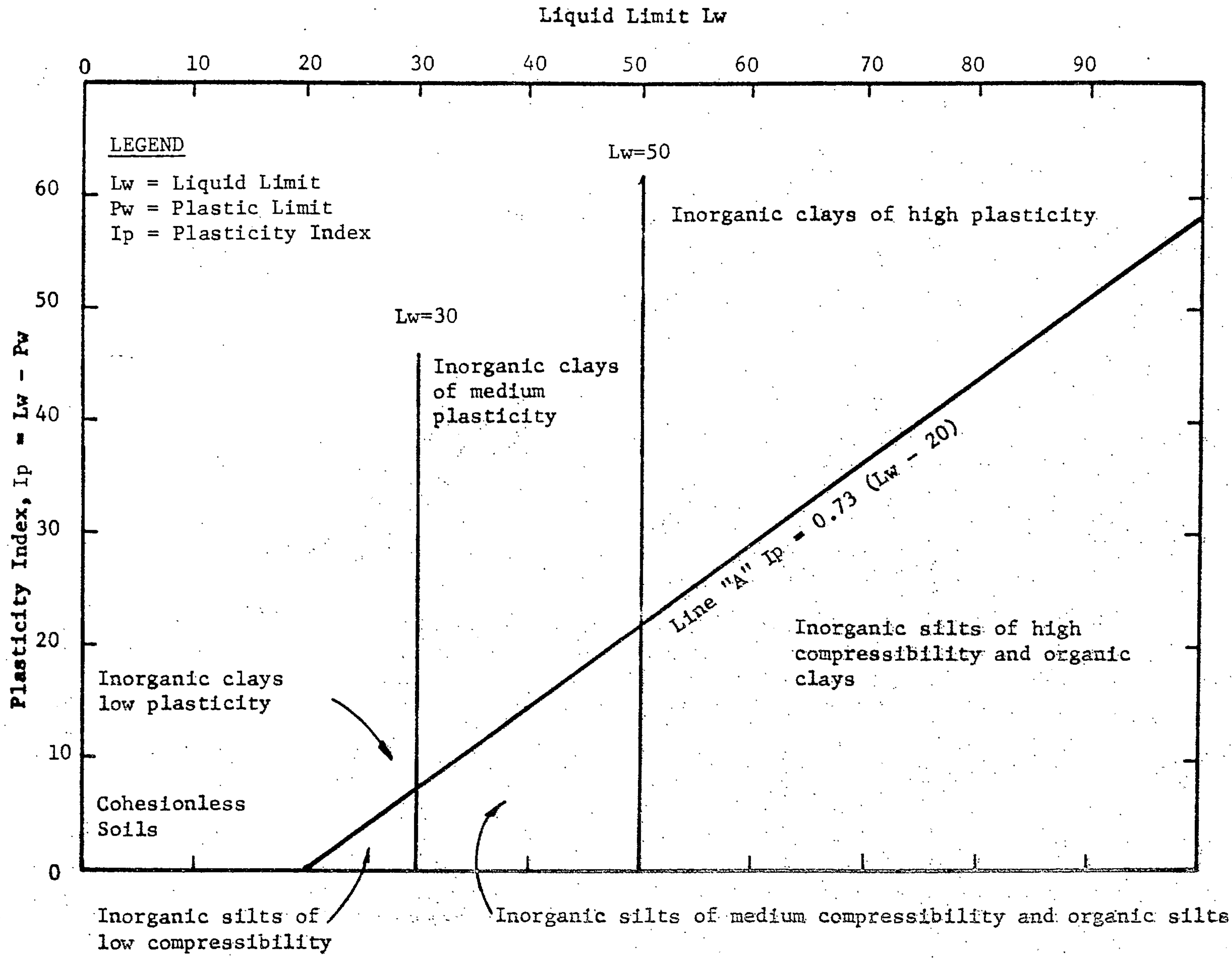
FIGURE 3-2



GRADATION OF COAL WASTES

FIGURE 3-3

(After National Coal Board, 1970)



(After A Casagrande)

FIGURE 3-4

PLASTICITY CHART

SOIL	LIQUID LIMIT	PLASTIC LIMIT	PLASTIC INDEX	SPECIFIC GRAVITY
Lean Clay	32	18	14	)
Fat Clay	80	28	52	)-Average 2.75
Sandy Clay	40	20	20	)
Sand		Non-plastic		Average 2.65
Gravel-Sand-Silt		Non-plastic		Average 2.68
Rock Fill		Non-plastic		Average 2.70

....

PLASTICITY CHARACTERISTICS OF SOME SOILS

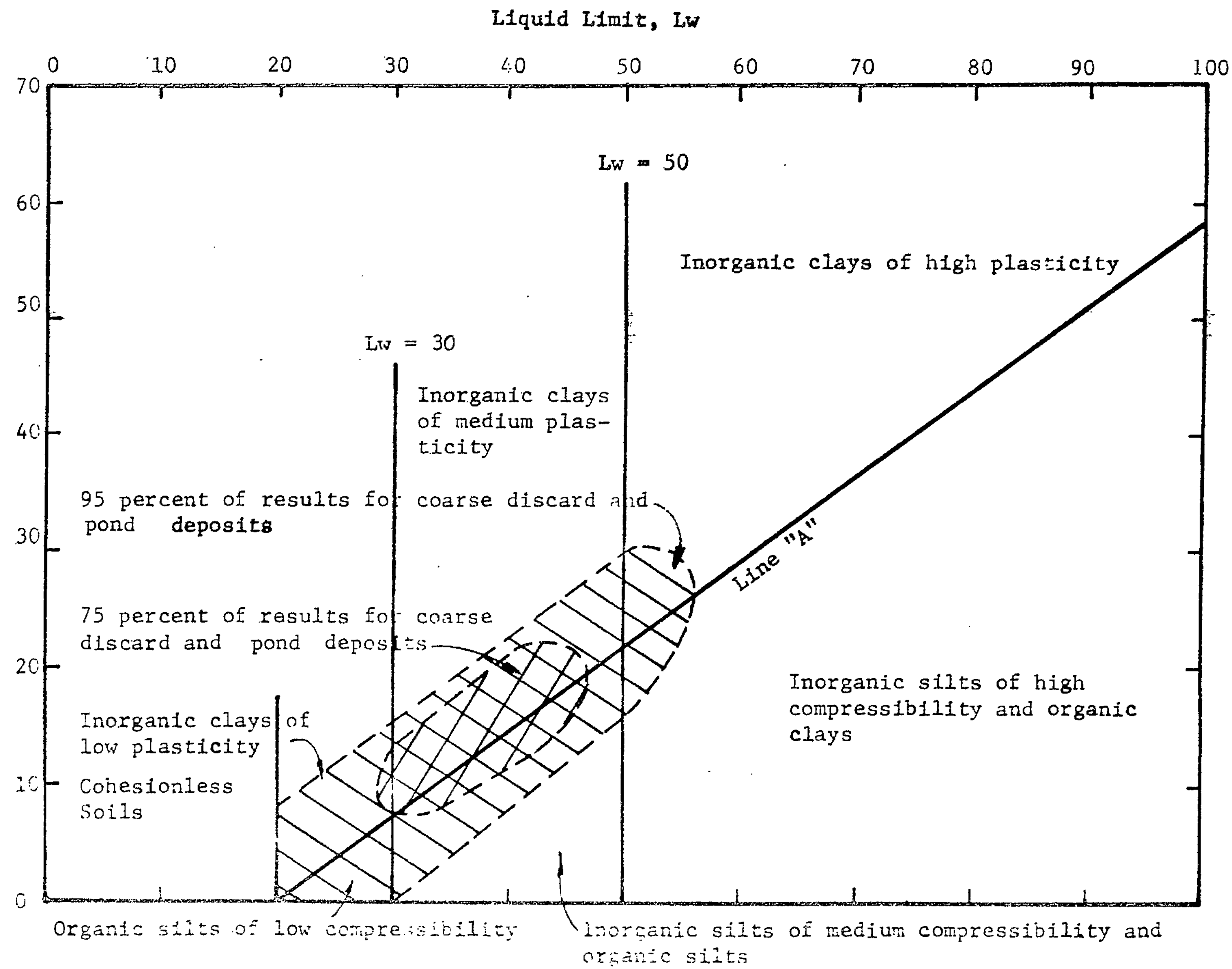
FIGURE 3-5

Locality	El Cobre	Hierro Viejo	Los Maquis	La Pataqua	Cerro Negro	Bellavista	El Sauce	Ramayana
Permeability of foundation soil, in centimeters per second	10^{-8}	low	medium	medium to high	high	medium	low	low
Percentage smaller than mesh No 200	92.8	99.6	100.0	99.8	100.0	87.5	92.9	99.5
Water content%	22.4-56.3	45.4	39.0	42.4	46.5	25.1	38.5	55.5
Liquid Limit, in percentage	19-47.7	54.7	35.1	42.8	47.0	25.6	28.5	48.7
Plasticity index	0-19	30.5	8.7	17.7	17.5	3.4	4.7	17.9

(After Debry & Alvarez, 1967)

FIGURE 3-6

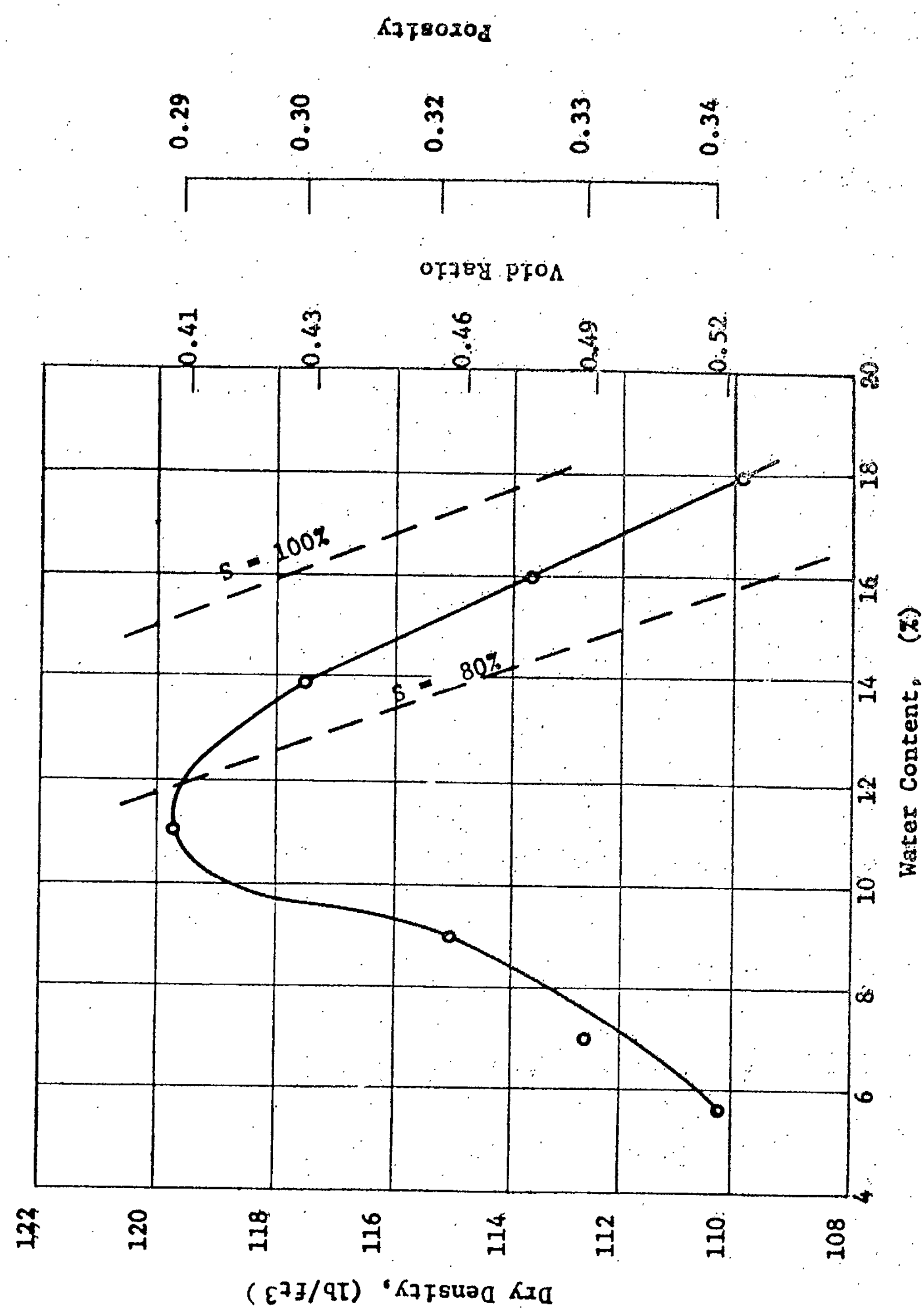
PLASTICITY CHARACTERISTICS OF SOME CHILEAN TAILINGS



(After Nat. Coal Board)

PLASTICITY OF COARSE DISCARDS AND POND DEPOSITS IN COAL WASTE EMBANKMENTS

FIGURE 3-7



NOTE:

For definitions of void ratio (e), porosity (n) and degree of saturation (s) see Appendix C.

TYPICAL CURVE
OF DRY DENSITY
VS
COMPACTION WATER CONTENT

FIGURE 3-8

Description	Porosity n (%)	Void Ratio, e	Water Content w (%)	Unit Weight lb/ft^3 γ_d	γ
1. Uniform sand, loose	46	0.85	32	90	118
2. Uniform sand, dense	34	0.51	19	109	130
3. Mixed-grained sand, loose	40	0.67	25	99	124
4. Mixed-grained sand, dense	30	0.43	16	116	135
5. Glacial till, very mixed-grained	20	0.25	9	132	145
6. Soft glacial clay	55	1.2	45	76	110
7. Stiff glacial clay	37	0.6	22	105	129
8. Soft slight organic clay	66	1.9	70	58	98
9. Soft very organic clay	75	3.0	110	42	89
10. Soft Bentonite	84	5.2	194	27	80
11. Rock Fill	41	.70	25	100	125

w = water content when saturated, in percent of dry weight
γ_d = unit weight in dry state
γ = unit weight in saturated state

TYPICAL DENSITIES OF NATURAL SOILS

FIGURE 3-9

Coefficient of Permeability k in cm per sec (log scale)

	10^2	10^1	1.0	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Drainage	Good						Poor		Practically Impervious			
Soil Types	Clean gravel		Clean sands, clean sand and gravel mixtures			Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits etc.				"Impervious" soils, e.g. homogeneous clays below zone of weathering.		
					"Impervious" soils modified by effects of vegetation and weathering							
Direct determination of k	Direct testing of soil in its original position - pumping tests, Reliable if properly conducted. Considerable experience required.											
	Constant-head permeameter. Little experience required.											
Indirect determination of k			Falling-head permeameter. Reliable. Little experience required.			Falling-head permeameter Unreliable. Much experience required.			Falling head permeameter. Fairly reliable. Considerable experience necessary.			
	Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels.									Computation based on results of consolidation tests. Reliable. Considerable experience required.		

(After Terzaghi and Peck, 1967)

FIGURE 3-10

PERMEABILITY COEFFICIENTS FOR SOILS

PERMEABILITY VS. DENSITY

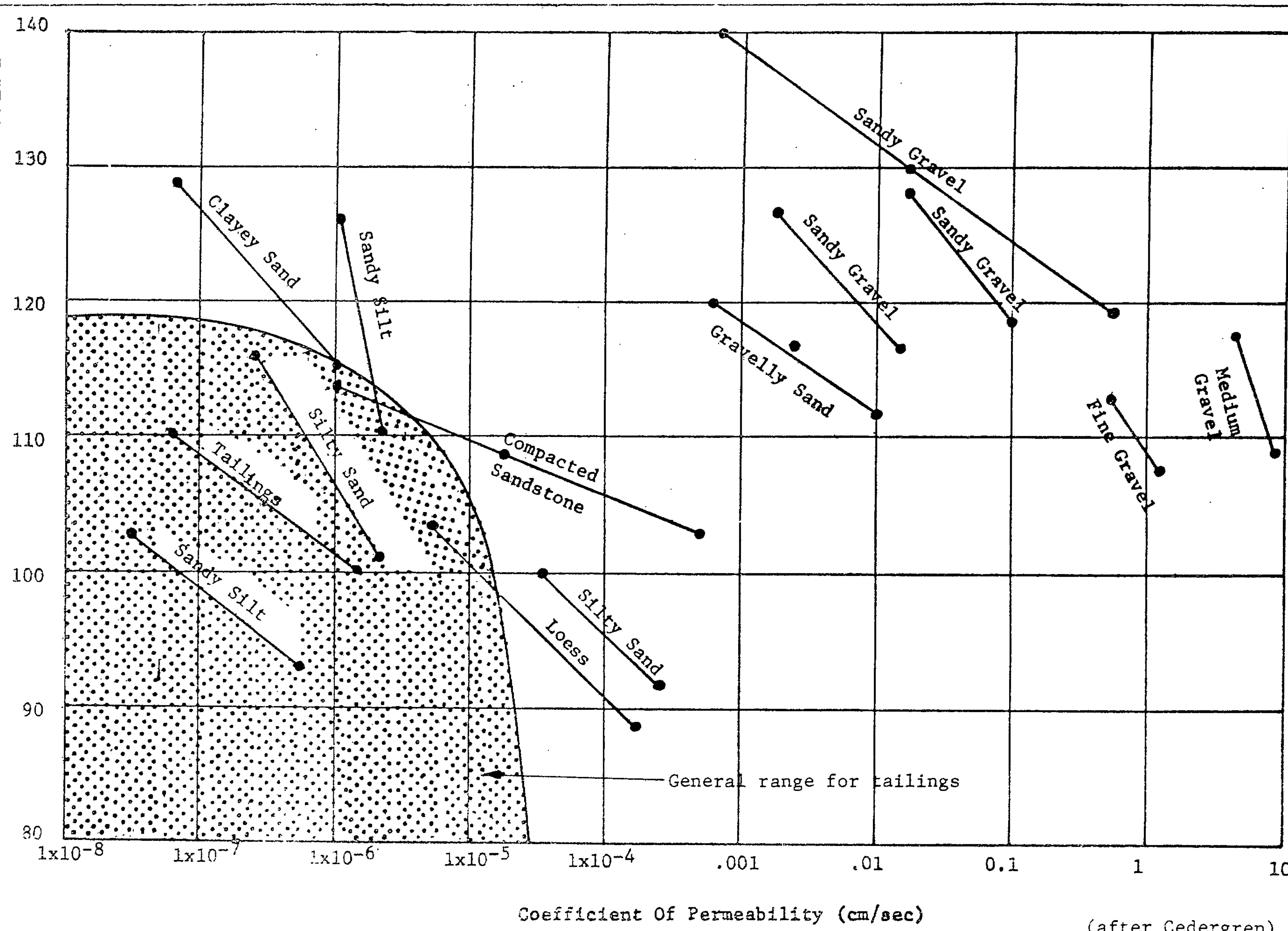
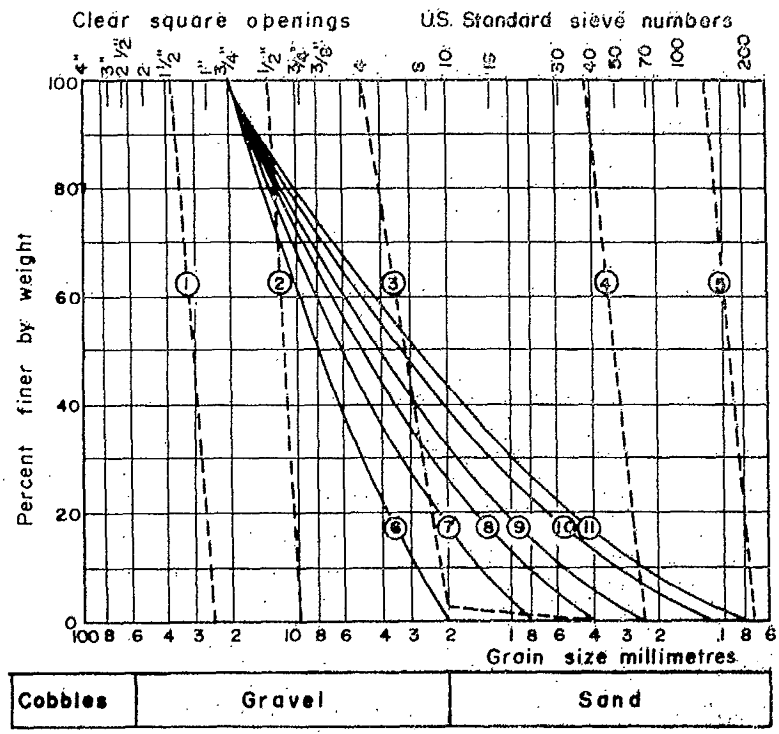


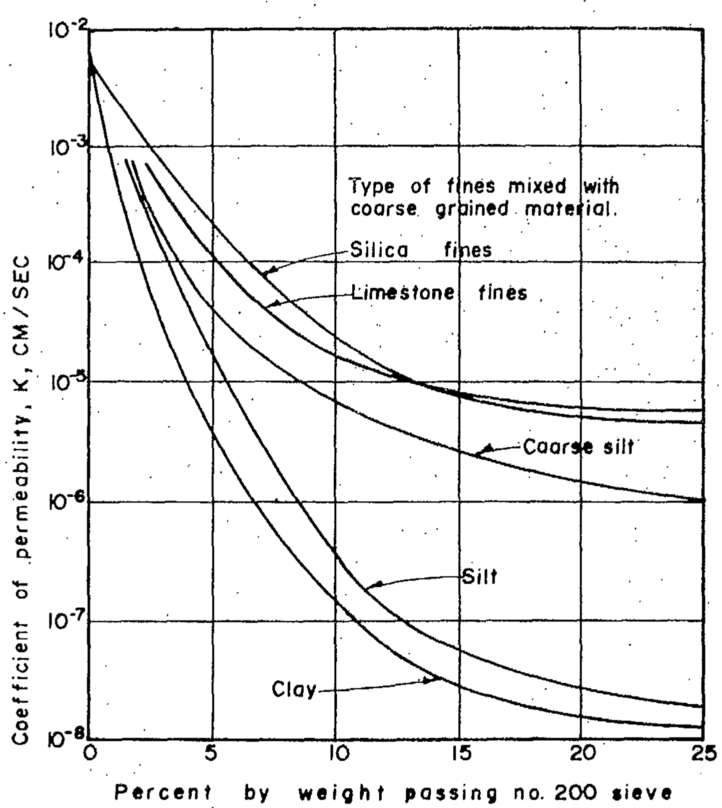
FIGURE 3-11



COEFFICIENT OF PERMEABILITY FOR CLEAN COARSE GRAINED DRAINAGE MATERIAL

CURVE	K, CM / SEC.
1	37
2	28
3	2.7
4	7×10^{-2}
5	5×10^{-3}
6	1.0
7	9×10^{-1}
8	3.5×10^{-1}
9	1.1×10^{-1}
10	4.0×10^{-2}
11	5.0×10^{-3}

PERMEABILITY VS. GRADATION



PERMEABILITY VS. FINES CONTENT

COEFFICIENT OF PERMEABILITY VS. GRADATION AND FINES CONTENT

FIGURE 3-12

(U.S. Dept. of Navy
Bureau of Yards and Docks.)

Degree of Permeability	Value of K, CM/SEC
High	over 10^{-1}
Medium	10^{-1} to 10^{-3}
Low	10^{-3} to 10^{-5}
Very Low	10^{-5} to 10^{-7}
Practically Impermeable	Less than 10^{-7}

After Terzaghi & Peck, 1967

See also Figure 3-10

PERMEABILITY CLASSIFICATION OF SOILS

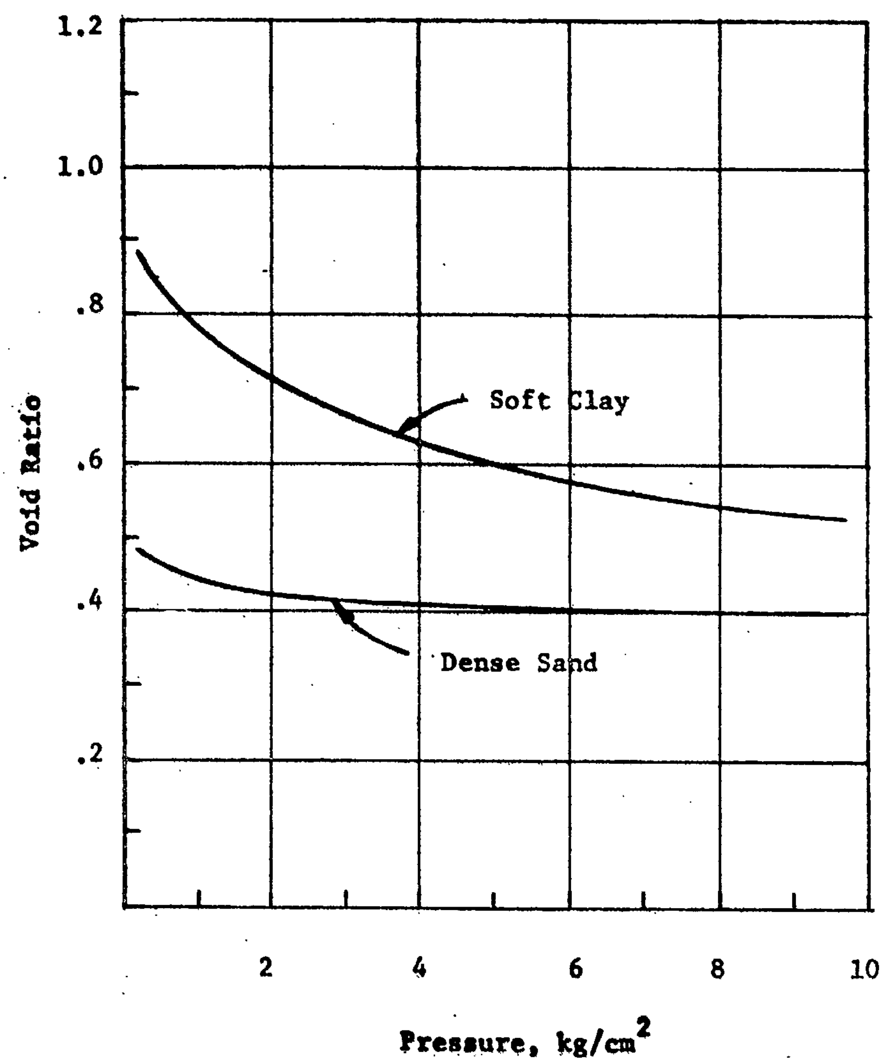
FIGURE 3-13

Soil	Effective Cohesion (c^1) (lbs./sq.ft.)	Effective Angle of Internal Friction (ϕ^1) (degrees)	
Bentonite shale	300	7	
Muddy sand	400	30	
Shale (fill cemented)	1000	34	
Sandstone (fill)	-	35 to 45	
Soft clay	400	Variable depending on rate of load application	
Very soft clay	370 to 200		
Stiff clay	1500 to 2000		
Silt (non-plastic) - medium dense		28 to 32	
Silt (non-plastic) - dense		30 to 34	
Uniform fine to medium sand - medium dense		30 to 34	
Uniform sand - dense		34 to 40	
Well-graded sand - medium dense		38 to 46*	Higher values occur at low confining pressures
Well-graded sand - dense		36 to 42*	
Sand and Gravel - medium dense		40 to 48*	
Sand and Gravel - dense		40 to 55*	

* Such high angles require confirmation by thorough and extensive testing.

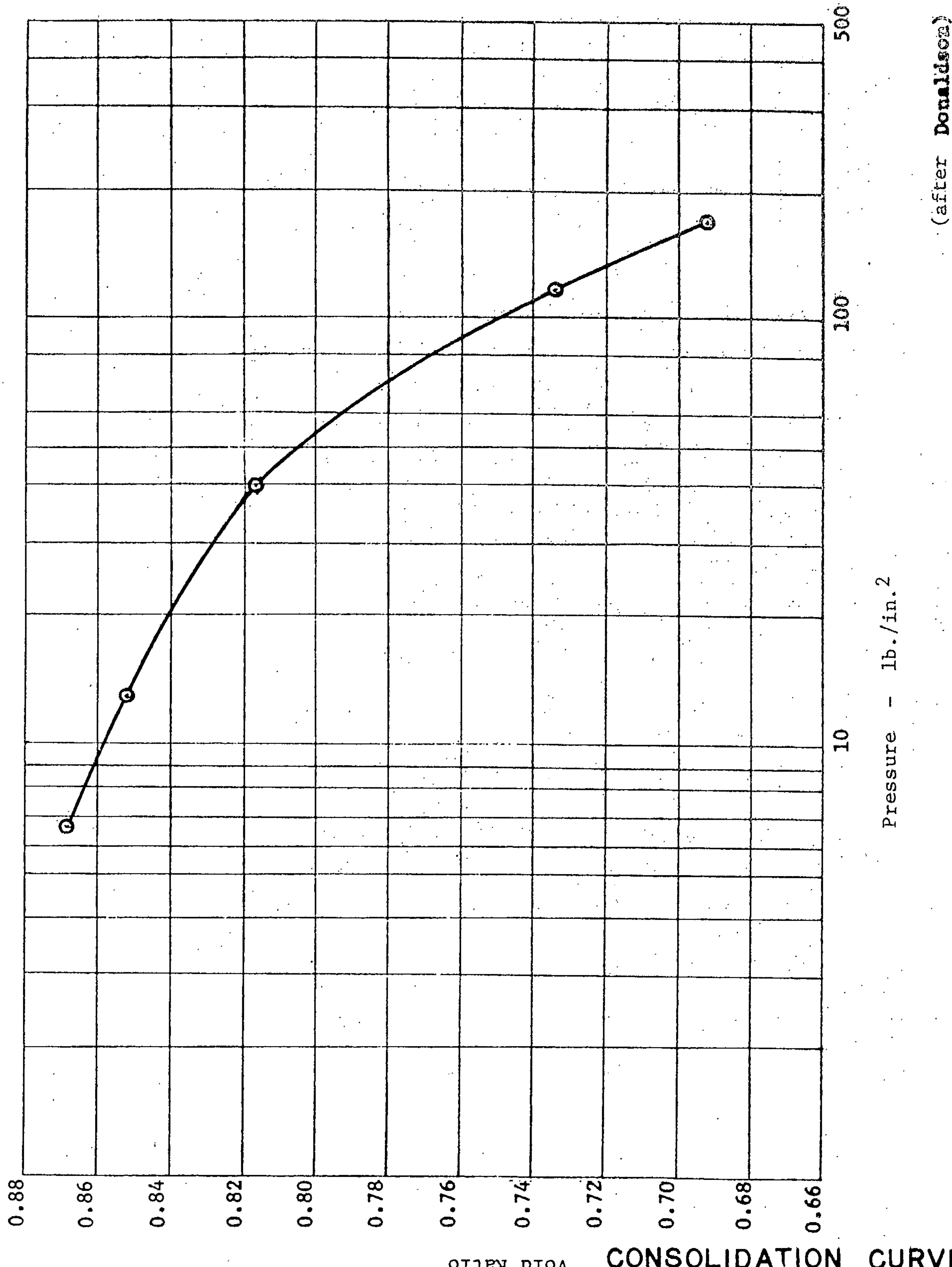
TYPICAL VALUES OF EFFECTIVE COHESION AND ANGLE OF INTERNAL FRICTION FOR SOILS

FIGURE 3-14



TYPICAL PRESSURE - VOID RATIO CURVES FOR SAND AND CLAY

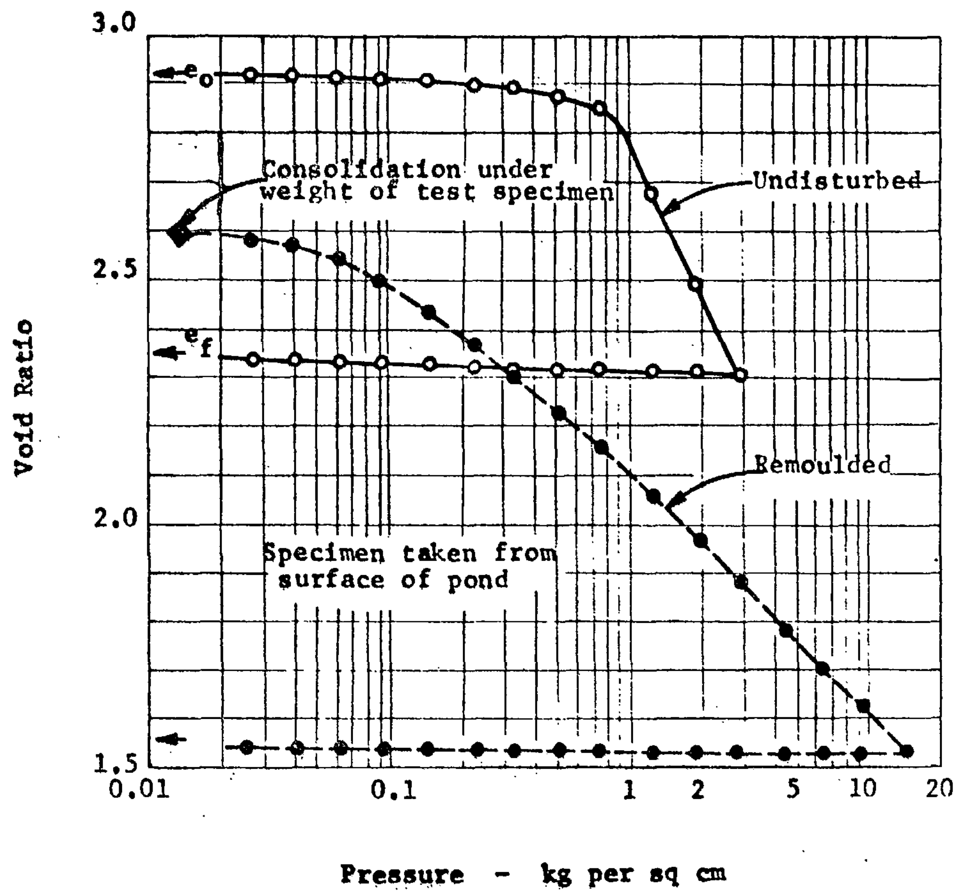
FIGURE 3-15



CONSOLIDATION CURVE
OF EAST GEDULD TAILINGS

FIGURE 3-16

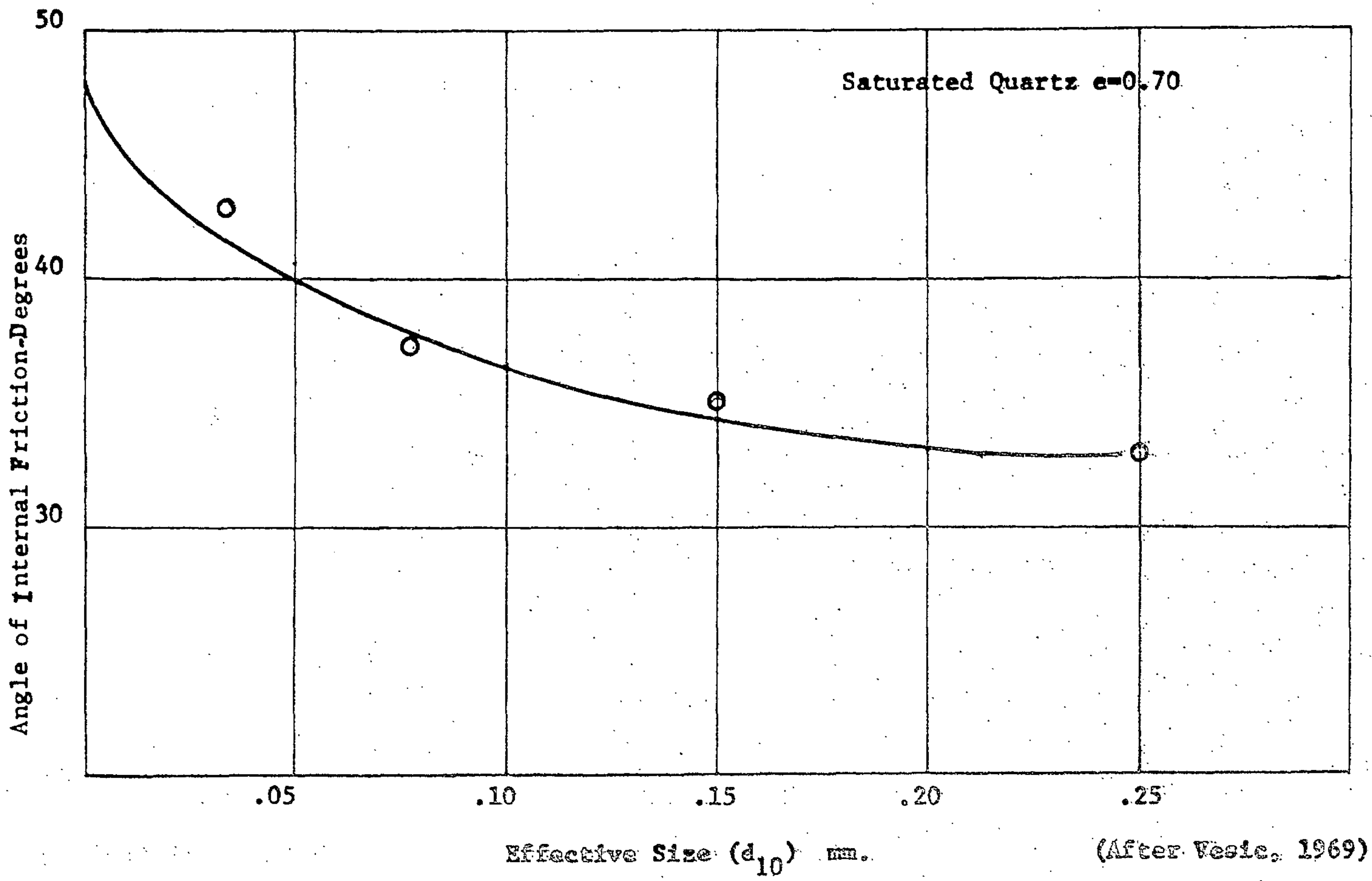
NOTE:
See figure 3-2 for
tailings gradation



(after Casagrande & McIver)

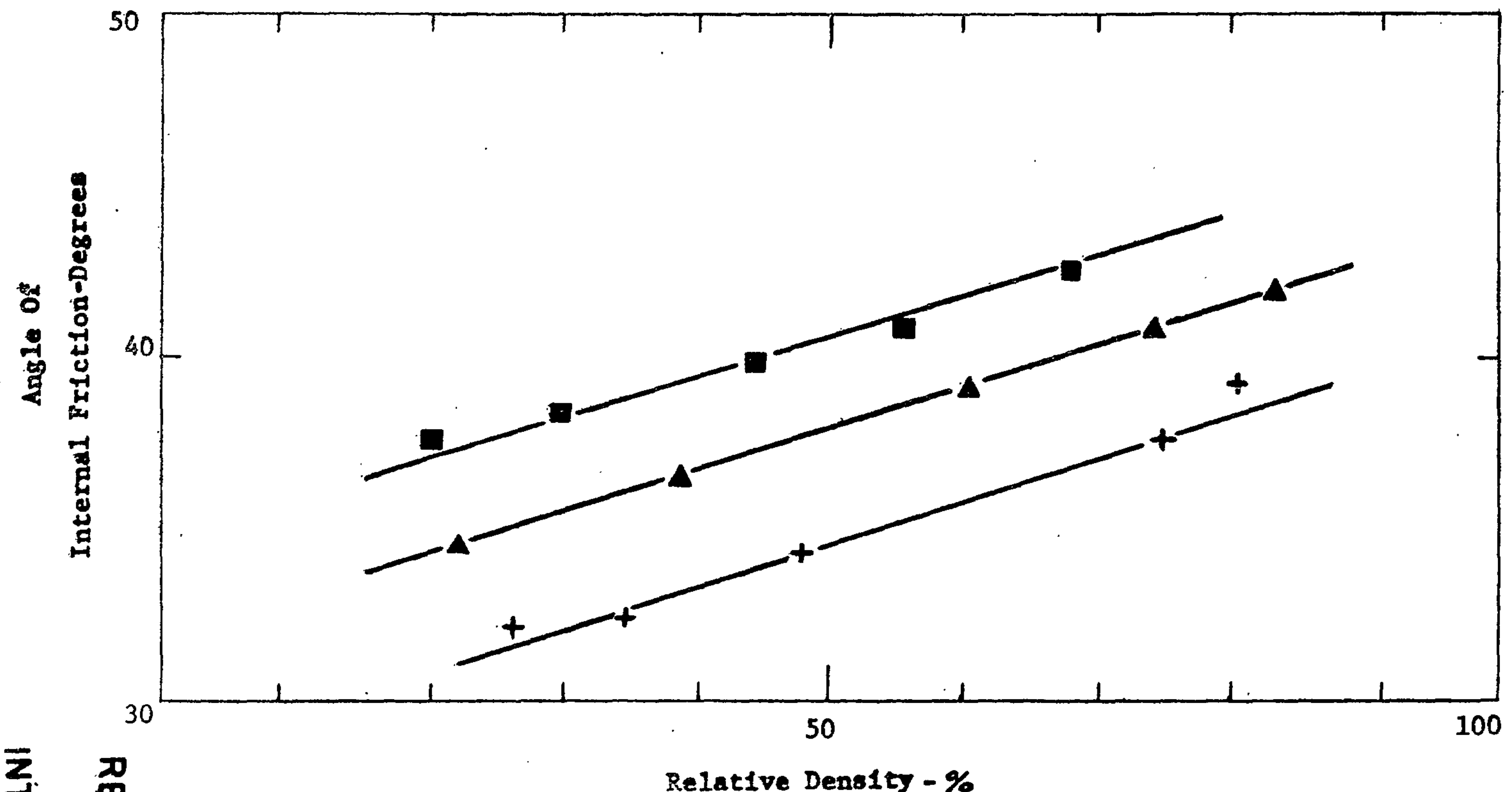
CONSOLIDATION CURVES OF UNDISTURBED AND REMOULDED TAILINGS

FIGURE 3 - 17



EFFECT PARTICLE SIZE ON ANGLE OF
INTERNAL FRICTION OF QUARTZ MATERIALS

FIGURE 3-18



Drained Tests On Saturated Quartz

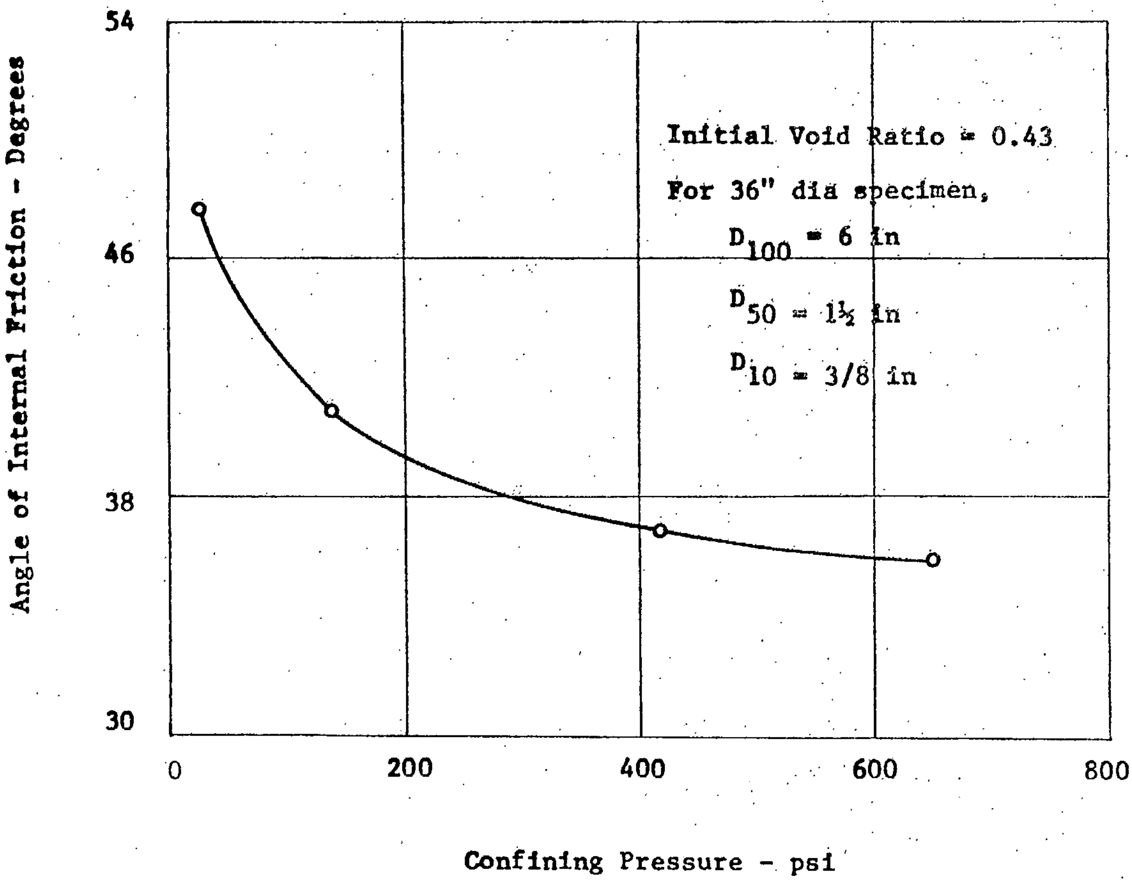
d_{10} = 0.25 mm. (#60); U = 1.25

Symbol	Particle Sphericity	Particle Shape	Min.& Max Dry Density lb.cu.ft.	
■	0.45	Angular	73.6	98.2
▲	0.58	Sub-Angular	86.2	103.8
+	0.67	Sub-Rounded	92.1	108.4

(After Lee)

EFFECT OF RELATIVE DENSITY ON ANGLE OF INTERNAL FRICTION OF QUARTZ MATERIALS

FIGURE 3-19



(After Marachi, Seed & Chan 1969)

ANGLE OF INTERNAL FRICTION
VS. CONFINING PRESSURE
FOR CRUSHED BASALT

FIGURE 3-20

Rock Type	Poissons Ratio	Modulus of Deformation PSI	Compressive Strength PSI
DIORITE	0.26 - 0.29	$10.9 - 15.6 \times 10^6$	$10 - 15 \times 10^3$
FELDSPATHIC GNEISS	0.15 - 0.20	$12.0 - 17.2 \times 10^6$	$6 - 12 \times 10^3$
GRANITE	0.23 - 0.27	$10.6 - 12.5 \times 10^6$	$8.5 - 25 \times 10^3$
LIMESTONE	0.27 - 0.30	$12.6 - 15.6 \times 10^6$	$4.5 - 15 \times 10^3$
QUARTZITE	0.12 - 0.15	$11.9 - 14.0 \times 10^6$	$15 - 20 \times 10^3$
SLATE	0.15 - 0.20	$11.5 - 16.3 \times 10^6$	$3 - 8 \times 10^3$
SANDSTONE	0.15 - 0.35	$4.0 - 10 \times 10^6$	$8 - 20 \times 10^3$

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TYPICAL MECHANICAL PROPERTIES OF ROCKS

FIGURE 3-21

SECTION 4

SITE AND MATERIAL INVESTIGATIONS

CONTENTS	PAGE

FIGURES

SECTION 4

SITE AND MATERIAL INVESTIGATIONS

CLIMATE AND HYDROLOGY

Records

Records of air temperatures, precipitation, wind, solar radiation and evaporation measured at various stations in Canada are published by the Meteorological Branch of the Department of Transport, Ottawa. Various climatic maps and atlases showing average distribution of extremes of temperature, humidity, wind, rainfall, snowfall etc. are published also. Use of some of these data is essential in estimating the evaporation from, and runoff into, mine tailings ponds. A list of the potentially most useful publications is included in the bibliography. They are available from Information Canada, Ottawa or from the Director, Meteorological Branch, 315 Bloor Street West, Toronto 5, except where noted.

Records of measured stream flows are published by the Department of Energy, Mines and Resources, Ottawa.

Evaporation

The rate of evaporation from water surfaces is influenced by solar radiation, air temperature, vapour pressure, wind and, possibly, atmospheric pressure. Since solar radiation is an important factor, evaporation varies with latitude, season, time of day and sky condition.

Evaporation pans have long been used for estimating the evaporation from lakes and reservoirs. Estimating methods based on measurements made in such pans are generally considered to give the most reliable results, the ratio of annual lake-to-pan evaporation being quite consistent, year by year, and not varying excessively from region to region.

The U.S. Weather Bureau Class A pan is in standard use in Canada and the United States. It is of unpainted galvanized iron, 4 feet in diameter by 10 inches deep, and is exposed on a wooden frame in order that air may circulate beneath the pan. It is filled to a depth of 8 inches and water is added each day to maintain this depth.

For relating the evaporation measured in the pan to meteorological factors, the following additional instruments are installed near the evaporation pan:

wet and dry bulb thermometers (for air and precipitation temperatures, vapour pressures, and dew points),

anemometer (for wind movement),

precipitation gauges - one non-recording and one weighing-type recording gauge.

Pan coefficients (the ratio of lake evaporation to pan evaporation) are used in estimating the evaporation from reservoirs. Comparisons of evaporation from Class A pans with measured lake evaporation values indicate that the annual Class A pan coefficient varies from about 0.6 to 0.8 and that a relatively small error would be involved in assuming an average annual coefficient of 0.7. However, these comparisons were with natural lakes and reservoirs, for which annual changes in energy storage in the lake are usually small. Part-year coefficients are more variable because energy storage can be appreciably different at the beginning and end of the period. Changes in heat storage can cause pronounced variations in monthly coefficients. Such changes could be an important factor in the evaporation from a tailings pond storing water at temperatures above the ambient air temperatures.

Methods for estimating the amount of evaporation from Class A pans, and from lakes, are given in Appendix A. They are based on the use of published data on solar radiation, air temperature, dew point and wind movement. The methods provide for adjustments to account for changes in heat storage in the lake.

Runoff

Two main influences of runoff require consideration in the design of mine tailings embankments: that of annual or seasonal runoff volumes on long-term pond water levels, and that of flood runoff volumes and peak flows on the short-term pond levels and on the discharge capacities of diversions or spillways possibly required to pass runoff water around the retaining embankment (assessment of peak runoff flows may be required also for the design of drainage structures for mine waste piles).

Where stream flow data do not exist or when an approximate figure is adequate, simple equations are available in hydraulic handbooks to calculate runoff flow rates, culvert requirements and spillway dimensions.

The most reliable methods for estimating runoff volumes and flows involve:

assessment of the precipitation over the catchment area; this will depend on the climate of the area and, in Canada, could be in the form of rainfall or snowfall; reliable predictions of precipitation to be expected on any particular date, or in any particular season, cannot be made far in advance but reasonably reliable estimates can be made of the probable maximum precipitation to be expected (say in a 24-hour period), and of the probable frequency of occurrence (in years) of precipitation exceeding particular values.

assessment of the runoff losses which will occur in the catchment area; these will include interception by vegetation, evaporation, depression storage and infiltration, all depending on the characteristics of the actual catchment area;

assessment of the rate of runoff, this again being dependant on the actual characteristics of the catchment area.

To be reliable, runoff estimates should be based on precipitation and stream flow records of some kind - the more relevant the data to the particular catchment area being considered, the more reliable will be the results of the computations. In many cases, there will be neither precipitation nor stream flow records available for the particular catchment area and records from adjacent catchments, or generalized data available on published climatic maps, will have to be used. Usually, in estimating runoff volumes, runoff losses will have to be estimated from experience with other catchments; often, the rate of runoff will have to be determined from a synthetic stream-flow hydrograph based on particular characteristics of the catchment area. Methods for making such estimates are described in Appendix B. Their accuracy can be substantially improved by only a few years of precipitation and stream flow records in the actual catchment area being considered. Such records can provide a correlation with long-term records available for other catchments, a check on actual runoff losses and determination of the actual shape of the runoff hydrograph.

For major embankments for which runoff could be a decisive factor in promoting failure, it would be advisable, therefore, to obtain at least a few years of records at the actual embankment site. This would entail the installation of a non-recording and a recording type rain gauge (such as those required with an evaporation measurement station), a snow storage gauge and a recording stream-flow stage measurement gauge. Stream flow measurements would also be required to determine the stage-discharge relationship of the stream gauge.

TOPOGRAPHY

Topographic maps useful in planning the location of mine waste embankments are published by the Department of Energy, Mines and Resources. These maps are generally at scales of 1: 50,000 and 1: 250,000 with contour intervals of 100 feet and 500 feet respectively. Aerial photographs of most parts of Canada are also available from the following services:

National Airphoto Library, Ottawa and Calgary

Surveys & Mapping Branch of the Department of Energy, Mines & Resources;

Provincial coverage, available through

Airphoto Libraries, Departments of Lands in British Columbia and Alberta and

Departmental Libraries in various branches of other Provinces;

Private sources

> generally it is advisable to check the national air-survey companies working in an area; often they have coverage not available through other sources and will make the photos available with the permission of an earlier client.

The small scale maps are most useful in studying the areas surrounding the waste embankment site to assess the possible effects of seepage, flows, runoff and the consequences of failure on nearby developments. The scales and contour intervals required for the design of the waste storage facilities will depend on the size of the area and the vertical relief. Generally, maps having a scale of 1: 5,000 approximately and a contour interval of 20 feet are adequate for preliminary studies. For detailed design, larger scale maps and smaller contour intervals are often required.

Photogrammetric mapping is usually used, as it is both fast and economical. Also, the aerial photographs obtained for the mapping can be very useful in interpreting the general geology of the site and in locating potential sources of construction material.

Topographic mapping of embankment sites should include location of ground surface contours, rock outcrops, forest cover, surface and underground drainage features, buildings, mine workings, services both above ground and buried, access routes and any features which might affect the security of the embankment or might be relevant in determining whether the site will be satisfactory. Data on present and former land use at the site may be relevant, as well as information on land ownership. The location of foundation exploratory holes and test pits, and of any foundation instrumentation, should also be recorded on copies of the topographic maps.

The area mapped should be sufficiently extensive to allow assessment of the areas which may be affected by pollution or by a failure of the embankment. The catchment area of tailing embankments should be mapped sufficiently to enable reliable runoff estimates, and accurate pond storage-elevation curves, to be made.

GEOTECHNICAL

General

Geotechnical investigations at mine waste embankment sites should be directed to determining the:

- nature of the soil deposits (geology, recent history of deposition, erosion and consolidation);
- depths, thickness and composition of each significant soil stratum;
- location of ground water, hydrostatic pressures;
- depth to bedrock and its general composition;
- the presence of old underground mine workings;

geotechnical properties of the soil and rock strata that affect the design of the structure;

availability of suitable construction materials.

This Section describes the planning of the soils investigations and various techniques suitable to obtain this information.

Geology

Geological investigation procedures can include:

research of regional geological reports, maps and aerial photographs,

field reconnaissance and mapping,

geophysical surveys,

exploratory drilling,

borehole photography,

measurement of ground water levels,

ground water pumping tests,

laboratory testing of samples of rocks and soils, including mineralogical analyses.

Generally, the most important aspect to be considered in the design of mine waste embankments will be the surficial geology - the nature of the soils in the foundations. Engineering investigations of these soils are described in the following pages. Significant items affecting behaviour which should receive geological consideration are:

mode of formation of surface deposits (residuum, aeolian, alluvium, colluvium, glacial drift),

evidence of buried channels,

evidence of instability, collapse, solution cavitation,

evidence of recent tectonic disturbance,

evidence of weak formations, bentonite layers, mylonite, shear planes,

seismic history.

The necessary extent of investigations will vary depending on the fill height and the complexity of the foundation. For all waste embankments, sufficient information should be obtained to define and assess the presence of weak zones in the foundation to establish that the foundation is strong enough to withstand the shear stresses, and that seepage can be controlled.

Consideration of the geological history of surficial deposits at, and adjacent to, the embankment site can often indicate the potential importance of these deposits to the design, and whether further investigation of them is required. This should indicate how the deposits were formed and whether they have been subjected to consolidation pressures since their formation. Information on whether soil deposits are residual (produced by the weathering of underlying bedrock), or of erosional, glacial or fluvial origin can indicate the general characteristics to be expected of them. A geological inspection of the site may be sufficient to identify the possibility of buried river or glacial channels under the site. This could have a major effect on the extent of further sub-surface investigations at the site. Any signs of past slides, creep or settlement of soil or rock strata near the site should always be considered. Such signs can provide important forewarning of potentially unstable conditions at the site.

Detailed investigations, such as core drilling of bedrock, should be required for mine waste embankments only in unusual circumstances. Such investigations are sometimes required for water storage dams because of the high stresses introduced into the foundations by the dams, or the danger of seepage and uplift pressures in the foundation rock strata. These considerations are relatively unimportant for mine waste piles not retaining water. However, they could be important for high tailings embankments retaining substantial depths of water in direct contact with the foundations.

The resistance to shearing on surfaces located within bedrock formations is usually substantially higher than that along surfaces located in overlying soils. Only where there are extensive soft seams in the bedrock is it likely to be critical for mine waste embankments. Such soft seams are particularly difficult to locate in bedrock formations; the reliable assessment of their dependable shear strength is even more difficult. While the final assessment of whether they are extensive, and whether they do constitute a danger to embankment stability, will often depend on judgment, their detection is important for waste embankments in the order of 100 feet or more in height.

Subsurface Explorations

The first step in foundation investigations should be an appraisal of any previous underground mining and the general geological character of the site. The second step should involve a few exploratory drill-holes, located after a detailed examination of soil and rock exposures at the site. Such an inspection may indicate that geophysical surveys and echo soundings in lakes would be useful at this early stage of investigation. Often a number of test pits and trenches will be useful in determining the nature of surface soils.

Subsequent steps in the investigation will depend on the size of the embankment and the general character of the soil profile. The importance of the structure or the results of earlier drilling may indicate the need for a more detailed drilling programme. At sites where the subsoil profile is erratic, the objectives should be to define the pattern of dissimilar soils, and the characteristics representative of the various strata encountered. Great use of sounding methods (such as the cone penetrometer) might be appropriate in such circumstances in place of or supported by representative and undisturbed samples.

The depth to which foundation boring should be carried will depend on: the size of the loaded area, the magnitude of loading, and the sub-soil profile.

As a general rule, the foundation borings should be deep enough to determine the sub-soil profile within the depth significantly affected by the structure. However, if bedrock or relatively incompressible soil deposits occur within the significant depth, then the borings need only define the upper boundary of rock or incompressible soil, provided it is established that this soil is underlain by stronger material.

Adverse conditions that require particular attention during sub-surface investigations are:

large boulders overlying bedrock (it can be difficult to differentiate between intact bedrock and large boulders from drill cores),

karstic limestone formations,

expansive clay shales; stiff, fissured, highly plastic clays,

buried, coarse, talus deposits.

A common error is in the definition of the bedrock surface. Often, large boulders occur in the foundation well above the bedrock surface and provide cores that seem to be compatible with bedrock. If the bedrock surface is flat, the type of rock known and the approximate depth to rock known, it may be adequate to limit the coring to 5-10 feet in rock; however, at locations where the bedrock is uneven, and where large boulders may be overlying the rock surface, coring should be 15-20 feet into rock for low embankments and may have to extend 50 feet into rock for high embankments.

Other problem foundations may include: limestones with solution cavities; seams of bentonite within otherwise competent clay or clay shale; gouge zones in otherwise sound rock; and buried talus deposits which are highly pervious.

Types of drilling equipment commonly used for foundation investigations are:

wash boring rig (the simplest, uses casing),

churn drill (the common well drill, versatile, uses casing),

rotary drill (of many types, including diamond drills, uses mud),

hammer drill (different methods, uses casing, air), power auger (all types, continuous flights, hollow stem).

Of the different types of equipment available, all can find application under certain site conditions.

Borrow Materials

The preliminary investigations of an area will usually disclose a number of deposits of material that may be suitable for the construction of an embankment. Further investigation is usually necessary to determine the extent and characteristics of the materials in the deposits. Finally, alternative sources of material can be compared in terms of volume, characteristics, and the cost of supply to the embankment.

Trenching and drilling are usually applied to borrow material investigations following the same pattern of work described for foundations. Sampling and testing should be sufficiently extensive to confirm an adequate quantity of material of the type required. Normally, testing would include determination of in-place moisture content, gradation, optimum moisture content for compaction of the borrow materials and shear strength.

Sampling

Samples are classified in accordance with the sampling procedures used. A common grouping is: wash samples; representative disturbed samples; and undisturbed samples.

Wash samples consist of drill cuttings removed from the hole by the circulating water or air, or by bailing. They have the serious limitation that some mineral constituents have been removed by washing; they are unsuitable for positive identification of the soil, and for laboratory testing, but they often permit a preliminary classification of the soil and approximate determination of stratigraphy.

Representative disturbed samples are usually obtained in thick-walled sample tubes. They are suitable for general classification tests, and identification of the soil, but they are not suitable for strength tests. They are commonly used to indicate water content, grain size, specific gravity and Atterberg limits.

Undisturbed samples require sophisticated sampling equipment and drilling techniques. They require great care to preserve the sample in its natural condition, should be sealed to prevent loss of water during handling and storage and should be protected from freezing. They should be protected from disturbance during handling and are usually shipped in tubes held in special shock-proof containers. Undisturbed samples are suitable for all laboratory tests and from them many characteristics of the insitu soil deposits can be determined.

The techniques of sampling from bore holes depend on the nature of the ground and the degree of sample disturbance that is acceptable. Those appropriate to various conditions are tabulated on Figure 4-1. The frequency of sampling is dependent on the variations found in the materials being penetrated, but, usually, sampling at 5-foot intervals is satisfactory. On occasion, sampling at more frequent intervals, or continuous sampling, is necessary to examine weaker zones or zones of relatively high permeability.

In cohesive soils, uncased and dry bore holes may sometimes be used to shallow depths, but stabilization with casing or drilling mud is usually required when undisturbed samples are to be obtained from deep bore holes.

Sampling in sand and gravel usually requires casing or drilling mud. The casing is filled with water to prevent caving and heaving of the material at the bottom of the hole when drilling below the water table. A representative sample of sand may be obtained with split-spoon samplers, piston samplers and some types of thin-walled Shelby tube samplers. Care and experience are necessary to obtain good sampling in these materials.

In gravelly soils, thick-walled drive samplers or barrel augers may be necessary.

Hammer drills, using air and percussion, are frequently used in exploring gravels and sands. Samples obtained from the discharge of such machines are valuable as indicators of the material at the bottom of the hole but cannot be considered truly representative, because of degradation and segregation of particles in transport up the casing.

Laboratory Testing

1. Grain Size Distribution

The laboratory procedure for determining the grain size distribution of soils is described in ASTM D422 (American Society for Testing and Materials Test Method). In this procedure, a sample of soil is divided into a coarse fraction and a fine fraction; the grain size distribution of material coarser than 200 mesh is then determined mechanically, using standard sieves; the grain size distribution of the material finer than 200 mesh is tested by hydrometer.

From a series of grain size distribution tests of a given soil, an envelope of distribution curves may be developed to form a basis for assessment of some of the soil's other characteristics.

2. Specific Gravity and Void Ratio

The specific gravity of soil particles is determined by laboratory test ASTM D854. Knowing the dry density, and the specific gravity of the soil particles, the void ratio can be determined from the following relationship:

$$\text{Dry Density } (\gamma d) = \gamma w \frac{(Gs)}{(1+e)},$$

where:

γw = unit weight of water,
Gs = specific gravity of the soil particles,
e = void ratio (the ratio of the volume of voids to the volume of solids in a soil).

3. Relative Density

The relative density of cohesionless soils is obtained by the determination of the void ratio of the soil in the loosest state, in the densest state and insitu. The relative density is given by the relationship:

$$Dr = \frac{e\ max - e}{e\ max - e\ min},$$

where:

Dr = relative density,
e max = void ratio in the loosest state,
e = void ratio insitu,
e min = void ratio in the densest state.

4. Plasticity

The Atterberg limit tests (Lambe and Whitman, 1969) are used to determine the plasticity of soils. The liquid limit defines the boundary between the liquid and plastic states and the plastic limit the boundary between the plastic and solid states. The difference between these two water content values is the range of water content over which the soil remains plastic, and is called the plasticity index. This parameter is used in classifying soils for estimating other physical properties which have been correlated empirically with the plasticity index (Casagrande System).

5. Compaction

The moisture-density relationship of a soil is obtained by the standard procedure ASTM D1557. A definite relationship exists between the water content of a soil at the time of placement and the amount of compactive effort required to achieve a given density. If the soil is too wet or too dry, the given density may be difficult or impossible to achieve by normal compaction methods. The objective of the laboratory procedure is to determine the optimum water content for the specified compaction.

6. Shear Strength

The shear strength of a soil is usually measured by triaxial compression tests or direct shear tests. Triaxial tests can define the shear strength under both drained and undrained conditions; conventional shear box tests can define only the shear strength under drained conditions.

Determination of meaningful shear strength parameters of soils, for use in design analyses, requires extensive knowledge and experience. Soil shear strength parameters are affected by many test factors including such items as: rate of loading, method of loading, principal stress ratios, rate of specimen strain, total specimen strain, degree of saturation, drainage conditions, etc. Moreover, in selecting the shear strength parameters that are to be used for specific analyses, the designer must estimate probable strains and rates of pore pressure dissipation in the field, and must decide whether to use "peak" or "residual" shear strength values. Soft soils will generally require more testing to define their shear strength characteristics adequately than will firm soils.

The data obtained from shear strength tests are normally presented in terms of effective stresses. During testing, both total stresses and pore water pressures are measured. The effective stresses are determined by subtracting pore water pressure from total stress. Effective stress plots, for several soil specimens at failure, are conventionally presented in the form of Mohr circles, as shown on Figure 4-2. The effective cohesion (c') and the effective angle of internal friction (ϕ') are defined by the position and slope of the Mohr failure envelope.

7. Consolidation Tests

Consolidation tests are made to determine the compressibility of a soil and the rate at which it will consolidate when loaded. Consolidation test procedures are detailed in ASTM D2435. The two soil properties usually obtained from a consolidation test are:

(C_c) the compression index, which indicates the compressibility of a soil;

(c_v) the coefficient of consolidation, which indicates the rate of compression under load.

The data obtained from a laboratory consolidation test are normally presented in the form of a plot of pressure versus void ratio. This curve is used to estimate the probable stress history of the soil, the item of particular interest being whether or not the soil was preloaded in its past geological history.

8. Permeability

Permeability test specimens may be either undisturbed or representative disturbed samples, depending on the purpose for which they are to be tested. Samples of foundation materials should be undisturbed, whereas samples of borrow material for embankments should be remoulded at a density comparable to that specified for construction. The method of permeability testing used will depend on the permeability of the soil to be tested. (Reference Figure 3-10). The constant-head permeameter is used to determine the permeability of granular soils, whereas the falling-head type is more suitable for soils of low permeability. For soils of very low permeability, consolidation test data may be used to estimate permeability. (Lambe and Whiteman, 1969 and Laboratory Soils Testing, EM1110-2-1906, U.S. Army Corps of Engineers). The permeability of a soil can also be estimated mathematically from its grain size distribution using formulae developed by Hazen (Terzaghi and Peck, 1967).

Field Testing

1. Density

Approximate estimates of the relative density of cohesionless soils may be obtained in the field from the "standard penetration test". The test is conducted by driving a split-spoon sampler having dimension of 1.375 inches ID, 2 inches OD, into the soil, using a 140 lb. weight dropping a distance of 30 inches. The number of blows required to drive the sample spoon 12 inches is called the standard penetration resistance (N value) of the soil. Empirical relationships have been developed (Terzaghi and Peck, 1967) which allow correlation of penetration resistance to relative density. This relationship is illustrated on Figure 4-3. Soundings with cone penetrometers can also be used for approximate determination of relative density.

Approximate correlations have also been made between standard penetration test blow count (N) and the consistency of a clay. This relationship, which is shown on Figure 4-3, is approximate, and should be used only as a guide.

Several methods are available for determining the in-place unit weight of soils in pits and excavations. Two commonly used procedures are: the sand cone method, ASTM D1556, and the rubber balloon method, ASTM D2167. In recent years, nuclear methods for determining field densities and water contents, have come into common usage. (Csathy, T; 1962; and ASTM, Sp. Publication No. 293, 1960.)

2. Shear Strength

Several methods of conducting vane shear and static penetration tests in boreholes are used to evaluate insitu shear strengths. (Hvorselev, M.J., 1949; British Geotechnical Society, London, 1970).

Occasionally, it may be desirable to conduct a large scale standard shear box test in the field. An insitu sample of soil or rock is enclosed within a box, and vertical and horizontal loads applied. The load-displacement curve is then obtained and analyzed, as in the laboratory test.

3. Permeability

Two methods are commonly used for determining the permeability of soils in the field:

Infiltration Test - water is introduced into a drill hole, well or test pit, and the rate of seepage observed under a fixed or variable head. A variation of this test is to bail the water out of the hole and measure the rate of inflow,

pumping-Out Test - water is pumped from a borehole or a test pit at a constant rate and the drawdown of the water table observed in observation wells placed on radial lines at various distances from the pump.

Both methods have the advantage of testing the soil in-place, so that the effects of natural structure, stratification, orientation of the grains, and other natural properties, are included in the tests. The pumping-out test method is relatively expensive to perform and computational methods are available to calculate soil permeabilities. (Cedergren, H;, 1967). Figure 4-4, illustrates a method of computing average permeability from the infiltration test data.

Test Hole Logs

The logs of sub-surface exploratory holes and sampling form the permanent record used in the analysis of an embankment site. Completeness and accuracy of observations are, therefore, critical to correct analysis. Logs should contain not only the data required for determination of the soil profile, and the location of the samples obtained, but also any observations of detail which will contribute to an appreciation of the condition of the samples and of the physical properties of the soil insitu. A trained observer should be present continuously during drilling to supervise the sampling, prepare the logs and adapt the exploratory programme to the conditions encountered.

Typical standard forms for logging samples are shown on Figures 4-5 and 4-6. A special effort should be made by different observers to standardize on their interpretation of various materials encountered and tested. Logging of samples should be kept up-to-date and the logs prepared in final form as soon as possible after the hole is completed. This subject has been treated in considerable detail by Hvorslev (1949).

MINE WASTES

General

The investigation and testing of mine wastes is generally similar to that of soils. Factors requiring investigation that are particularly critical to the stability of waste embankments are:

- degradation of the waste materials resulting in a loss of shear strength,
- water levels in the embankments,
- sealing of embankment exteriors by weathering,
- the effects of drainage water entering the embankments,
- incompetent materials at the bases of the embankments.

Investigations required can include:

- drilling and sampling as for soils,
- research on performance of similar wastes in existing embankments,
- accelerated degradation tests of waste materials,
- instrumentation of embankments to monitor continuing behaviour (piezometers to measure water pressures; deformation measurements; and photographs at regular time intervals to record changes in appearance).

Notes on the investigation of degrading wastes and of existing embankments follow. Instrumentation is described in Section 6.

Degrading Wastes

The strength properties of some waste materials change after the material has been placed in a pile. In some cases, experience with similar materials will indicate that particular wastes are likely to become unstable. However, in other cases, an apparently sound material may break down under the attack of some particular chemical or effect of exposure.

Materials that commonly degrade are shales, siltstones and mudstones. Often, when exposed to freezing, thawing, wetting and drying, or to the relief of stress by excavation, these materials will gradually lose their strength. If this loss of strength is substantial, a waste pile designed on the basis of the strength of the sound material may become unstable in future years. Therefore, piles of wastes that may be susceptible to degradation should be designed to be stable at the lowest strength that the material might achieve.

The final strength characteristics of degrading materials can be difficult to determine. Inspection in the field may reveal degraded materials which can be sampled and tested. Alternatively, accelerated tests can be performed in the laboratory using rapid cycles of freezing and thawing, wetting and drying, or exposure to acids or other materials that might attack and influence the waste.

Accelerated degradation tests are not likely to indicate the final strength of the material reliably, but they could indicate the likelihood and general mechanism of breakdown. If the tests indicate that the waste materials could be attacked and their strength altered significantly, confining structures may have to be provided rather than relying on the material itself to remain stable. In some cases, the strength of the material may be maintained if the waste pile is isolated from the agents, such as water and air, that cause loss of strength.

In summary, the best procedure to be followed in investigating materials that may be suspected of degrading in storage is to:

investigate the waste materials on-site and the conditions to which they might be exposed in storage;

perform accelerated degradation tests to duplicate as closely as possible the conditions that are expected in the waste pile; the results of these tests may indicate the approximate lower-level of strength that might be developed, or methods to prevent the degradation of the waste;

perform insitu tests where the actual agents of attack can operate and the results may be observed; methods of accelerating the rate of attack or the rate of strength change could be introduced;
detailed examination of the pile may reveal an exposure in which accelerated degradation is already evident.

Existing Waste Embankments

It is sometimes necessary to investigate a tailings embankment or waste pile in the absence of any history of earlier investigation, design or construction procedures. The investigation may be necessary to assess the present state of stability of the embankment, or to evaluate the pile for the disposal of additional material.

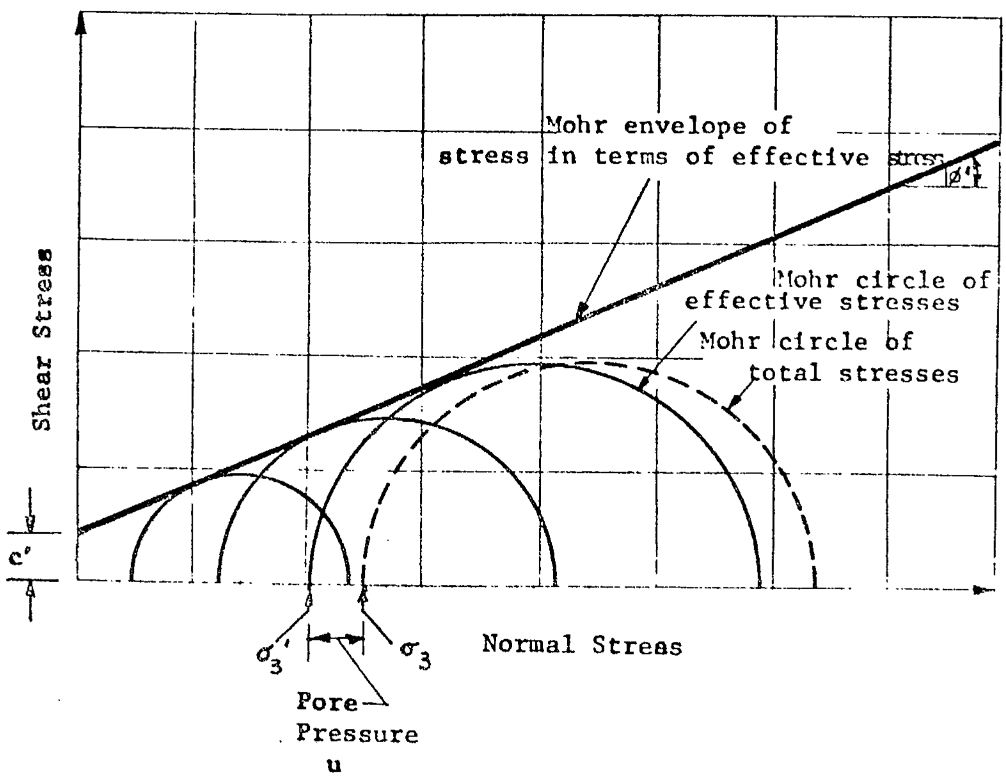
The investigation of existing embankments is similar to that described for embankment foundations, except that additional information may be obtained from persons who have worked on the site in earlier years. Every effort should be made to obtain information available from this source, particularly information on the general methods of operation, equipment used, the initial state of the foundation and the distribution of different materials in the embankments. Exploratory equipment and drill sites can then be chosen to provide the necessary additional information with the smallest possible outlay of time and money. In general, drilling and sampling should extend well into the foundation beneath the embankment; drill holes should be instrumented for continuing observation of water levels. Where waste embankments are suspected of incipient failure, installation of movement measuring devices may be necessary.

SAMPLING METHODS FOR VARIOUS SOIL TYPES

TYPE OF SOIL	METHODS OF BORING	RECONNAISSANCE EXPLORATIONS REPRESENTATIVE SAMPLES	DETAILED EXPLORATIONS SMALL UNDISTURBED SAMPLES	SPECIAL EXPLORATION LARGE UNDISTURBED SAMPLES	SURFACE SAMPLING UNDISTURBED SAMPLES CONTROL SAMPLES
	Methods shown in parentheses are rarely used	Sampling in Borings of each Significant Stratum but 5 ft maximum spacing	Sampling in Borings Continuous Samples Diameter 2 to 3 In.	Sampling in Borings of Controlling Strata Diameter 4 to 6 In.	Sampling Close to Surface Accessible Explorations Earth Structures
Common Cohesive and Plastic Soils	Displacement, Wash, Auger Continuous Sampling (Percussion, Rotary)	Augers 1 to 2 In. Piston or Open Drive Sampler	Thin-Wall Drive Sampler Open or with Stationary or Free Piston	Thin-Wall or Composite Drive Sampler with Free or Stationary Piston (Cut. Wire, Vacuum Relief)	2 to 6 In. Thin-Wall, Open Drive or Free Piston Sampler 4 to 8 In. Adv. Trim. Sample 8 to 12 In. Sq. Box Sample
Slightly Cohesive and Brittle Soils including Silt, Loose Sand above Ground Water	As above but keep boring dry for undisturbed sampling above ground water	As above	As above	Thin-Wall Drive Sampler Free or Stationary Piston (Vacuum Relief)	As above but advance trimming or box sampling preferable
Very Soft and Sticky Soils	Displacement, Wash Bailers, Sandpumps Continuous Sampling (Auger, Rotary)	Slit or Cup Sampler 1 to 2 In. Piston or Open Drive Sampler (Core Retainers)	Thin-Wall Drive Sampler with Stationary Piston	Thin-Wall or Composite Drive Sampler with Stationary Piston Vacuum relief required	2 to 6 In. Thin-Wall, Open Drive or Sta. Piston Sampler Danger of soil movements and disturbance before sampling
Saturated Silt and Loose Sand	Displacement, Wash Bailers, Sandpumps Continuous Sampling (Rotary)	As above Release stat. piston before any intentional overdriving	Thin-Wall Drive Sampler Free or Stationary Piston 2 In. Diameter	Thin-Wall Drive Sampler Free or Stationary Piston Vacuum relief or freezing bottom of sample required	2 to 6 In. Thin-Wall Sampler Open or Free or Sta. Piston 4 to 8 In. Adv. Trim. Sample Depress ground water level
Compact or Stiff and Brittle Soils including Dense Sand, Partially Dried Soils	Wash, Augers Percussion, Rotary Continuous Sampling	Augers and 1 to 2 In. Thick-Wall Piston or Open Drive Sampler	Medium-Wall Open Drive or Piston Sampler. Hammering may be required (Partial Disturbance)	Core Boring may be better than Drive Sampling but danger of contamination in partially dry soils	4 to 8 In. Adv. Trim. Sample 8 to 12 In. Sq. Box or Block Samples. Auger Core Boring. Bag Sample and Field Density
Hard, Highly Compacted or Partially Cemented Soils, no Gravel or Stones	Percussion, Rotary Continuous Sampling	Thick-Wall Open Drive Sampler. Core Boring	Thick-Wall Open Drive or Piston Sampler. Core Boring. Samples small diam. often partially disturbed	Core Boring preferable to Drive Sampling. Danger of fluid contamination in permeable soils	8 to 12 In. Sq. Box Samples or Irregular Block Samples
Coarse Gravelly and Stony Soils including Compact and Coarse Glacial Till	Percussion, Barrel Auger Loosen by Explosives Thick-Wall Drive Sampler	Barrel Auger Thick-Wall Drive Sampler (Core Retainer)	Not practicable	Advance Freezing then Core Boring	8 to 12 In. Sq. Box Samples Bag Sample and Field Density
Gaseous or Expanding Soils (Organic Soft Clay, Silt, Sand)	According to soil but keep boring filled with water or drilling fluid	As above according to basic soil type	Thin-Wall Sampler with Free or Stationary Piston. Force closed sampler through expanded soil. Determine original sample length and volume. Sealing to prevent expansion.		Thin-Wall Drive Sampler Open or Piston Type. Danger expansion of soil before sampling
Gradual or Sudden Changes in Soil Properties within a Single Drive	As above according to basic soil type	As above according to basic soil type	Safe length of sample increased when progressing from weak to firm strata and vice versa. Thin soft strata often disturbed. Withdraw after passing firm stratum		As above according to soil type. When possible separate coarse and fine-grained soil
Soils with Secondary Structure	As above according to basic soil type	As above according to basic soil type	As above according to basic soil type, but the results of strength, consolidation, and permeability tests do not always represent properties of undisturbed deposit		Large Box or Block Samples. Large test specimens. Detail field tests and observations

After Hvorslev, 1949

FIGURE 4-1



c' = Effective Cohesion

ϕ' = Effective Angle Of Shearing Resistance

σ_3 = Total Normal Stress

u = Pore Pressure

σ_3' = Effective Normal Stress

Shear Strength = $S = c' + (\sigma - u) \tan \phi'$

(Mohr Diagram Showing Envelope Of Soil Strength In Terms Of Effective Stresses, And Relationship Between Effective And Total Stresses.)

TYPICAL MOHR DIAGRAM

FIGURE 4-2

Approximate Relationship Between

Standard Penetration

And

Relative Density And Consistency

For Cohesionless Soils

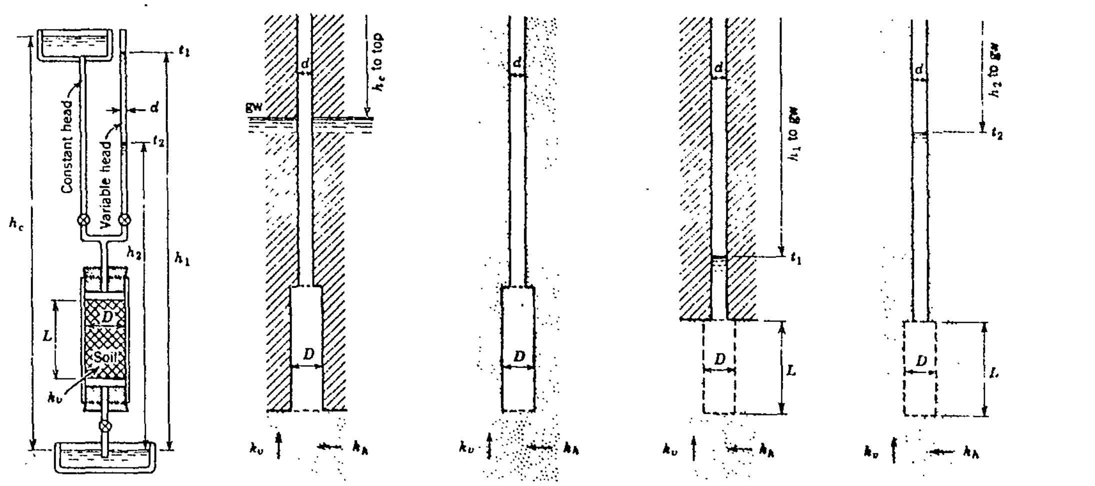
No Of Blows(N)	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
Over 50	Very Dense

For Cohesive Soils

Consistency	Very Soft	Soft	Firm	Stiff	Very Stiff	Hard
No. of Blows (N)	2	2-4	4-8	8-15	15-30	30
Unconfined Compressive Strength (tons/ft.^{2})	Less than 0.25	0.25 to 0.50	0.5 to 1.0	1.0 to 2.0	2.0 to 4.0	over 4.0

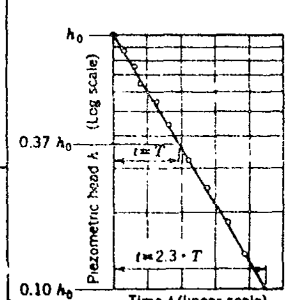
STANDARD PENETRATION VS.
RELATIVE DENSITY
AND CONSISTENCY

FIGURE 4-3



Laboratory permeameter (consolidometer) *(a)* | Flush bottom at impervious boundary *(b)* | Flush bottom in uniform soil *(c)* | Well point-filter at impervious boundary *(d)* | Well point-filter in uniform soil *(e)*

DEFINITION SKETCH

Case	Constant head	Variable head	Basic time lag	Notation
a	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 \cdot h_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ $k_v = \frac{L}{t_2 - t_1} \ln \frac{h_1}{h_2}$ for $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ for $d = D$	D = diam. intake, sample, cm d = diameter, standpipe, cm L = length, intake, sample, cm h_c = constant piez. head, cm h_1 = piez. head for $t = t_1$, cm h_2 = piez. head for $t = t_2$, cm
b	$k_m = \frac{q}{2 \cdot D \cdot h_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln \frac{h_1}{h^2}$ $k_m = \frac{\pi \cdot D}{8 \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ for $d = D$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ for $d = D$	q = flow of water, cm³/sec t = time, sec T = basic time lag, sec k_v = vert. perm. ground, cm/sec k_v' = vert. perm. casing, cm/sec
c	$k_m = \frac{q}{2.75 \cdot D \cdot h_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ $k_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ for $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ for $d = D$	k_h = hors. perm. ground, cm/sec k_m = mean coeff. perm. cm/sec m = transformation ratio $k_m = \sqrt{k_h \cdot k_v}$ $m = \sqrt{k_h / k_v}$ ln = $\log_e$ = 2.3 $\log_{10}$
d	$k_h = \frac{q \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ for $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{2mL}{D} > 4$	
e	$k_h = \frac{q \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{h_1}{h_2}$ for $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{mL}{D} > 4$	Determination basic time lag T

Assumptions: Soil at intake, infinite depth and directional isotropy (k_v and k_h constant); no disturbance, segregation, swelling or consolidation of soil; no sedimentation or leakage; no air or gas in soil, well point, or pipe; hydraulic losses in pipes, well point or filter negligible. (After Hvorslev, U.S. Corps of Engineers, W.E.S.)

FORMULAE FOR DETERMINATION OF PERMEABILITY

FIGURE 4-4

LOCATION Mill River Bridge - West Abutment
SURFACE ELEV. 423.4 FIXED DATUM M.S.L.
BORING 6" Casing - Franks Drill Rig No. 11
SAMPLERS 2" Split Barrel - 4¾" Piston Sampler
SAMPLES J=Jar, P=Paraffined, L=4.68" I.D. Liner
DATE START 5-20-1946 DATE FINISH 5-24-1946

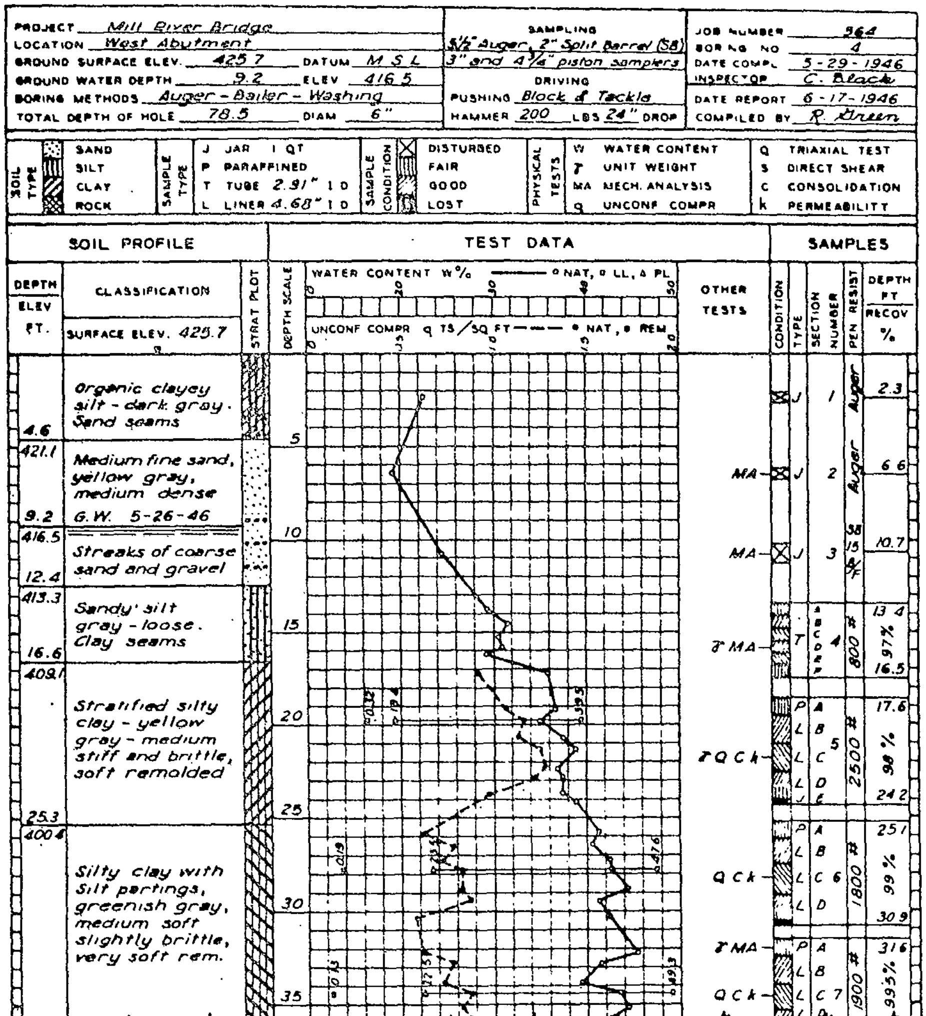
PROJECT NO 564
BORING NO. 3
SHEET 1 OF 4
FOREMAN S. White
INSPECTOR C. Black

SOIL PROFILE		DEPTH		SAMPLES				BORING AND SAMPLING NOTES
DESCRIPTION	DEPTH	SCALE	TYPE	NO.	DEPTH	REC.	FORCE	
	G.S.	0						
Topsoil								5-20-1946 - cloudy-mild
Org. clayey silt	1.3							
								Hole to 4.5 - auger
Clayey silt								
medium - dark			J	1	2.5			Casing to 4.6
gray - seams of								
brown sand								Sample 1 with auger
	3.8							
								Hole to 9.0 - auger
Gray sandy silt		5	J	2	5.0			
	5.7							Sample 2 with auger
								Sample 3 with auger
Medium - fine sand			J	3	7.0			
Yellow gray								Casing to 9.2
Medium dense				G.W.	8.3	5-20	1946	Hole to 10.4 - bailer
Streaks of coarse								
sand, little gravel								Water 9.9 8.7 8.5
								Time 11:45 12:05 12:25
	10.2	10						River Stage 15.6
								River Elev. 411.7
Gray silty sand			J	4	10.4	1.2	15	
with a little clay					11.6	1.1	Bl/Ft	Sample 4 - 2" split barrel
	11.8							Hammer 200#, Drop 24"
Gray clayey silt								Casing to 12.0
sand seams			P	5-A	12.82			Casing filled with water
								Cleanout to 12.7 with
	14.3		L	5-B				Calyx jet auger
		15				5.43	400	
Silty clay						5.39	to	Sample 5
Yellow gray			L	5-C		99.2	2400	4¾" Piston sampler to
Medium stiff						%	lbs.	18.25 with block and
								tackle in 7 seconds
	17.2		L	5-D				0.22' bot. sample lost
				Lost	18.25			
Silty clay								Casing to 17.9
Olive gray			P	6-A	18.65			Cleanout to 18.5
Medium soft								with Calyx jet auger
		20	L	6-B				

(After Hvorslev, 1949)

TYPICAL FIELD LOGGING SAMPLES

FIGURE 4-5



(After Hvorslev, 1949)

TYPICAL LABORATORY LOGGING OF SAMPLES

FIGURE 4-6

SECTION 5

DESIGN

CONTENTS — PAGE

FIGURES

FIGURES - Cont'd

SECTION 5

DESIGN

GENERAL

Methods of analysis commonly used in the design of mine waste embankments are described in this Section. Those for tailings embankments are described first, followed by those for mine waste piles. Since methods of stability analysis are common for both tailings embankments and waste piles, they are described last. Whereas methods of reinforcing embankments have been developed in the construction industry, they are not treated in this volume; some experimental development work could lead to worthwhile payoffs.

TAILINGS EMBANKMENTS

Functions

The primary function of tailings embankments is to store tailings solids and to keep them confined within a preselected area. A common ancillary function is to temporarily store water from the tailings slurry for clarification prior to its release or reclaim from the tailings pond. These functions have been explained in more detail in Section 2. Notes on determination of the minimum pond size necessary for effective water clarification are given below. The importance of keeping the pond distant from the embankment, which implies keeping it as small as possible, has already been explained.

Pond Size

The area of the tailings pond required for adequate clarification of the water prior to reclaim or discharge to local streams is difficult to determine by theoretical means. Although the settlement velocities of various types and grain sizes of solids can be determined theoretically and experimentally, many factors influence the effectiveness of the pool. Basically, the problem is to provide sufficient retention time to permit the very fine fractions to settle before they reach the point of decantation. Factors affecting the settling time are the size of grind, the tendency to slime (clay type minerals), the pH of the water, wave action and depth of water.

The size of grind required for liberation of the metal is usually sufficiently fine to produce particle sizes whose settling rate is governed by Stokes' Law, with a high percentage under 200 mesh. Particles in the range of 300 mesh or 50 microns with a settlement rate of 0.05 inches per second can be affected by wind action, but will settle in a reasonable time. The major problem is caused by the small percentage of particles in the range of 2 microns or less which produce turbidity. These particles have settlement rates less than 0.01 inch per second in still water and, under conditions prevalent in most tailings ponds, require some days to settle below the turbulence caused by wave action.

Various rules for clarification have been accepted as a result of observation in existing ponds. Among these are:

the pool should be sized to allow 5 days retention time,

the area of the pool should be sized to provide 10 acres to 25 acres of pond area for each 1,000 tons of tailings solids delivered per day. An average of 15 acres per 1,000 tons is usually considered adequate, unless some unusual conditions are present.

Construction Materials

Fill materials for construction of tailings embankments may include:

soil or rock excavated from borrow areas located within economic haul distance of the embankment,

overburden or waste rock materials available from open pit or underground mining operations,

solid waste materials reclaimed from the tailings disposal operation.

The materials used in the construction of the embankment should be those that permit its construction at least cost commensurate with adequate stability. For any given height of embankment, the steepest permissible fill slopes, and hence the volume of fill comprising the embankment, will be governed by the shear strength of the embankment fill, and/or by the maximum permissible shearing stresses within the foundations beneath the structure. At locations where the sub-surface foundation soils are incompressible and exhibit high shear strength characteristics, the slopes of the upstream and downstream embankment surfaces will be governed by the shear strength and drainage characteristics of the embankment fill, and by the height of the embankment. Where the embankment is constructed over weak foundations, the embankment slopes will be governed by the shear strength characteristics of the foundation materials.

Regardless of the materials incorporated within the embankment section, control is required to assure that the quality, gradation, and density of the fill materials are in accordance with the design requirements. Alternatively, the designer must be aware of any differences so that modifications can be made to the design if required to accommodate the fill materials available.

Construction Methods and Embankment Types

The design of tailings embankments is closely related to the methods used in their construction, particularly when the primary embankment construction material is sand obtained from the tailings slurry. In this case the embankment construction is basically a part of the tailings disposal operation; the embankment design may be influenced strongly by the need to arrive at the most economical overall system.

Embankment construction methods are described in Section 6. An overriding factor will be the necessity to keep the embankment crest above the pond surface. This can affect the entire design and the basic construction methods. Where tailings sand is the principal embankment fill material, one of three basic placement procedures can be used. These, and the types of embankment cross-section resulting from their use, are shown on Figure 5-1. When other borrow or dry-waste materials are incorporated in the embankment, many alternative types of embankment are possible. Some of these are illustrated on Figures 5-2 to 5-4. Notes on their applicability are given on the figures.

Embankment Design Procedure

The design of a tailings embankment is a procedure of successive trials and refinements of the design concepts. Briefly, the general steps required in developing the final embankment cross-section are:

from the topographic data within the prospective storage area, calculate storage volumes and plot the results in the form of a storage volume - elevation curve;

estimate the total volume required for storage of the mine tailings; from the storage-elevation curve, estimate the final crest elevation and ultimate height of the required tailings embankment;

select a trial embankment section that incorporates the most economic and readily available fill materials;

make a stability analysis for the trial section to determine its factor of safety; the stability analysis should take into account the shear strength parameters and the densities of the materials comprising both the foundation and the embankment, as well as the pore water pressures within the embankment and the foundation;

pore water pressures resulting from steady seepage within the embankment, and within pervious foundations, can be estimated from flow nets; if compressible foundation strata are located beneath the embankment, foundation pore pressures estimated on the basis of consolidation-time theory should be taken into account in the analysis, and should be checked by field measurements during and after construction;

if the stability analysis for the trial embankment cross-section indicates that the section is unsafe, or that the factor of safety is unduly high, the section should be adjusted and the stability analysis repeated;

stability analyses should be repeated until a section has been found that has the required factor of safety;

prepare detailed construction drawings and specifications for foundation treatment, fill placement and waste disposal, if these are an integral feature of the design.

Seepage Analysis and Control

1. Control of Seepage Pressures

Control of seepage pressures and erosion caused by seepage (rather than the rate of seepage) are the important considerations in considering seepage effects on the stability of tailings embankments. Some hydraulic structures that pass seepage at a rate of several hundred gallons per minute have operated successfully for decades. Other structures where the rate of leakage has been only a few gallons per minute have become unstable and have failed. Regardless of the rate of leakage, seepage pressures must be controlled so that quick conditions and piping do not develop.

Water supply and pollution control requirements may necessitate the use of special design features such as impervious cores, cut-offs, blankets, etc., to maintain the rate of leakage from the tailings pond within tolerable limits.

2. Flow Nets

Steady flow of an incompressible fluid through a porous medium can be expressed mathematically by Laplace's equations, of which the flow net is a graphical solution. The flow net is a grid formed by the intersection of two sets of orthogonal lines. One set of lines, the flow lines, represents the loci of particles of liquid as they pass through the porous medium. The other set of orthogonal lines, known as the equipotential lines, are piezometric contours; they represent the loci of points having the same pressure head. The flow net is used in estimating the rate of seepage flow and to predict the piezometric pressures at any point within the embankment cross-section.

Flow nets may be constructed using graphical procedures (sketched by hand), electric analogs or from model studies. Alternatively, the Laplacian equations may be solved by digital computer using either the finite-element or the finite-difference methods (Kealy and Busch, 1971, Sharp, 1970).

Flow nets produced by sketching methods are economically and easily obtained and, for many problems of seepage analysis, they give sufficiently accurate results. When estimating the rate of seepage through the embankment, even a crudely drawn flow net will yield reasonably accurate results. When the flow net is used to predict the piezometric head at any point within the embankment, accurately drawn flow nets are required. Several examples of flow nets are illustrated on Figures 5-5 and 5-6; general rules for their construction, using graphical procedures,are given on Figure 5-7. A more detailed treatment of flow net construction by sketching methods has been given by A. Casagrande (1937). Experience is required to draw accurate flow nets.

3. Underdrains

The position of the phreatic surface (water table) within an embankment has a marked influence on its stability. For example, a slope composed of cohesionless sand having a total unit weight of 120 pounds per cubic foot and an angle of internal friction of 35 degrees will remain stable at a slope angle of 35 degrees. (1.43 horizontal to 1 vertical) regardless of the height of the slope, providing the phreatic surface is below the elevation of the toe to the slope. If the phreatic surface is raised to the surface of the slope, the steepest stable slope angle is reduced to 18-1/2 degrees (3 horizontal to 1 vertical), less than one-half the angle for the drained case.

Economic considerations dictate that the main bulk of a tailings embankment be constructed using the most readily available fill materials commensurate with adequate stability of the structure. If the permeability of the bulk of the embankment fill is of the same order of magnitude or is less than the tailings adjacent to the embankment, drains should be provided beneath the downstream shell of the embankment to lower the phreatic surface. The drainage system may consist of granular blankets, strip drains, or of drainage pipes installed prior to placement of the embankment fill. An underdrainage system has the following advantages:

> the phreatic surface will be located below the downstream face of the embankment, thereby avoiding the problem of erosion and sloughing at the point where seepage might otherwise exit; where the downstream slope of an embankment is composed of fine-grained materials, water should not be allowed to flow out of this slope;
>
> lowering the phreatic surface increases the stability of the embankment section, thereby permitting the use of steeper downstream slopes and a reduction in the quantity of fill required for construction of the embankment;

frost penetration below the downstream face will not impede drainage; since the phreatic surface within the embankment will be located below the zone of frost penetration, the void spaces between soil particles will not become blocked by ice;

the formation of ice lenses, and surface sloughing with subsequent thawing, is less likely since the water will not be available for the formation of ice lenses as the soils on the downstream slope freeze;

Figures 5-5 and 5-6 illustrate the effectiveness of underdrains and pervious foundations in lowering the phreatic surface.

4. Design of Underdrains

The choice between drainage blankets, strip drains and pipe drains depends on the availability of suitable drainage materials, the drainage capacity required, the cost of construction, and the foundation conditions. The permeability of the drain material should be at least 100 times greater than the permeability of the "impervious" zone, and its gradation must satisfy the requirements for filters.

If the foundation beneath the tailings embankment is compressible, and significant differential settlements are anticipated, the use of pipe drains should be avoided. The lateral strains associated with differential settlements may result in opening of pipe joints, loss of material into the pipes and development of internal erosion that may be virtually impossible to control.

Where pipe drainage systems are used, the pipes should be designed to withstand the maximum anticipated loads, including those imposed by settlement of the overlying fill. If perforated drainage pipes are used, they should be installed with the perforations down, and the diameter of the perforations should not be larger than one-half of the 85 per cent size of the drainage material surrounding the pipe. Pipe drains can seldom be repaired. In view of the serious consequences that may ensue as a result of the collapse of pipe sections, or opening of pipe joints, pipe drainage systems should be avoided; strip drains and blanket drains are preferable.

Strip drains consist of strips of pervious drainage material placed on the foundation (in some cases at higher levels also) prior to placement of overlying embankment fill. The arrangement and alignment of strip drains will be governed by the contours of the foundation surface. The drains should be provided with adequate fall to outlets located beyond the downstream toe of the embankment.

Use a flow net to estimate the rate of seepage per unit length of embankment. The estimated rate of seepage through the embankment will depend on the value selected for the coefficient of permeability of the impervious material through which the seepage must pass before entering the drain. To ensure that the drain design is adequate, the upper limit of the probable range of coefficients of permeability should be used in these calculations.

Determine the thickness of the drainage blanket, or the dimensions of strip drains, to ensure that their capacity is not less than the calculated rate of seepage through the embankment. The lower limit of the probable range of coefficients of permeability of the drain materials should be used in these calculations. Where the foundation strata are relatively permeable and the natural groundwater table is high, the design capacity of the drainage system should take into account any seepage that may enter the drainage system from the foundation strata. Where the natural groundwater table is at an appreciable depth, some of the seepage through the embankment may drain into the foundation strata, thereby reducing the required capacity of the drainage system. If the foundation strata contain layers or laminations of relatively impervious material, loss of seepage into the foundation may be severely restricted, in which case an impermeable foundation should be assumed. The dimensions of the drains should be as generous as practicable commensurate with the quality and cost of the materials available, the need to construct the drains without constrictions, gaps, or segregation of materials, and the extent to which the construction will be supervised. The thickness of blanket drains and of strip drains should be at least 12 inches and the width of the drain should not be less than 10 per cent of the difference in elevation between the pond surface and the drain.

Blanket drains and strip drains should be designed to be capable of passing full design flow when the phreatic surface within the drain is at or below the upper surface of the drainage material.

Granular materials incorporated in underdrainage systems should be compatible with the properties of the seepage water they are designed to carry. Drainage materials composed of carbonate rocks are unsuitable if the seepage collected by the system is acidic.

5. Transitions and Filters

Where the embankment cross section is constructed of zones of materials having significantly different gradation, or where the gradation of foundation materials is significantly different than the gradation of materials used in the construction of the embankment, these zones of significantly different gradation should be separated by transition and filter zones to prevent piping and subsurface erosion.

Filters are required to separate zones where the seepage water passes from finely-graded to coarsely-graded materials. To be effective, the filter must serve two functions; it must be more permeable than the adjacent finer soil, so that it will act as a drain and freely conduct water away from the interface between the protected zone and the filter, and it must have a gradation such that its voids are sufficiently small to prevent the passage of soil particles from the protected soil. The gradation of the filter material should be such that segregation of particle sizes does not occur during handling and placement of the materials.

To ensure that the above criteria are satisfied, the following rules should be applied in selecting suitable filter materials. Rules 1, 2 and 4 are illustrated on Figure 5-8.

Rule 1: $\dfrac{\text{The 15\% size of the filter}}{\text{The 85\% size of the protected soil}}$ should be less than 5.

Rule 2: $\dfrac{\text{The 50\% size of the filter}}{\text{The 50\% size of the protected soil}}$ should be less than 25.

Rule 3: The filter material should be smoothly graded; gap graded materials should be avoided.

Rule 4: $\dfrac{\text{The 15\% size of the filter}}{\text{The 15\% size of the protected soil}}$ should be greater than 5.

Rule 5: The filter should not contain more than 5 per cent of particles, by weight, finer than the No. 200 sieve, and the fines should be cohesionless.

Rule 6: The coefficient of uniformity of the filter should be equal to or less than 20.

Rules 1, 2 and 3 assure that the particles of the protected soil do not pass through the filter. Rules 4 and 5 assure that the coefficient of permeability of the filter will be sufficiently high to freely conduct the seepage that emerges from the downstream boundary of the protected soil. Rule 6 makes the filter material less susceptible to particle segregation during placement.

As illustrated on Figure 5-8, Rules 1 and 2 should be applied to the fine side of the grain size envelop for the protected soil. However, where the material to be protected contains an appreciable percentage of gravel, or larger sized particles, the rule should be applied to the grain size envelop representing the minus 10-mm. (3/8-inch) fraction of the soil to be protected.

To ensure that the filter will be appreciably more permeable than the coarsest zones of the soil to be protected, Rule 4 is applied to the coarse limit of the protected soil and the fine limit of the filter.

To provide a suitable filter between fine grained soil and very coarse fill (for example a coarse rock fill), or where it is necessary to limit the hydraulic gradient within a drain to a small amount and only a limited cross-sectional area is available for discharge of seepage, it may be necessary to use two filter layers. Each filter layer should be progressively coarser in the direction of seepage flow and should satisfy the filter rules with respect to the adjacent finer material.

The design thickness of filter zones will be dependent on the inclination of the zone and the method of placement. Where machine placement is used to construct steeply inclined filter zones within an embankment section, the width of the zone should be sufficiently wide to permit efficient operation of the construction equipment. Filter zones placed on relatively horizontal surfaces using machine placement should generally have a thickness of not less than 2 feet. Filter zones placed by hand methods in confined spaces should have a minimum thickness of 6 inches.

6. Relief Wells

Where embankment foundations are subject to artesian pressures, stability requirements may necessitate the use of relief wells to control pore water pressures beneath the embankment. Water from the relief wells should be discharged in suitably constructed ditches or pipe drainage systems located downstream of the toe of the embankment.

The required spacing between relief wells will depend on the hydrogeology of the foundation; publications describing their design are listed in the bibliography. (U.S. Army Corps of Engineers). However, since unknown variations in the stratigraphy and the permeability of the foundation soils may have an appreciable effect on the drawdown surface produced by the wells, the adequacy of an installation should be checked by piezometers. The capacity of a relief well system can be readily expanded if the piezometric data shows that the initial installation is not adequate to produce the required degree of control.

Attention should be given to the design details of relief wells to ensure their permanence, and to allow for inspection and maintenance. Chemical analyses of the groundwater should be performed to determine the dissolved ions carried by the groundwater so that the design can incorporate special features that may be required to guard aginst corrosion of any part of the relief well system. Changes in groundwater chemistry that may develop after startup of the milling operations should also be considered. The internal diameter of perforated piping or well screens will be governed by the flows anticipated, but should not be less than 6 inches. To assure free flow of water into the well casing, screens should be surrounded with drainage materials conforming to the gradation requirements for filters. Monitoring of relief wells by piezometers and flow measurements are essential to detect deterioration and to indicate timing of necessary maintenance.

7. <u>Rate of Seepage</u>

The rate of seepage through the tailings embankment and its foundation is controlled by the difference in piezometric elevation between the pond and the downstream toe of the embankment, the hydraulic gradient, and the coefficients of permeability of the materials through which seepage must pass to escape from the tailings pond. The rate of seepage may be reduced by increasing the length of the seepage path (reducing the hydraulic gradient) or by constructing an "impervious" (meaning "low-permeability") barrier across the path that the seepage must follow in passing through and beneath the structure.

The mathematical expression for the rate of seepage is:

$$q = k \frac{n_f}{n_d} h,$$

where: q = the rate of seepage per unit length perpendicular to the plane of the flow net,

k = the coefficient of permeability of the soil,

n_f = the number of flow paths (determined from the flow net),

n_d = the number of equipotential drops (determined from the flow net),

h = the difference in piezometric head between the point of seepage entry and the point of seepage exit.

An approximate value of the coefficient of permeability can be determined from the results of field and laboratory tests as described in Section 4. The prediction of the applicable average coefficients of permeability requires considerable judgment, particularly for foundation materials.

8. Cores, Blankets and Membranes

Where the most economically available materials for embankment construction are pervious, or where the tailings disposal area is underlain by pervious foundation soils, pollution requirements and/or limited supplies of mill make-up water, may necessitate the use of impervious cores, blankets, or membranes to reduce the rate of seepage from the pond.

Seepage through a pervious embankment can be controlled by incorporating a vertical or inclined zone of impervious material within the embankment section. Where a pervious foundation is underlain at shallow depths by impervious deposits, a trench may be excavated to permit extension of the core through the pervious foundation soil. The portion of the core extending below ground surface is referred to as the core trench (or cutoff trench). In general, a core or cutoff trench should be located within the upstream portion of the embankment so that drained conditions will be maintained within as much of the embankment cross-section as practicable. A core trench is illustrated schematically on Figure 5-3.

As an alternative to the cutoff trench, blankets or membranes may be used to extend the impervious zone upstream of the embankment. The main purpose of the blanket or membrane is to increase the length of the seepage path through the pervious foundation, thereby reducing seepage losses from the pond as well as the hydrostatic pressures within the foundation beneath the embankment. The thickness of the impervious core within the embankment and the thickness and areal extent of upstream blankets and membranes, will be governed by the tolerable rate of seepage and the coefficient of permeability of the material incorporated within the impervious zones. The dimensions of the impervious zones should be such that the estimated leakage does not exceed the maximum allowable rate when the upper limit of the probable range of the coefficient of permeability is applied.

As an alternative to methods designed to reduce seepage, and depending on site conditions, it may be possible to operate a closed system whereby the seepage that emerges on the downstream side of the embankment is collected and either treated, returned to the pond or to the milling operations.

9. Hydraulic Barriers

Where the tailings dam is constructed on a thick pervious foundation, and pollution control requirements preclude the escape of water from the tailings pond, seepage losses may be controlled by development of a hydraulic barrier downstream of the tailings dam.

The hydraulic carrier (illustrated on Figure 5-3) can be produced by a line of pumping wells and a line of injection wells downstream of the embankment, the injection wells being located downstream of the pumping wells. Freshwater is supplied to the injection wells, while groundwater is extracted from the pumping wells. Providing the piezometric water levels along the line of the injection wells are maintained at elevations higher than the piezometric water levels along the line of the pumping wells, a hydraulic barrier will be formed that will prevent the flow of seepage from the tailings pond past the line of pumping wells. This should be checked in the field by piezometric measurements.

Surface Runoff Control

Tailings embankments having a substantial drainage area may require some means of controlling the runoff into the pond. Methods of estimating the magnitude of the runoff have been described in Section 4. Generally, required water-reclaim-system capacities are proportional to the capacity of the ore processing plant. Even for a large plant of 25,000 tons per day capacity, reclaim requirements are not likely to exceed 20 cubic feet per second. With drainage areas exceeding even one-half square mile, the maximum rate of runoff may exceed several hundred cubic feet per second, greatly in excess of the reclaim capacity required by the plant. In order to handle such peak flows, decant or other reclaim systems may have to be designed specifically to pass them, or other measures must be taken. Such other measures could include:

> storing the whole volume of flood runoff in the pond; in this case sufficient freeboard would always have to be available during flood seasons to provide the necessary storage capacity without overtopping of the embankment;
>
> intercepting a portion of the runoff before it reaches the pond and diverting it around the embankment; measures would still be required to control the runoff from the catchment area located downstream of the diversion channel intake and, possibly, any runoff flows in excess of the diversion channel capacity;
>
> providing spillways at various temporary crest levels to convey runoff water safely past the embankment; particularly, such spillways may be required during periods when mining has been suspended and at the final embankment crest level.

Determination of the freeboard required at any time to store flood runoff will involve calculating the pond storage versus elevation curve from topographic maps. The embankment construction schedule would then have to ensure that this required freeboard was always available during seasons when floods could occur. The most critical period will always be during the early years of waste disposal, when the storage capacity of the pond is small.

Methods for the design of diversion channels and spillways are described in readily available hydraulics handbooks. Several of these are listed in the references. Usually, the most critical point in their design is the avoidance of erosion affecting the safety of the embankment. For this reason, the gradients of diversion and spillway channels should be kept sufficiently flat that erosive velocities will not occur near to the embankments. Alternatively, channels may be protected against erosion with various kinds of lining or with stone paving. Tables and graphs showing the magnitude of permissible flow velocities for various classes of natural soils and the sizes of paving stones required to prevent erosion, are given on Figures 5-9, 5-10 and 5-11. To be effective in preventing erosion of underlying fine soils, paving stones should be based on a layer of filter gravel graded as described previously.

Embankment Freeboard and Wave Protection

In addition to freeboard required for the storage of runoff water, a minimum freeboard should be provided on tailings embankments to prevent overtopping of the embankment crest by waves generated by winds. The height of wave depends on the wind velocity, the duration of the wind, the fetch (the distance over which the wind can act on the water) and the depth of water. For most tailings ponds, the maximum wave height is governed by the fetch distance.

If a broad, flat beach is maintained on the upstream side of an embankment, waves will break and their energy will be dissipated on the beach, thereby providing a measure of protection against overtopping by breaking waves. On steep upstream slopes, riprap will limit the uprush of the waves to approximately 1.5 times the height of the waves and will prevent erosion of the face by the wave action. Riprap could be necessary on tailings embankments constructed across the bays of natural lakes, or on completed embankments which are left impounding a substantial pond of water. Tables of approximate wave heights for various values of wind velocity and fetch, and the necessary freeboard and riprap gradation for 3:1 riprapped slopes, are given on Figure 5-12. For 2:1 slopes, the nominal thickness should be increased by 6 inches. With fine embankment materials, a layer of filter gravel should be provided beneath the riprap.

The freeboard should be measured from the maximum flood water level to the crest of the embankment. The maximum flood level usually will be a function of the type and capacity of the spillway provided to pass runoff flows.

Minimum Embankment Crest Width

The most suitable crest width for a tailings embankment will depend on the allowable percolation distance through the embankment at full pond level, the height of the structure and the practicability of construction. For equipment operation, the width of the crest should be not less than 12 feet. For embankments under about 100 feet in height, a suitable minimum crest width is given by the equation:

$$W = \frac{z}{5} + 10,$$

where W = crest width in feet,

z = height of the crest above the foundation at its lowest point.

Tailings dams over 100 feet in height should have crests not less than 30 feet in width.

WASTE PILES

General

To appraise any proposed waste disposal area, the designer must know the approximate total volume of the waste materials that will comprise the waste pile. Maps or plans showing the topography within the area proposed for waste disposal should be available, and the strength parameters of both the waste and the foundation materials should be known.

Since the stability of both the waste and the foundation materials is influenced by pore water pressures, the construction of waste piles across major drainage courses should be avoided or, alternatively, provision should be made for unimpeded passage of the flows beneath the base of the piles. To minimize ponding and entry of surface runoff water into the base of the pile, ditches should be constructed along the uphill side of the waste dispoal area to intercept and divert surface runoff water, and to conduct it beyond the lateral limits of the proposed disposal area.

The areal extent of the waste pile will be governed by the volume of waste, the height of the pile, and the permissible slopes at its perimeter. The stability of the waste pile will be governed by one or more of the following factors: the shear strength characteristics of the materials comprising the waste pile; the shear strength characteristics of the in situ foundation materials on which the waste pile is constructed; the pore water pressures within the waste pile and its foundation; the slope of the ground surface on which the waste materials are placed; the height of the waste pile; and the slopes at the perimeter of the pile.

The height of the pile and the slopes surrounding its perimeter are interdependent; both are governed by the shear strength of the materials comprising the pile and/or the shear strength of the foundations. In general, the height of the waste pile can be increased if the slope is flattened and, conversely, the slope can be steepened if the height of the pile is reduced.

If the shear strength characteristics of the foundation are appreciably higher than the shear strength characteristics of the materials comprising the waste pile, the maximum height of the pile and the maximum permissible slope will be governed by the shear strength of the waste materials. Where the shear strength of the foundation is appreciably lower than the shear strength of the waste materials, the maximum permissible pile height and slope will be governed by the shear strength of the foundation.

The strength of the waste materials is governed by the type of materials, their density and the pore water pressures within the waste pile. The density and the pore water pressures are governed, to a degree, by the methods used to place the waste materials within the pile.

With the exception of the case where free-draining cohesionless waste materials are placed on a competent foundation, the maximum permissible height and slope of the waste pile must be determined by stability analyses.

Classification of Foundations

Foundations for waste piles may be classified according to the strength characteristics of the in situ subsurface materials and according to the topography of the ground surface. As used here, the term "competent foundation" refers to foundation materials which exhibit shear strength characteristics that are higher than those of the waste materials; the term "weak foundation" refers to foundation materials that exhibit lower shear strength characteristics than the waste. The terms "competent foundation" and "weak foundation" are therefore dependent on the strength characteristics of waste materials comprising the waste pile.

"Level foundation" refers to areas where the slope of the surface of the waste pile foundation is less than 10 degrees. Where the slope of the foundation surface within the waste disposal area is steeper than 10 degrees, the term "sloping foundation" is used.

A waste pile on a level foundation will generally be safe against mass sliding along its base. Where a waste pile is to be constructed on a sloping foundation, an analysis should be made to check the stability of the pile with respect to mass displacement as a result of shearing along its base. The slope of the ground surface within some areas may be so steep as to be unsuitable for disposal of waste materials.

Where the slope of the ground surface is relatively uniform, the analysis should consider plane down-slope sliding of the pile along its base. Providing the base of the waste pile is drained so that excess hydrostatic pressures cannot develop, the steepest permissible foundation slope can be determined from the equation:

$$\tan i \leq \frac{\tan \delta}{F},$$

where: i = slope of the foundation,

δ = friction angle between the base of the pile and its foundation,

F = factor of safety.

In some instances, the slope of the ground surface within the prospective waste disposal area may be non-uniform, slope angles at lower elevations being flatter than i, while those at higher elevations are steeper than i. For these foundation slope conditions, the stability against shearing along the base of the pile should be checked by the wedge method of stability analysis described later. (See Figure 5-19).

If the stability analysis indicates that the factor of safety for the completed waste pile will be unacceptably low, the steeper sloping portions of the prospective area will be unsuitable for disposal of waste materials. If the wedge method of analysis indicates that the completed waste pile will have an acceptable factor of safety against sliding along its base, it will be necessary to place waste materials on the flatter portions of the slope, prior to placement over the steep upper slope, to support the upper portion of the waste pile.

If a waste pile constructed on a sloping foundation has an adequate factor of safety against sliding along its base, the maximum height of the waste pile, and the maximum permissible slope, can be determined by the same methods of analysis used for waste piles on level foundations.

The factor of safety of a waste pile will most likely be lowest, and failure most likely to occur, following a period of prolonged rainfall. After prolonged rainfall, the near-surface soils surrounding the waste pile may also be saturated. If failure were then to occur along the downslope side of the waste pile, the unstable mass of waste material could move over the saturated soils surrounding the pile. These soils, if relatively impervious, may be able to offer little more than their undrained shear strength to resist the movement, and may be inadequate to resist the shearing stresses imposed by the unstable mass of waste material. Under these circumstances, the moving pile may accelerate rapidly and travel considerable distance downslope. An unstable waste pile on a sloping foundation is potentially very dangerous. For this reason, the factors of safety used in the design of waste piles on sloping foundations should be higher than those for waste piles on level foundations.

Classification of Waste Materials

Waste materials may be divided into three broad classifications - frictional, cohesive and degrading. Frictional materials include permeable sand, sand-gravel mixtures and broken rock of any gradation and particle size, providing the mineralogical composition of the rock is such that the material does not suffer significant loss in strength, or reduction in permeability, during long-term storage in the waste pile.

Cohesive wastes include clays, silts, cohesive glacial till and other materials which exhibit both frictional and cohesive properties and which do not suffer significant reduction in their shear strength parameters, or their permeability, during long-term storage in the waste pile.

Degrading wastes are those which break down by mechanical or chemical processes when exposed to air and/or free moisture and which may suffer significant reduction in shear strength and permeability following placement in the pile.

Piles of Permeable, Frictional Wastes

When the waste materials are frictional, and consist of blasted rock or of permeable sand-gravel mixtures, the initial layer of waste material in contact with the foundation may be placed in the pile by casting, end dumping, or by bulldozing over a face. Segregation of coarse particle sizes will occur as the materials cascade down the face, so that a concentration of the coarest fraction of the materials will be deposited at the base of the lift. As the leading edge of the fill is advanced, the coarse sorted fraction will be covered, thereby forming an underdrain at the base of the pile. This layer of coarse segregated material will generally preclude the development of hydrostatic pressures at the base of the pile.

If the waste materials consist of sand, segregation of particle sizes on the advancing face of the fill may not be significant, so that the permeability of the materials as the base of the waste pile may not be significantly higher than the average permeability of the materials comprising the bulk of the pile. Drainage from within the pile may result in minor sloughing at the toe. If the waste materials must be confined within stringent boundaries, it may be advisable to provide a toe drain at the perimeter of the waste pile. The gradation of the materials used to construct the toe drain should conform to the gradation requirements for filters.

Where pervious, frictional waste materials are placed on a competent foundation, the maximum possible slope at the perimeter of the pile will be equal to the angle of repose of the waste materials. The angle of repose represents the lower limit of the angle of internal friction for the waste material and will normally vary between 30 and 40 degrees. Where the foundations within the proposed waste disposal area are level and competent, and where drained conditions are maintained within the pile, frictional waste materials can be placed to practically unlimited height at their angle of repose.

Where frictional waste is placed on a "weak foundation", the maximum permissible height of the waste pile and the average perimeter slopes will be governed by the shear strength characteristics of the foundation. The pile heights, and the perimeter slopes, should be determined by stability analyses.

The various approaches in determining the permissible heights and slopes of piles of frictional wastes are illustrated on Figure 5-13.

Piles of Cohesive Wastes

The permissible heights and perimeter slopes for waste piles containing cohesive materials should be determined by stability analyses. Approaches in determining the permissible combinations of height and slope for any required factor of safety are illustrated on Figures 5-14 and 5-15.

Where the cohesive materials are placed on a competent foundation, the height of the pile and the perimeter slope will be governed by the strength of the materials comprising the pile. Where the cohesive materials are placed on a weak foundation, the strength of the foundation will govern.

Both the friction angle and the cohesion of the materials in a waste pile can be increased if the materials are compacted. The maximum shearing stresses within the pile occur beneath the perimeter slopes. Compaction of the materials beneath these slopes will permit the use of higher fills and steeper slopes if the pile is based on a competent foundation. This higher strength perimeter zone will provide lateral support for weaker uncompacted materials placed within the central portions of the pile. Effective limits of compacted perimeter zones are indicated on Figure 5-16. In some cases, itmay pay to use compaction equipment and methods developed for highway and earth dam embankments. Stability studies for special cases may indicate that the boundaries of the compacted zones may be shifted outward slightly from those shown on the figure.

As compaction will also reduce permeability, it may also be advantageous to place a drainage layer on the foundation beneath the perimeter of the pile. The drainage layer will tend to lower the phreatic water surface beneath the slope thereby increasing the stability of the pile.

When hauling equipment is used to place waste materials within a pile, the traffic of the hauling equipment may produce zones of lower than average permeability within the pile. These zones will be roughly horizontal and will tend to impede downward drainage within the pile, possibly contributing to the development of pore pressures and reducing permissible heights and slopes.

Where the waste disposal area is limited, it may be necessary to incorporate horizontal drainage layers within the pile during construction, and/or to provide inclined drainage zones at the perimeter of the pile as indicated on Figure 5-16. The drainage zones will serve to control pore water pressures within the pile.

To retard entry of surface water into waste piles during periods of rainfall and snow melt, exposed surfaces should be adequately drained as described in Section 7.

Degrading Wastes

One of the most difficult problems in the design of piles for the storage of wastes which degrade by weathering, softening or chemical change is the determination of the shear strength parameters to be used in stability analyses. The shear strength parameters used in the design should be the lowest to which the materials may be reduced as a result of long-term storage in the waste pile. Methods for investigating the design shear strength parameters of degrading materials are described in Section 4. In other respects, design procedure for piles containing degrading wastes are similar to those described for piles of cohesive wastes.

Usually, the degree to which degrading waste materials lose strength can be reduced by sealing the surface of the pile, to reduce entry of surface water and air circulation.

STABILITY ANALYSES

General

Soil and rock materials fail in shear if the applied shearing stresses on any surface exceed the shear strength of the materials along that surface. Stability analyses involve comparing the shearing stresses along potential failure surfaces with the available shearing resistance along those surfaces. The factor of safety (F) is defined as that factor by which the shear strength parameters must be divided (c'/F and ϕ'/F) to bring the potential sliding mass into a state of limiting equilibrium. The stress-strain characteristics of most soils are such that relatively large plastic strains may occur as the applied shearing stresses approach the shear strength of the material. In the design of a slope or embankment, the factor of safety must be greater than unity so that the strains will not exceeed tolerable limits, and to allow for differences between the pore water pressures and shear strength parameters assumed in design and those that may actually exist within the slope.

Stability analysis is a procedure of successive trials. A potential failure surface is chosen and the factor of safey against sliding along that surface is determined. Different potential failure surfaces are selected and the analysis is repeated until the potential failure surface having the lowest factor of safety is found. This failure surface is known as the critical failure surface. The factor of safety against sliding along the critical failure surface is the indicated factor of safety for the slope.

The critical failure surface may be located completely within an embankment; it may lie totally outside of the embankment if it passes through retained materials and/or the foundation soils, or it may be locatedat any position between these two limits.

With the exception of a few special cases, all stability calculations to determine factors of safety should be based on effective stress analyses. The determination of the effective stresses requires a knowledge of the position of the phreatic surface within the embankment. For a fully compressed fill subjected to steady seepage, the effective stress can be determined from flow net. When the fill and/or the foundation are compressing under the weight of overlying material, the pore water pressures must be estimated using consolidation theory. Where the location of the phreatic surface is likely to be critical to stability, piezometers should be installed within an embankment to determine the actual location of the phreatic water surface. Piezometers are described in Section 6. If the actual water pressures are found to be significantly higher than those assumed in design, the stability should be rechecked. Some modification to the design section may be required to maintain the desired factor of safety.

In the design of a tailings embankment, the stability of the downstream slope is usually of principal concern. However, the stability of the upstream slope should also be checked, particularly if it is raised a considerable height above the surface of the pond. If the upstream face of the embankment is relatively steep and water level fluctuations over a considerable range are expected, the analysis should take into account the pore water pressures that may be produced by drawdown conditions, (see Figure 5-5). Drawdown may be a factor also if tailings are to be recovered from the pond, or the embankment is breached to drain the pond.

Failure Surfaces

The most commonly assumed failure surface used in stability analyses is the cylindrical surface, the axis of which is oriented parallel to the strike of the slope. On the two dimensional cross-section used for convenience in most stability analyses, the cylindrical surface is represented by a circular arc. Observations of full-scale slope failures in the field, show that some failure surfaces are nearly circular. However, many carefully documented examples are available which show that the shape of the rupture surface is clearly non-circular. When a slope failure occurs, differential shearing takes place along that surface on which the factor of safety is lowest. Although the calculations are made simpler if the failure surface is assumed to be circular, stability analyses based solely on assumed circular failure surfaces may significantly over-estimate the factor of safety.

The true surface of sliding will deviate from the commonly assumed circular surface if the potential failure surface passes through zones having different shear strength characteristics or different pore water pressure conditions. Methods of stability analyses applicable to non-circular failure surfaces are included in the descriptions given below.

Methods of Analyses

1. Methods of Slices

With the exception of a few special cases, the methods used to calculate the factor of safety for any trial failure surface should account for changes in the shear strength parameters and varying water pressure conditions along the potential failure surface. Changes in the strength parameters and pore water pressure conditions can be taken into account by the general procedure known as the method of slices. In the method of slices, a trial failure surface is chosen and the potential sliding mass is divided into a number of vertical slices. Each slice is acted upon by its own weight, by shearing and normal forces on its vertical boundaries and by shearing and normal forces along its base.

2. Method of Infinite Slices

In the method of infinite slices, a circular trial failure surface is selected, and the stability of the potential sliding mass is considered as a whole, rather than the stability of each individual slice. Since the forces acting on the vertical boundaries of the slices produce zero net moment about the centre of rotation of the potentially unstable mass, the side forces are neglected. The shearing stresses and the normal stresses on the base of each slice are assumed to depend only on the weight of the slice and on the pore water pressures at its base. If the potential failure mass is divided into slices of unit width, the forces on the base of each slice will be numerically equal to the stresses on the base of the slice. This procedure is illustrated on Figure 5-17.

Factors of safety determined using the method of infinite slices will be in error on the conservative side, since the method completely neglects the side forces on the individual slices.

3. Simplified Bishop Method

Stability analysis using the "Simplified Bishop Method" is a variation of the method of slices and is limited to the analysis of circular arc failure surfaces. The potential circular failure surface is selected; the potential failure mass is divided into a number of vertical slices, and the stability of each slice is considered in turn using the assumption that the factor of safety for each slice is equal to the factor of safety for each of the other slices. Each slice is acted upon by its weight, and by shearing forces and normal forces on its vertical boundaries.

The shearing and normal forces acting on the vertical boundaries depend on the stress-deformation characteristics of the materials comprising the slide mass and cannot be evaluated rigorously. However, the summation of the forces acting on the vertical boundaries of the slices is zero, and these forces may be neglected without serious reduction in accuracy. Neglecting the forces acting on the sides of the slices, the factor of safety is expressed by the equation:

$$F = \frac{\Sigma\ c'b \sec \alpha + N' \tan \phi'}{\Sigma W_o \sin \alpha},$$

Where W_o = the weight of material within the slice,

c' = the effective cohesion for the soil,

b = the width of the slice,

α = the angle of inclination at the centre of the base of the slice,

N' = effective normal force on the base of the slice,

ϕ' = the effective angle of internal friction,

F = the factor of safety.

For the Simplified Bishop Method, N' is determined by the sum of the forces in the vertical direction according to the equation:

$$N' = \frac{W_o - b \sec \alpha \left(u \cos \alpha + \frac{c'}{F} \sin \alpha\right)}{\cos \alpha + \frac{\tan \phi' \sin \alpha}{F}},$$

where u = the pore pressure. Therefore,

$$F = \frac{\Sigma\ \left(c'b + (W_o - ub) \tan \phi'\right) \dfrac{\sec \alpha}{1 + \dfrac{\tan \phi' \tan \alpha}{F}}}{\Sigma W_o \sin \alpha}.$$

Since F appears on both sides, the equation must be solved by successive approximations. The procedure for calculating the stability for a single trial failure surface is indicated on Figure 5-18.

Using a more rigorous form of Bishop's analysis, the shearing forces and the normal forces acting on the vertical boundaries between adjacent slices may be taken into account. However, if the surface of sliding is circular, the improvement in accuracy is not likely to exceed 10 to 15 per cent.

4. Morgenstern-Price Method

Where the potential failure surface deviates significantly from the circular configuration, methods of analysis that neglect the effect of the shearing and normal forces on the lateral boundaries of the slice may lead to significant error. Morgenstern and Price (1965) have presented a method for calculating the factor of safety for non-circular surfaces of sliding. This method takes into account the shearing forces and normal forces acting on the lateral boundaries of the slices and satisfies the conditions for horizontal, vertical and moment equilibrium for each slice. Owing to the number of iterative steps required to obtain solutions using the Morgenstern-Price analysis, calculations using electronic computers are virtually mandatory. Each solution indicates the factor of safety for a single trial failure surface. Additional trial failure surfaces must be selected and the factor of safety computed for each until the critical failure surface has been located.

It is emphasized that, regardless of the sophistication of the method of analysis and the capacity of the computing facilities available for stability analyses, the reliability of the calculated factors of safety is governed primarily by the degree to which input parameters are representative of the actual conditions within the embankment and its foundation.

5. Wedge Analyses

If computing facilities and appropriate programmes are not available to the designer, a reasonably accurate assessment of the factor of safety can be made for non-circular surfaces of sliding by manual computation. Where the potential surface of sliding does not differ greatly from a circular arc, the method of analysis illustrated on Figure 5-18 may be used. Where the configuration of the trial failure surface conforms approximately to two or more intersecting tangents, the factor of safety may be determined by using the wedge analysis illustrated on Figure 5-19.

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6. Horizontal Translation

Where a soft foundation stratum is located beneath the embankment, the factor of safety against horizontal translation should be checked. In checking the stability against horizontal translation, a trial failure surface is chosen that passes through the soft foundation layer, and the components of all of the forces acting on the potential failure mass are determined. The degree of safety is presented by the sum of the horizontal components of all forces tending to resist horizontal translation divided by the sum of the horizontal components of all forces tending to produce horizontal translation. An example showing the method of analysis is shown on Figure 5-20.

Effects of Earthquake

1. Embankment Distortions

Where an embankment or a slope is subjected to a seismic disturbance, accelerations that accompany the ground motions produce stress fluctuations so that the dynamic shearing stresses are alternatively higher and lower than the static shearing stresses.

The general distribution of 100-year return period earthquake accelerations for eastern and western Canada is shown on Figure 5-21. Risk from this factor can be reduced by either increasing the freeboard, increasing the width of the crest or decreasing the slope angle. In the latter case, a reasonable basis for design is to ensure that the factor of safety indicated by an equivalent static analysis is greater than unity when the 100-year acceleration forces are included.

Strains which may occur during intervals of higher-than-static stress may result in distortion of the embankment as shown on Figure 2-9. Procedures for estimating the magnitude of these strains have been proposed by Newmark, (1965) and by Goodman and Seed, (1965). However, these analyses are very complex. Provision to limit excessive distortion of embankments by earthquake shocks can be made by including additional horizontal acceleration forces in an equivalent static stability analysis. The value of the acceleration forces used in the analyses should be selected on the basis of the probability of earthquakes of various magnitudes occurring in the region of the embankment. Such information can be obtained from the Seismology Division, Earth Physics Branch, Department of Energy, Mines and Resources, Victoria and Ottawa.

2. Liquefaction

In addition to the increased shearing stresses produced by a seismic disturbance certain types of materials may also suffer a significant reduction in their shearing strength. Loose, saturated fine-to-medium sands may liquefy.

The gradation and the mode of deposition of most mine tailings makes them susceptible to liquefaction, and failures of tailings embankments resulting from liquefaction are by no means uncommon. In 1929, liquefaction resulted in failure of the Barahona tailings dam in Chile, permitting 4 million tons of liquefied tailings to flow into a valley below the dam and killing 54 people. The Chilean earthquake of March 28, 1965, resulted in the failure of at least 11 tailings dams located in the area north of Santiago. Of these, the most catastrophic was the failure of the El Cobre tailings dam, which released approximately 2 million tons of liquefied tailings from the impoundment area. These tailings flowed a distance of approximately 7-1/2 miles down the valley below the dam, destroying part of the town of El Cobre and killing more than 200 people.

In Canada and the United States, several failures of tailings embankments have occurred under static conditions. Some of these failures were caused by lateral yielding of the retaining embankments, which permitted lateral straining of the loose and saturated tailings impounded in the ponds. Increases in pore water pressure that accompanied these strains reduced the shearing strength of the tailings which then exerted increased lateral pressures against the embankments. Being already in a state of incipient failure, sections of the embankments collapsed and large volumes of impounded materials flowed out of the ponds.

Most tailings deposited by sluicing or spigotting remain loose and, if saturated, are particularly susceptible to liquefaction. When the upstream method is used for construction of a tailings embankment, the critical failure surface is located at progressively increasing distances from the downstream slope of the embankment as its height increases. For a high embankment, a large proportion of the critical failure surface may be located within the sedimented tailings. Under these circumstances, the stability of the retaining embankment is almost entirely dependent upon the shear strength of the sedimented tailings. Thus, in areas subject to seismic activity, high tailings embankments constructed by the upstream method are particularly susceptible to failure by liquefaction.

For any given saturated sand, the danager of liquefaction as a result of cyclic loading (or of progressive unidirectional shearing strains) is governed by:

the density of the sand; the lower the density, the more easily liquefaction will occur;

the confining pressure acting on the sand; the lower the confining pressure, the more easily liquefaction will develop;

the magnitude of the cyclic stress or strains; the larger the stress or strain, the lower number of cycles required to induce liquefaction.

the number of stress cycles to which the sand is subjected; the greater the number of stress cycles, the greater the probability that liquefaction will occur;

in the case of unidirectional strain, the larger the rate and magnitude of the strain, the greater is the likelihood of liquefaction.

Of these factors, the embankment design cannot influence the magnitude of the cyclic stresses, nor the number of cycles to which the structure may be subjected. However, by incorporating drainage facilities, by maintaining the pond surface as far as practicable from the embankment and/or by compacting the fill materials during construction, the density, saturation and confining pressures can be controlled so as to reduce the likelihood of failure of the embankment as a result of liquefaction.

If the tailings embankment is constructed of fine sands, compaction of these sands will increase their density and reduce their susceptibility to liquefaction. Compaction to in situ relative densities of 60 per cent or greater provides reasonable protection against liquefaction.

Non-saturated sands do not liquefy. Hence, if the phreatic surface can be maintained at a position well below the surface of the embankment, those materials located above the phreatic surface will not be subject to liquefaction. Lowering the water levels within the embankment also increases the effective weight of the materials located above the phreatic surface, thereby increasing the confining pressures acting on the saturated materials below the phreatic surface and reducing their susceptibility to liquefaction. Effective underdrainage systems which substantially reduce the level of the phreatic surface increase and embankment's resistance to failure by liquefaction.

In summary, providing the materials comprising the embankment have a relative density of 60 per cent or greater, and/or providing the phreatic surface is maintained at a position well below the surface of the embankment, the embankment itself will have a reasonable degree of safety against failure by liquefaction. Liquefaction of the impounded tailings adjacent to the upstream face of the embankment may nevertheless occur. The mass of the embankment should be sufficient to provide the required factor of safety against horizontal displacement along its base, when the assumption is made that the shear strength of the tailings adjacent to the upstream face is reduced to zero. The methods of providing protection against failure by liquefaction are illustrated on Figure 5-22.

Factors of Safety

The use of a factor of safety in stability analyses makes two important provisions - it allows for the margin of error between the parameters used in design and those that may actually exist in the field and it limits strains. Many soils undergo relatively large plastic strains as the magnitude of the applied shearing stresses approaches the shear strength of the soil. Thus if the ultimate strength of the soil is used in design, a factor of safety greater than unity is required to maintain strains within tolerable limits.

In choosing the factor of safety, the possible consequences of a failure and the degree of confidence that can be placed in knowledge of the shear strength characteristics of the embankment and the foundation materials, the groundwater conditions and the drainage conditions within the embankment should be considered. As a general rule, it is desirable to introduce into the stability analyses more adverse, but possible, values for some of the factors pertinent to stability, so that their significance can be assessed.

Where failure presents a potential danger to life and property, the field and laboratory investigations should be carried out in sufficient scope and detail to determine the average and lower limits of the shear strength parameters for the materials. Sufficient control should be provided during construction to ensure that materials placed within the embankment conform to the standards assumed in design. Instrumentation should be installed to monitor pore pressures within the embankment and the foundation, if these are significant in determining the stability of the embankment. Pore pressures significantly higher than those assumed in design may necessitate a modification of the design section.

Listed following are suggested minimum design factors of safety. The values presuppose that the stability analysis has been sufficient to locate the critical failure surface and that the parameters used in the analysis are known, with reasonable certainty, to be representative of actual conditions which will exist in the embankment:

	Case I*	Case II**
design based on peak shear strength parameters	1.5	1.3
design based on residual shear strength parameters	1.3	1.2
analyses that include the predicted 100-year return period accelerations applied to the potential failure mass	1.2	1.1
for horizontal sliding on base of embankments retaining tailings in seismic areas assuming shear strength of tailings reduced to zero	1.3	1.3

* Case I - where it is anticipated that persons or property would be endangered by a failure.

** Case II - where it is anticipated that persons or property would not be endangered by a failure.

Where the ratio of residual to peak shear strengths is 0.9 or greater, the embankment design can be based on the peak strength values using the appropriate factors of safety listed.

Where the number of field and laboratory test results on either the embankment fill or the foundations is small, or where the scatter of test results within individual strata or zones is large, conservative values of strength and pore water pressures should be selected for the design, or, alternatively, an increased factor of safety should be used.

Where a waste pile is constructed on a steeply sloping foundation, the suggested minimum factors of safety should be increased by 10 per cent.

SETTLEMENT ANALYSES

If the foundations beneath an embankment consist of dense glacial till, dense sand and gravel or of rock, vertical deformations under the weight of the embankment will be largely elastic. The settlements will occur as the loads are applied and their magnitude will be sufficiently small that their effect will not be significant with respect to the performance of the embankment.

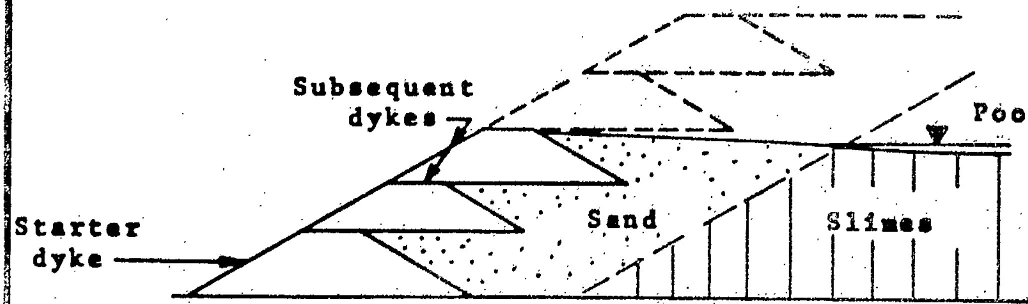
If the foundations beneath the embankment contain layers or strata of normally consolidated fine grained sediments, such as silts and clays, significant vertical deflection of the foundation may occur under the weight of the embankment fill as the fine grained sediments consolidate. The magnitude of the foundation settlement will depend on the height of the embankment, the depth and thickness of the compressible strata within the foundation and their compression indices. The rate at which the foundation settlements occur will depend on: the magnitude of the change in vertical stress (the rate of fill construction); the permeability of the compressible material; and the drainage characteristics of the foundation.

The magnitude of the anticipated settlement can be estimated using the compression indices obtained from the results of laboratory consolidation tests on samples recovered from the compressible strata. Compression indices determined from field settlement records at other sites underlain by compressible strata having similar water contents and index properties are useful in assessing the probable range. The rate of settlement is much more difficult to predict. Computations based on laboratory consolidation test data can be very much in error since in many instances the rate at which foundation settlements occur is controlled by minute geological details, which may not be detected even by carefully conducted foundation investigation programmes.

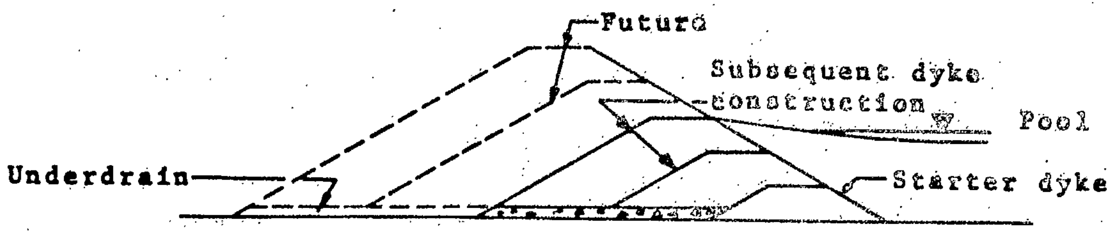
Estimates of the magnitude and the rate of settlement should be made to the extent possible with the best available data. However, these predictions should be checked by instrumenting the initial stage of the embankment and measuring the rate and magnitude of the settlement that actually occurs. These field settlement data may then provide a more reliable basis for a revised prediction of the magnitude of the total and differential settlements to which the ultimate structure will be subjected.

Differential foundation settlements may produce cracking of an embankment which could lead to subsurface erosion. Differential settlements may also cause damage to pipe drains and decant lines installed within or beneath the structure. In assessing the effects that foundation settlements may have, it should be assumed that localized areas will be subjected to differential settlements of at least twice those indicated by the contours of predicted settlements.

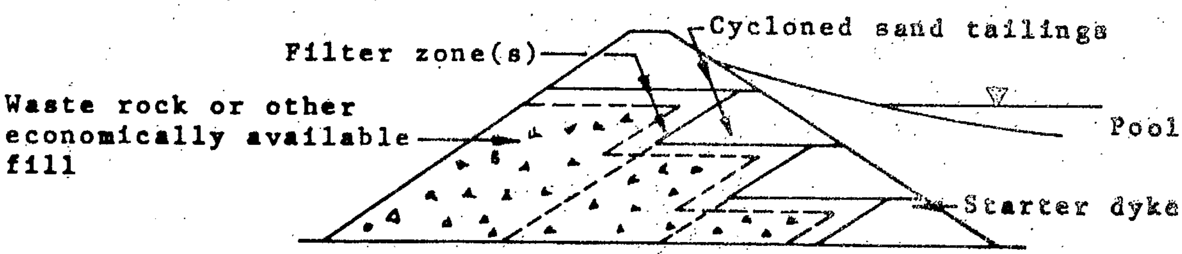
Installation of pipes within or beneath an embankment should be avoided at locations where significant foundation settlements are expected to occur, and decant pipes through a tailings embankment should be located as far as possible away from the areas of maximum anticipated settlement.



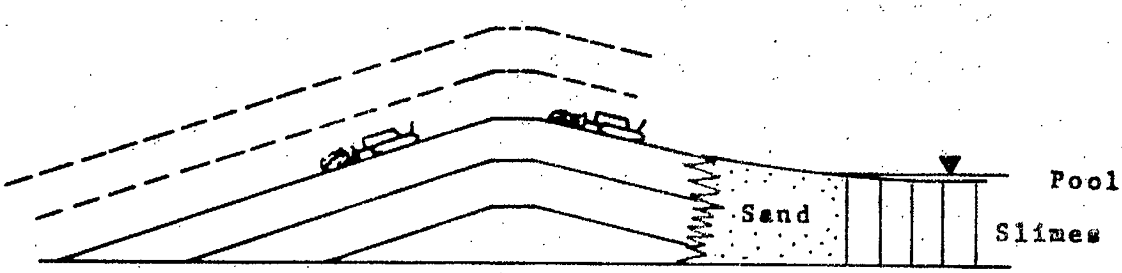
Upstream Method



Downstream Method



Downstream Method
(Supplementary fill incorporated within embankment section because rate of production of suitable cycloned tailings is limited).

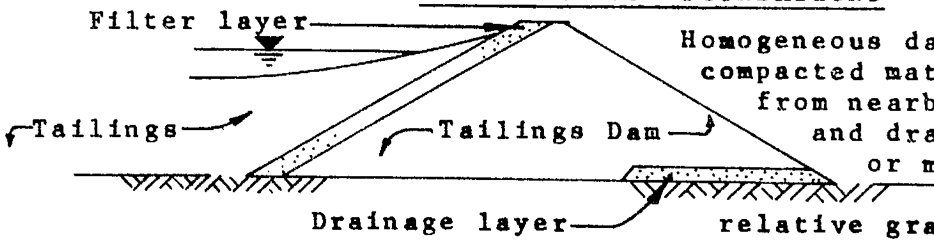


Centre Line Method

TAILINGS EMBANKMENT CONSTRUCTION METHODS

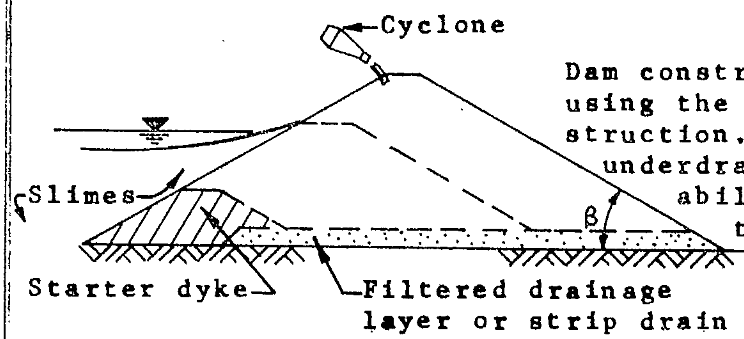
FIGURE 5-1

TYPES OF TAILINGS DAMS ON IMPERVIOUS FOUNDATIONS



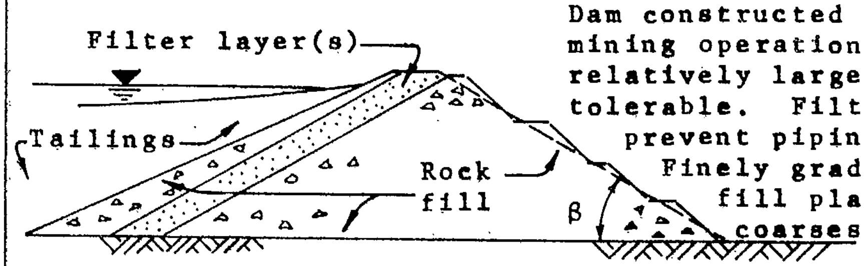
(a)

Homogeneous dam constructed of compacted materials borrowed from nearby source. Filter and drainage layers may or may not be required depending on relative grain size and permeability of tailings, dam fill, and foundation.



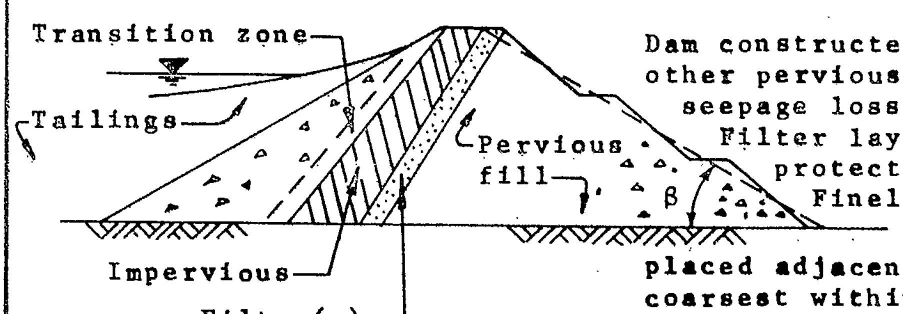
(b)

Dam constructed of cycloned tailings using the downstream method of construction. Requirements for filtered underdrain depend on relative permeabilities of slimes and cycloned tailings. Cotangent β usually greater than 2.0.



(c)

Dam constructed of waste rock from mining operation. Suitable where relatively large seepage losses are tolerable. Filter layers required to prevent piping and internal erosion. Finely graded portions of rock fill placed adjacent to filter; coarsest rock placed within downstream portions of section. Cotangent β usually greater than 1.5. Slopes between berms at angle of repose.



(d)

Dam constructed of waste rock or other pervious fill where large seepage losses not tolerable. Filter layers required to protect impervious zone. Finely graded portions of pervious fill placed adjacent to filter; coarsest within downstream portions of section. Cotangent β usually greater than 1.5. Slopes between berms at angle of repose.

FIGURE 5-2

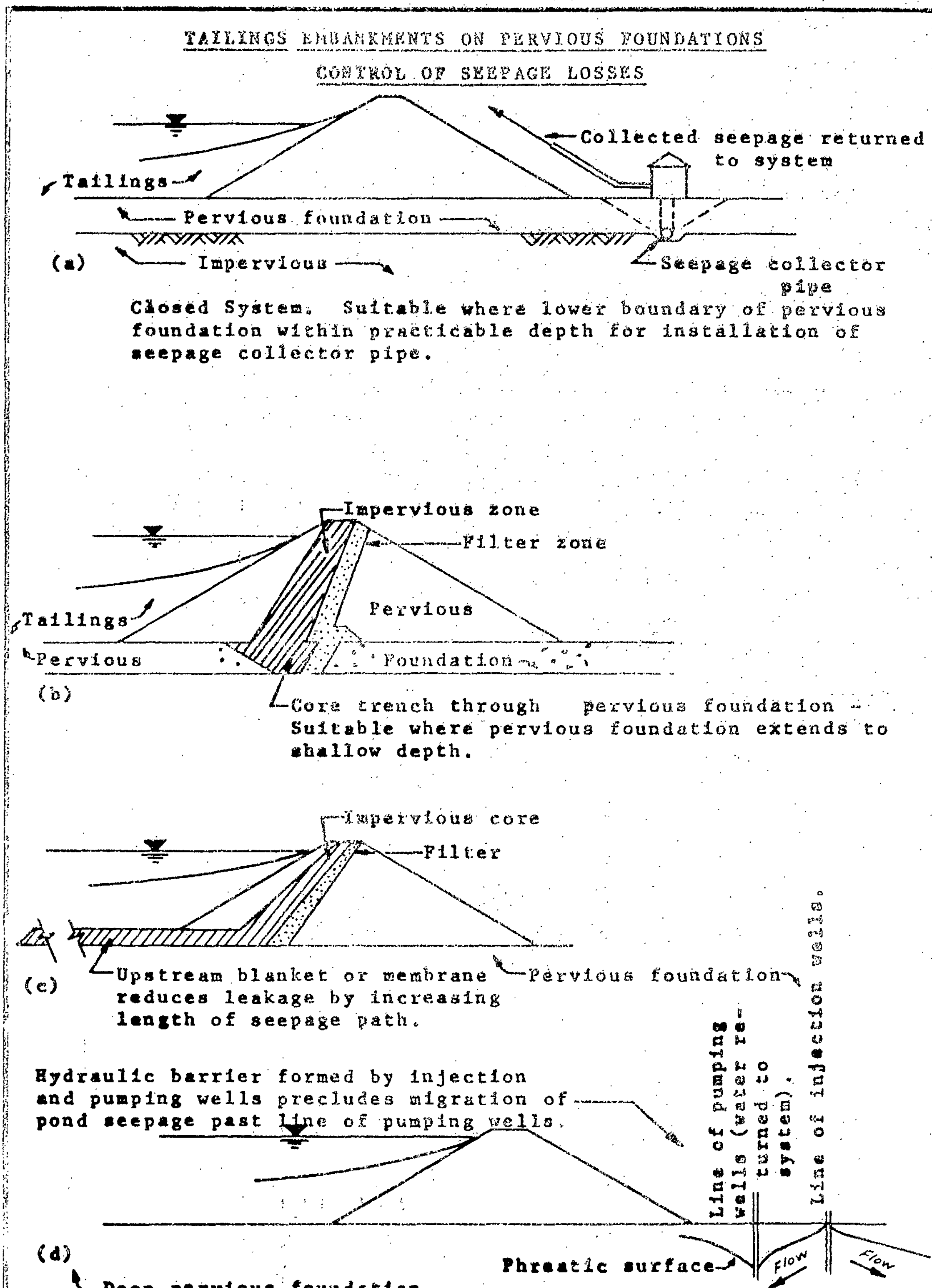
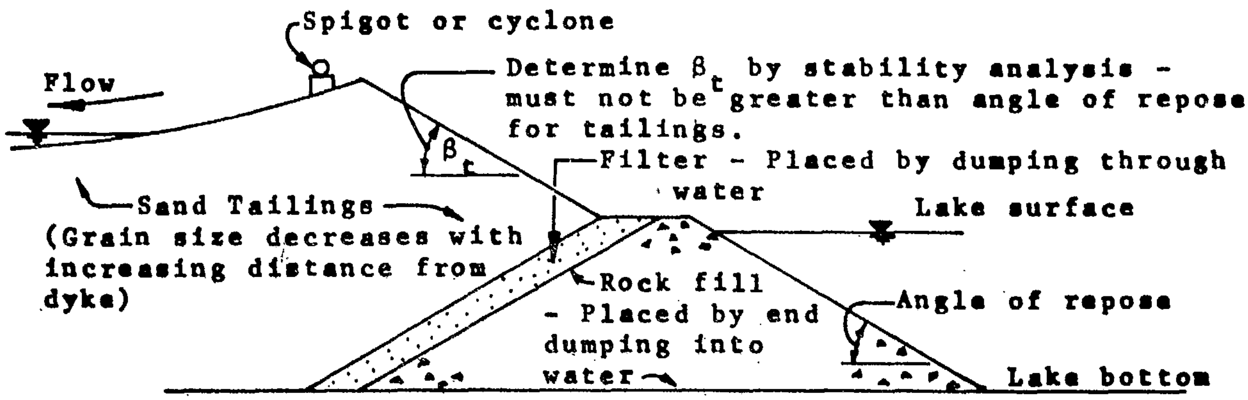
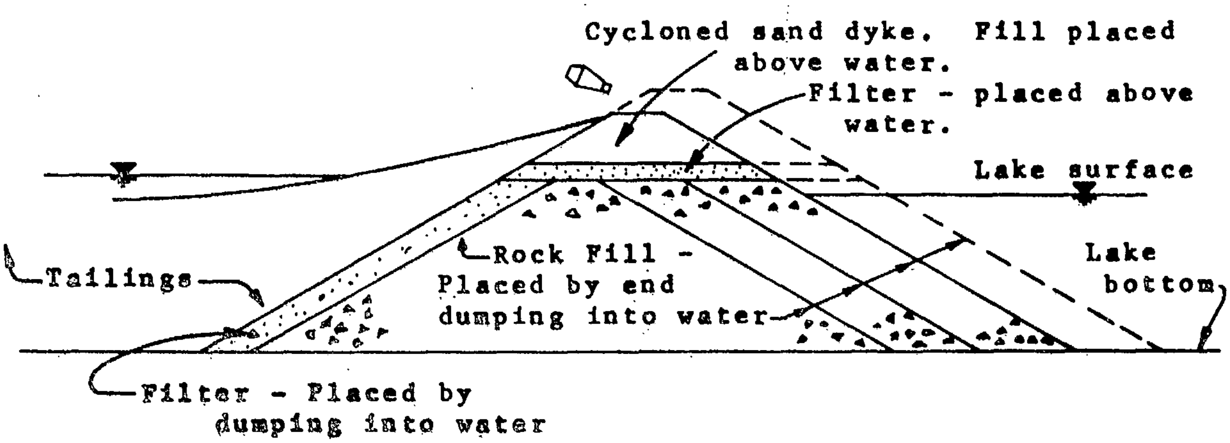

TAILINGS EMBANKMENTS ON PERVIOUS FOUNDATIONS
CONTROL OF SEEPAGE LOSSES
Collected seepage returned to system
Tailings
Pervious foundation
Impervious
(a)
Seepage collector pipe
Closed System. Suitable where lower boundary of pervious foundation within practicable depth for installation of seepage collector pipe.
Impervious zone
Filter zone
Pervious
Tailings
Pervious
Foundation
(b)
Core trench through pervious foundation - Suitable where pervious foundation extends to shallow depth.
Impervious core
Filter
(c)
Upstream blanket or membrane reduces leakage by increasing length of seepage path.
Pervious foundation
Line of injection wells.
Line of pumping wells (water re-turned to system).
Hydraulic barrier formed by injection and pumping wells precludes migration of pond seepage past line of pumping wells.
(d)
Phreatic surface
Flow
Flow
Deep pervious foundation

FIGURE 5-3

TAILINGS DAMS CONSTRUCTED IN WATER



Tailings Dam Constructed in Water - Upstream Method

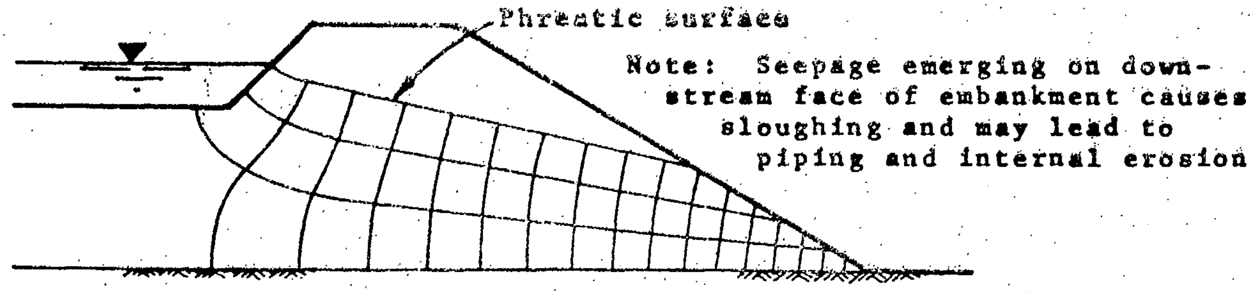


Tailings Dam Constructed in Water - Downstream Method

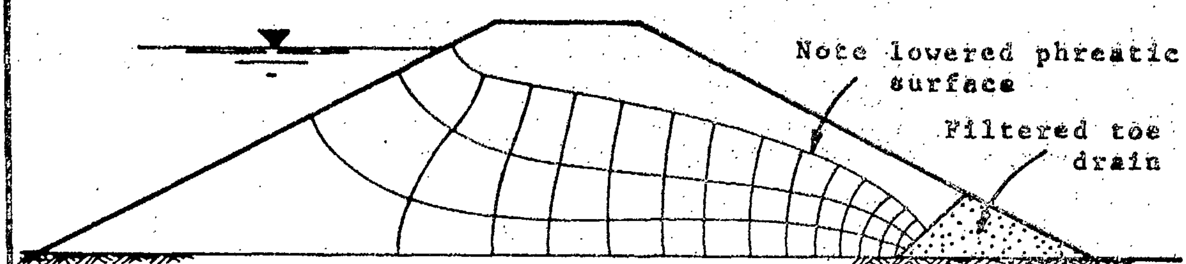
(Suitable where large quantities of rock fill readily available)

FIGURE 5-4

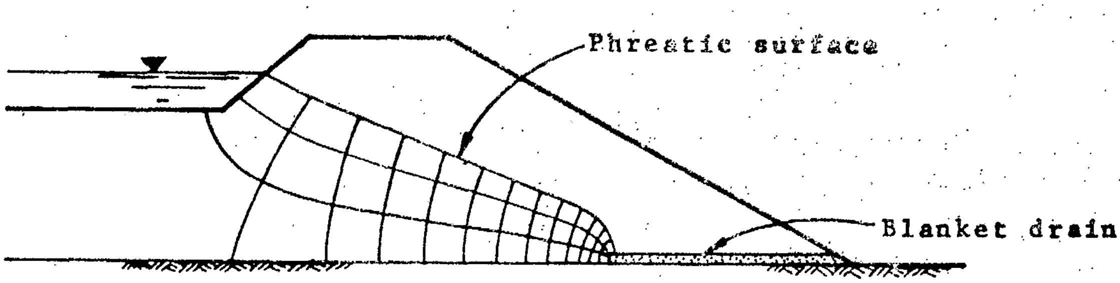
FLOW NETS FOR EMBANKMENTS ON IMPERVIOUS FOUNDATION



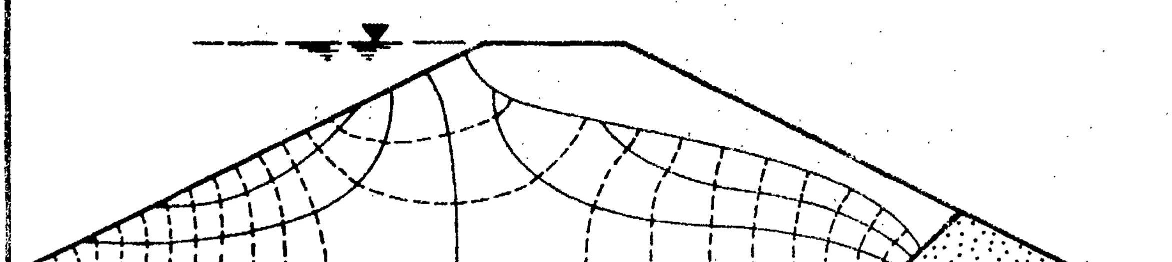
Homogeneous Section



Homogeneous Section with Toe Drain



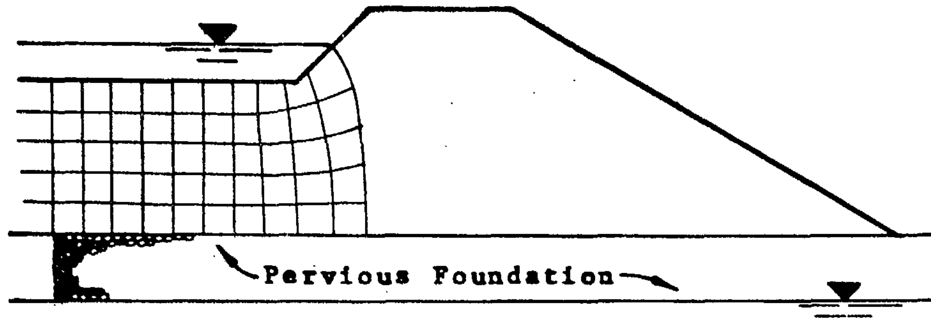
Homogeneous Section with Blanket Toe Drain



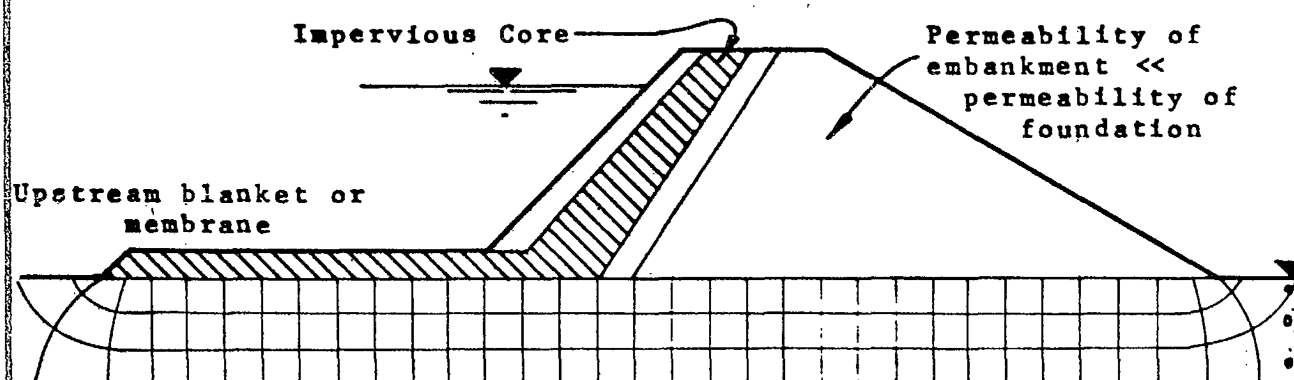
Flow Net for Rapid Drawdown

FIGURE 5-5

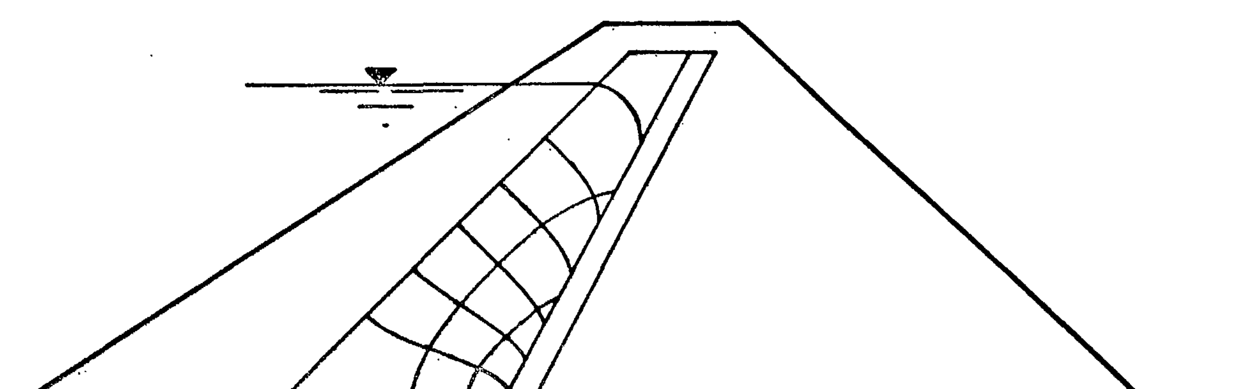
EXAMPLES OF FLOW NETS



Flow Net for Tailings Embankment and Pond Underlain by Pervious Foundation

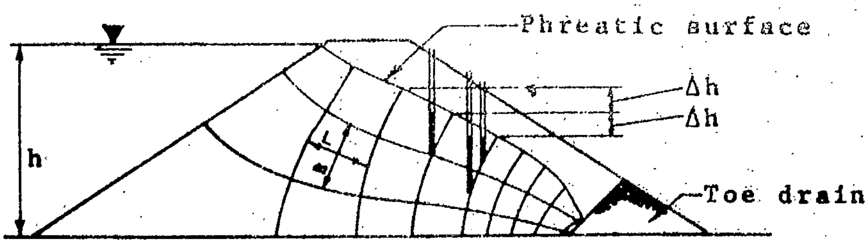


Flow Net for Seepage through Pervious Foundation -Upstream Blanket or Membrane used Increase Length of Seepage Path

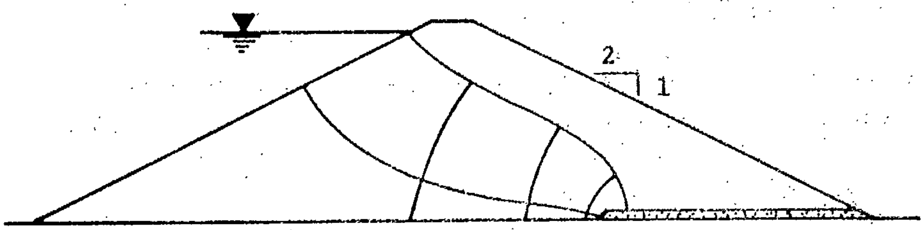


Approximate Flow Net for Seepage through Impervious Core

FIGURE 5-6

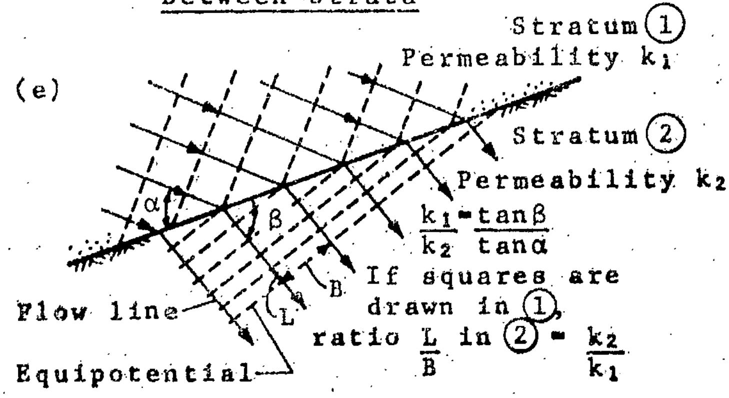


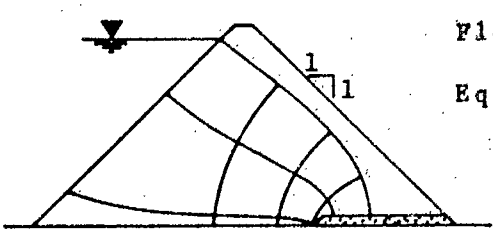
(a) $k_h = k_v$



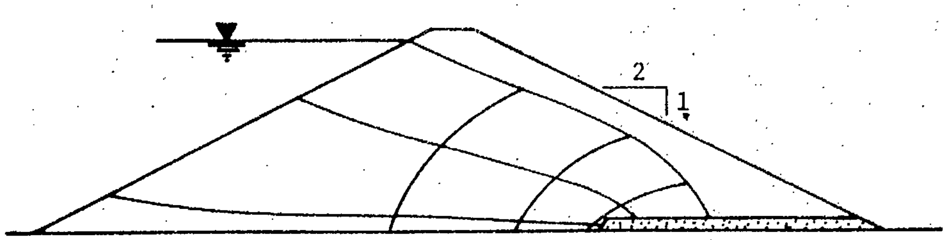
(b) Flow net for true section where $k_h = k_v$

Transfer Conditions at Interface Between Strata





(c) Flow net for section (b) above if $k_h = 4k_v$
Horizontal dimensions of section reduced by $\sqrt{\frac{k_v}{k_h}}$



(d) Flow net (c) above transposed to true section

Rules for Flow Net Construction

1. When materials are isotropic with respect to permeability, the pattern of flow lines and equal potential lines intersect at right angles. Draw a pattern in which the flow lines and equal potential lines form "square" figures.

2. Usually it is expedient to start with an integral number of equal potential drops, by dividing total head by a whole number, and drawing flow lines to conform to these equal potentials. In the general case, the outer flow path may form distorted square figures. The shape of these distorted squares (the ratio B/L) must be constant.

3. The upper boundary of a flow net which is at atmosphere pressure is the "phreatic surface". Integral numbers of equal potentials intersect the phreatic surface at points spaced at equal vertical intervals.

4. A discharge face through which seepage passes is an equal potential line if the discharge is submerged, or a free water surface if the discharge is not submerged. If it is a free water surface, the flow net figures adjoining the discharge face will not be squares.

5. In a stratified soil profile where the ratio of permeability of the layers exceeds 10, the flow in the more permeable layer controls. That is, the flow net may be drawn for the more permeable layer assuming the less permeable layer is impervious. The head on the interface thus obtained is imposed on the less pervious layer for construction of the flow net within that layer.

6. In a stratified soil profile where the ratio of permeability of the layers is less than 10, the flow net is deflected at the base in accordance with the diagram (e) at left.

7. When materials are anisotropic with respect to permeability, the cross section should be transformed by changing the scale as indicated by (c) at left. The flow net is then drawn as for isotropic materials. In computing the quantity of seepage, the differential head is not altered for the transformation.

8. Where only the quantity of seepage is to be determined, an approximate flow net suffices. Where pore pressures are to be determined, the flow net must be accurate.

FLOW NET
CONSTRUCTION

FIGURE 5-7

GRADATION REQUIREMENTS
FOR FILTERS

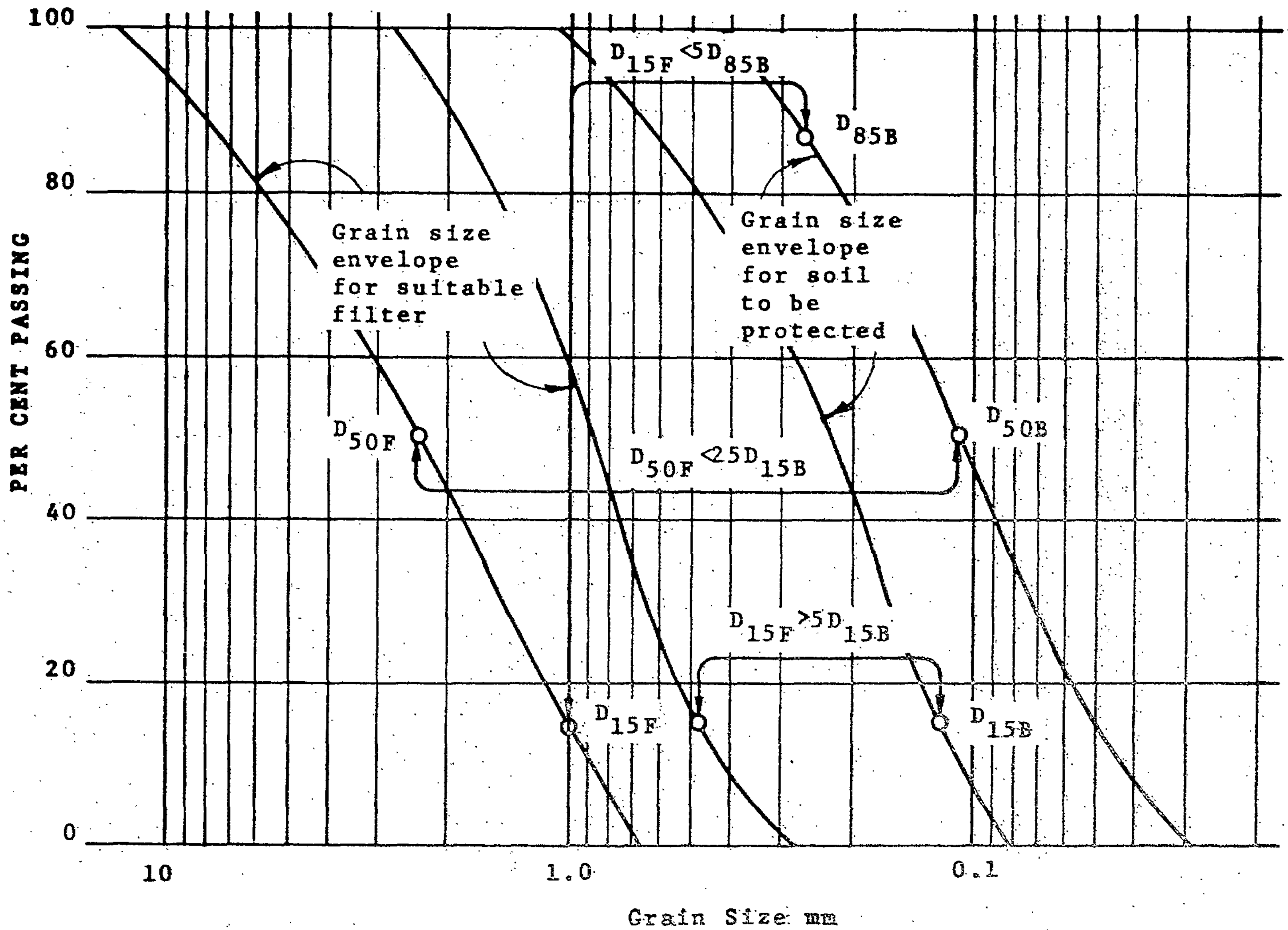


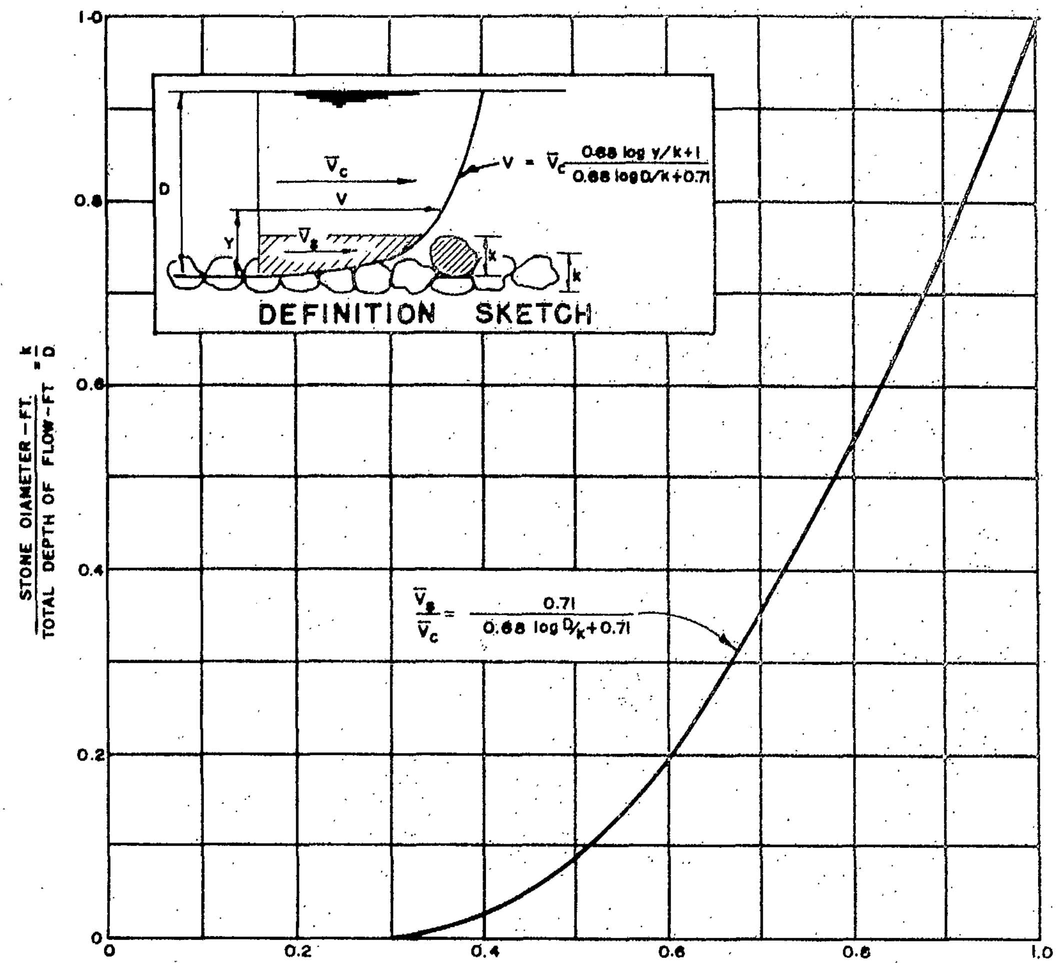
FIGURE 5-8

Original material excavated	Clear water no detritus (f.p.s.)	Water transporting colloidal silts (f.p.s.)	Water transporting non-colloidal silts, sands, gravel or rock fragments (f.p.s.)
Fine sand, non-colloidal	1.50 ...	2.50	1.50
Sand loam, non-colloidal	1.75 ...	2.50	2.00
Silt loam, non-colloidal	2.00 ...	3.00	2.00
Alluvial silts, non-colloidal	2.00 ...	3.50	2.00
Ordinary firm loam	2.50 ...	3.50	2.25
Volcanic ash	2.50 ...	3.50	2.00
Fine gravel	2.50 ...	5.00	3.75
Stiff clay, very colloidal	3.75 ...	5.00	3.00
Graded, loam to cobbles, non-colloidal	3.75 ...	5.00	5.00
Alluvial silts, colloidal	3.75 ...	5.00	3.00
Graded, silt to cobbles, colloidal	4.00 ...	5.50	5.00
Coarse gravel, non-colloidal	4.00 ...	6.00	6.50
Cobbles and shingles	5.00 ...	5.50	6.50
Shales and hardpans	6.00 ...	6.00	5.00

Note: These velocities are applicable to waterways on mild slopes and with long tangents. (King and Brater)

MAXIMUM PERMISSIBLE VELOCITIES
IN UNLINED WATERWAYS

FIGURE 5-9



$$\frac{\text{Average Velocity Against Stone-FPS}}{\text{Average Velocity in Channel -FPS}} = \frac{V_s}{\overline{V}_c}$$

NOTES:

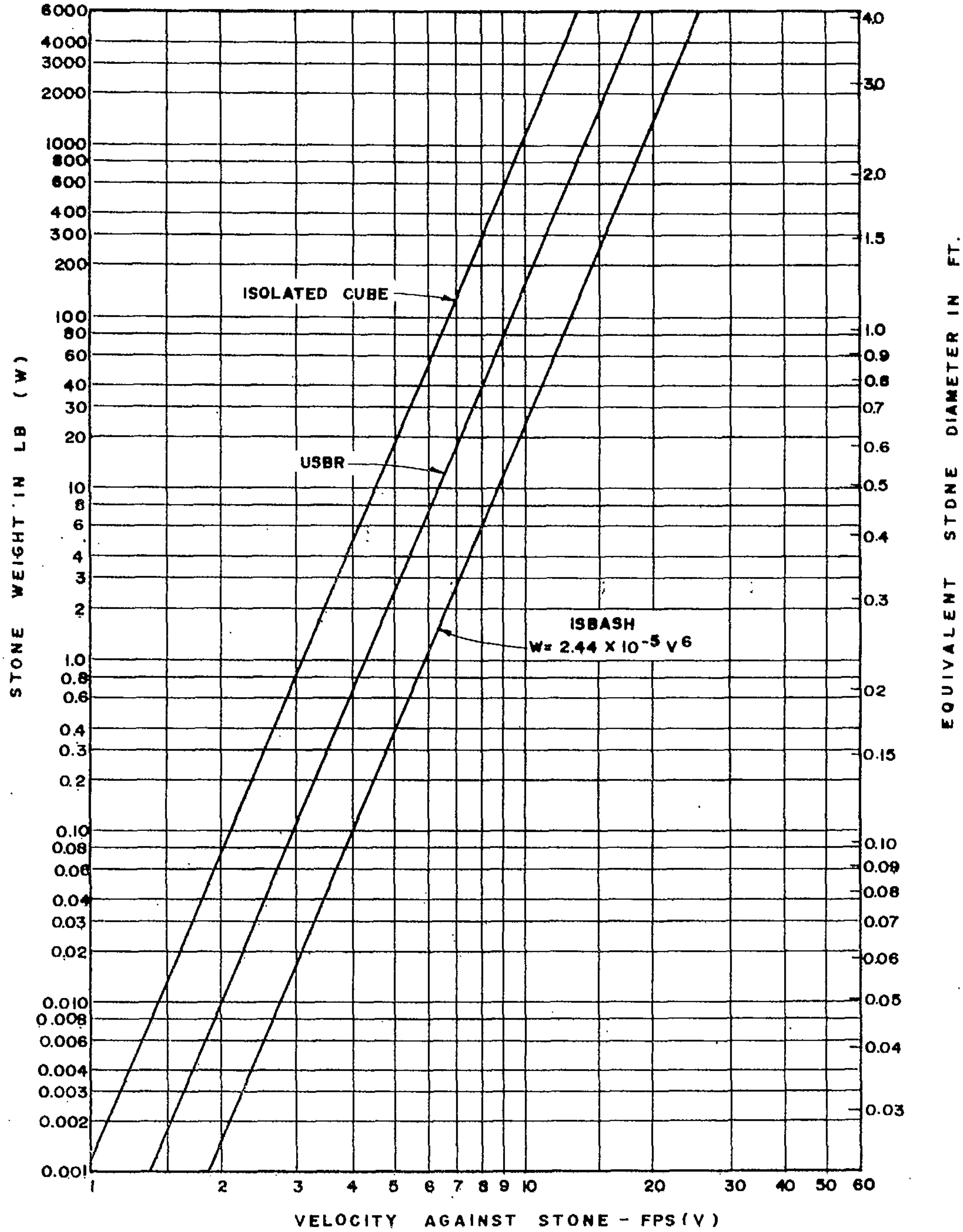
Equations apply to two-dimensional flow

$\overline{V}_s$ = Average velocity against stone - FPS
$\overline{V}_c$ = Average velocity in channel - FPS
D = Total depth of flow - FT
k = Stone diameter - FT

Equation developed from velocity distribution over rough boundary given in Engineering Hydraulics, edited by Rouse, 1950, Wiley and Sons. (U.S. Army Corps of Engineers)

STONE PAVING
AVERAGE VELOCITY AGAINST
STONE ON CHANNEL BOTTOM

FIGURE 5-10



NOTE :
Specific weight of rock
= 165 lb. / cu. ft.

(U.S. Army Corps of Engineers)

STONE PAVING
VELOCITY VS. STONE WEIGHT
FIGURE 5-11

Fetch, miles	Wind velocity, miles per hour	Wave height, feet
1	50	2.7
1	75	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1

APPROXIMATE WAVE HEIGHTS

Fetch, miles	Normal freeboard feet	Minimum freeboard feet
Less than 1	4	3
1	5	4
2.5	6	5
5	8	6
10	10	7

FREEBOARD REQUIRED FOR WAVE ACTION

Reservoir fetch, miles	Nominal thick-ness, inches	Gradation, percentage of stones of various weights (pounds)			
		Maxi-mum size	25 percent greater than	45 to 75 percent from-to	25 percent less than [1]
1 and less	18	1,000	300	10- 300	10
2.5	24	1,500	600	30- 600	30
5	30	2,500	1,000	50-1,000	50
10	36	5,000	2,000	100-2,000	100

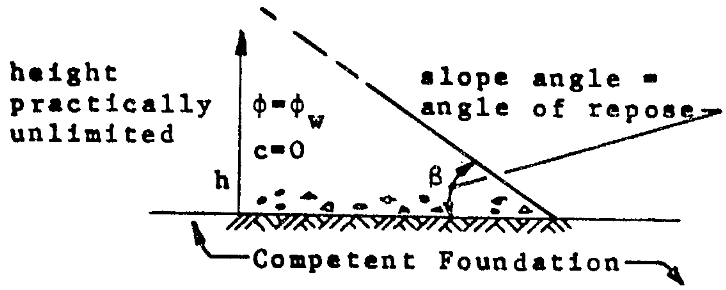
1 Sand and rock dust less than 5 percent

RIPRAP REQUIRED ON 3:1 SLOPES
FOR PROTECTION AGAINST WAVES

(U.S. Bureau of Reclamation)

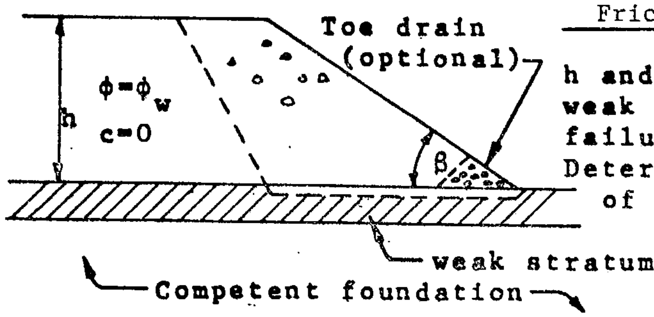
EMBANKMENT FREEBOARD
AND WAVE PROTECTION
FIGURE 5-12

PILES OF FREE DRAINING, FRICTIONAL WASTES



Coarse Frictional Waste on Competent Foundation

Pile can be constructed to any height providing $\beta \leq$ angle of repose.

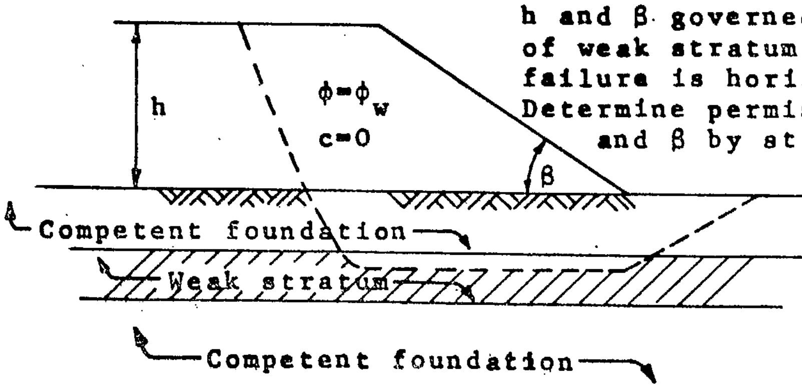


Frictional Waste on Shallow Weak Foundation

h and β governed by strength of weak stratum. Probable mode of failure is horizontal translation. Determine permissible combinations of h and β by stability analysis.

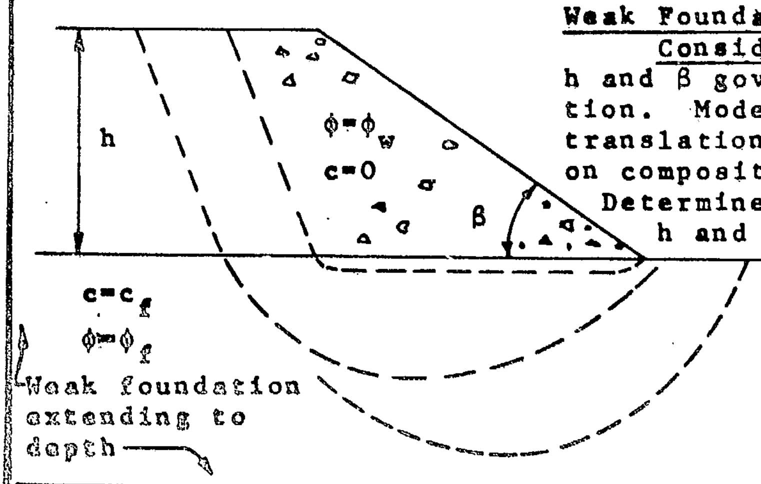
Weak Stratum at Shallow Depth

h and β governed by depth and strength of weak stratum. Probable mode of failure is horizontal translation. Determine permissible combinations of h and β by stability analysis.



Weak Foundation Extending to Considerable Depth

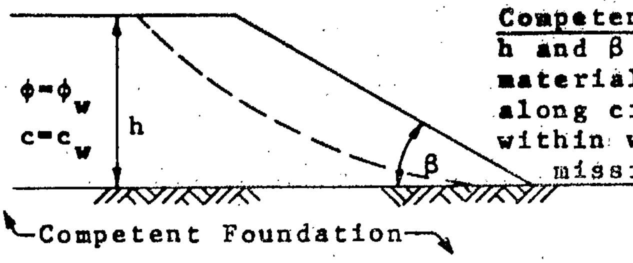
h and β governed by strength of foundation. Mode of failure may be horizontal translation or deep seated shear failure on composite surface of sliding. Determine permissible combinations of h and β by stability analysis.



DETERMINATION OF PERMISSIBLE HEIGHTS AND SLOPES

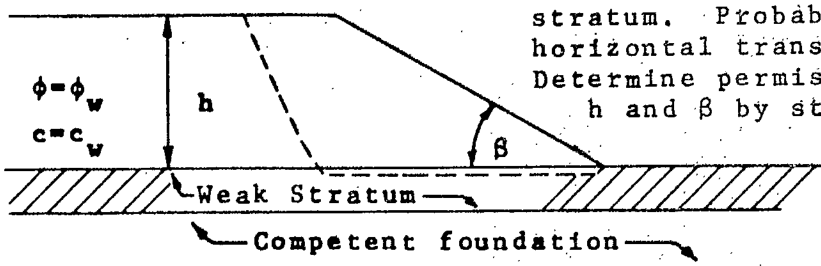
FIGURE 5-13

COHESIVE OR DEGRADING WASTE PILES



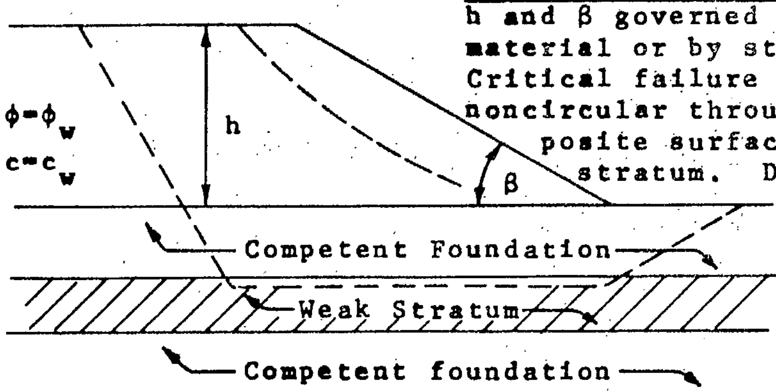
Competent foundation

h and β governed by strength of waste material. Probable mode of failure is along circular or noncircular surface within waste material. Determine permissible combinations of h and β by stability analysis.



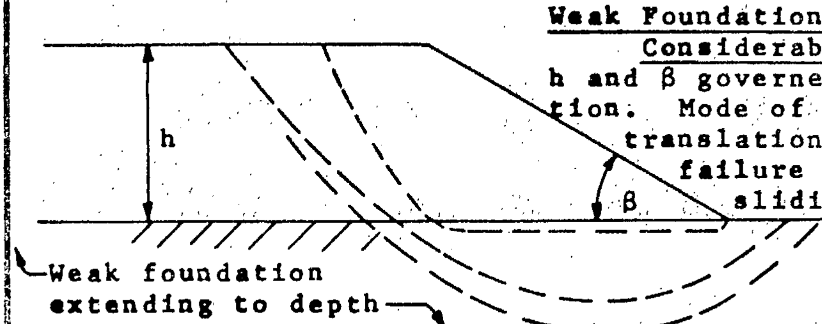
Shallow Weak Foundation

h and β governed by strength of weak stratum. Probable mode of failure is horizontal translation (spreading). Determine permissible combinations of h and β by stability analysis.



Weak Stratum at Shallow Depth

h and β governed by strength of waste material or by strength of weak stratum. Critical failure surface may be circular or noncircular through waste, or may be composite surface passing through weak stratum. Determine permissible combinations of h and β by stability analysis.

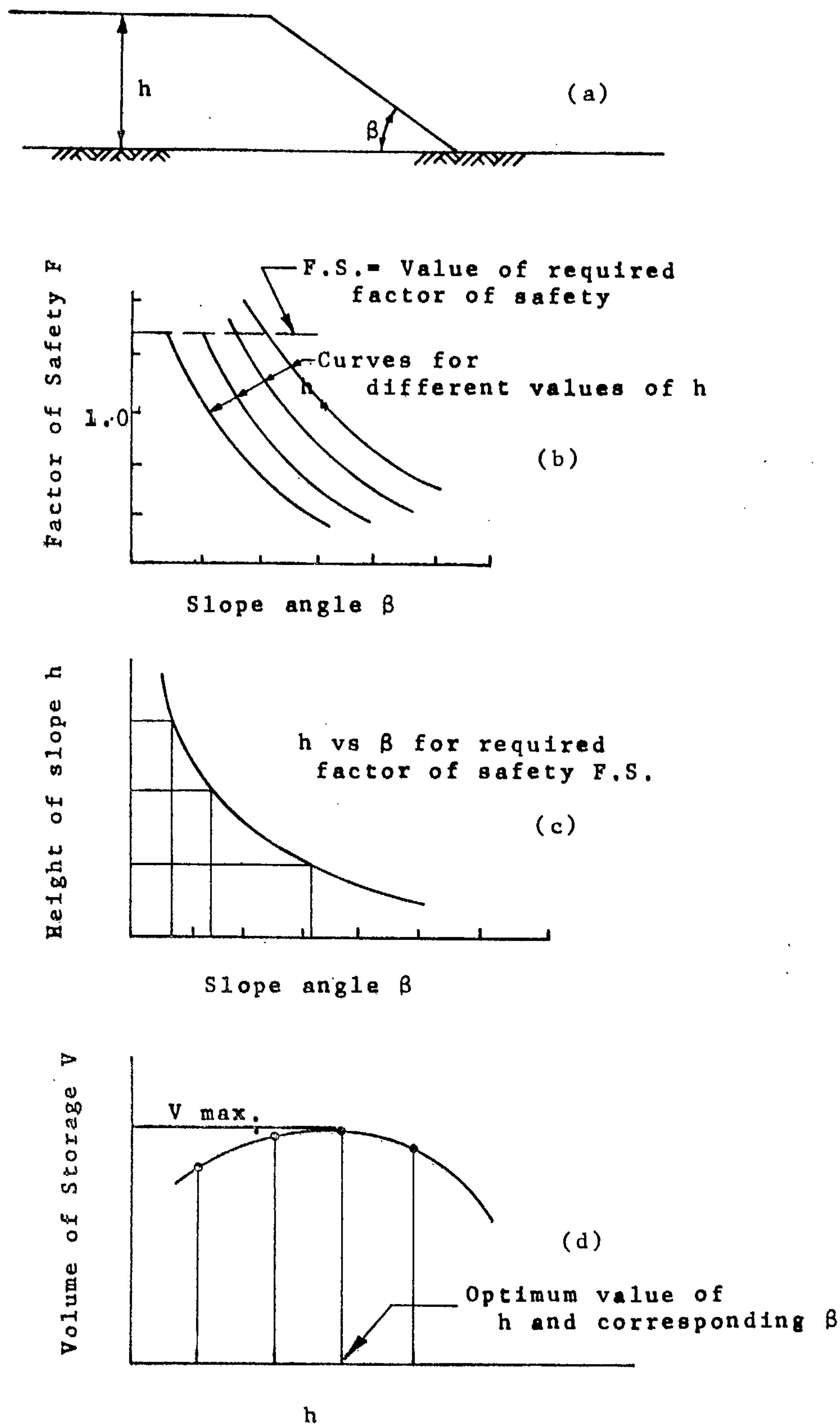


Weak Foundation Extending to Considerable Depth

h and β governed by strength of foundation. Mode of failure may be horizontal translation, or deep seated shear failure on composite surface of sliding. Determine permissible combinations of h and β by stability analysis.

DETERMINATION OF PERMISSIBLE HEIGHTS AND SLOPES

FIGURE 5-14


h
β
(a)
Factor of Safety F
1.0
F.S.= Value of required factor of safety
Curves for different values of h
h
(b)
Slope angle β
Height of slope h
h vs β for required factor of safety F.S.
(c)
Slope angle β
Volume of Storage V
V max.
(d)
Optimum value of h and corresponding β
h

5-15

DESIGN OF WASTE PILES

1. If the waste pile is to be constructed on a sloping foundation, check the stability of the pile with respect to shearing along the base of the pile.

2. Choose a trial height 'h' for the waste pile and a trial slope angle β. Calculate the factor of safety F, using stability analysis.

3. Keeping h constant, choose new values for the slope angle β, and recalculate the factor of safety for each trial value of β.

4. Repeat steps 2 and 3 above for different values of h.

5. Plot the results of the calculations obtained from steps 2, 3, and 4 above in the form of a graph of F vs. β. This plot is illustrated on (b) at left.

6. Select the desired factor of safety, F.S., for the waste pile.

7. From the graph of F vs. β, (Fig. b), select values of h and β corresponding to the desired factor of safety F.S. Plot these data in the form of a graph of h vs. β, (corresponding to the required factor of safety, F.S., Fig. c).

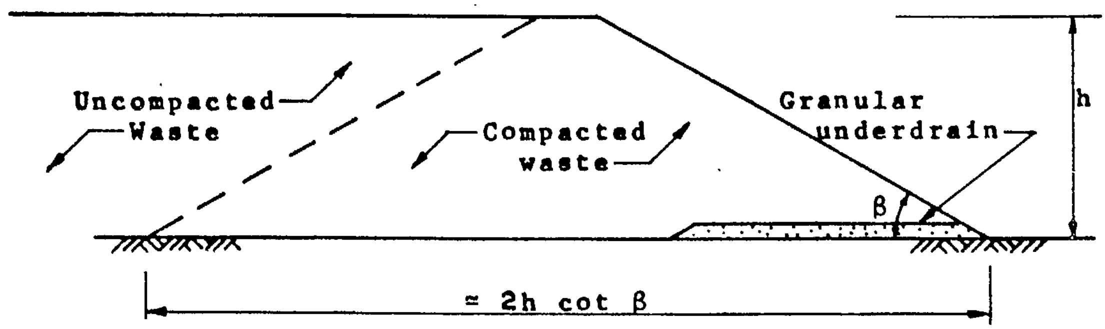
   If the maximum volume of available storage within the available area is required, proceed with step 8.

8. Calculate the volume of available storage using combinations of h and β selected from the graph (c). By the process of bracketing, the optimum combination of h and β can be found that provides for maximum storage within the area available, and provides the required factor of safety. (d)

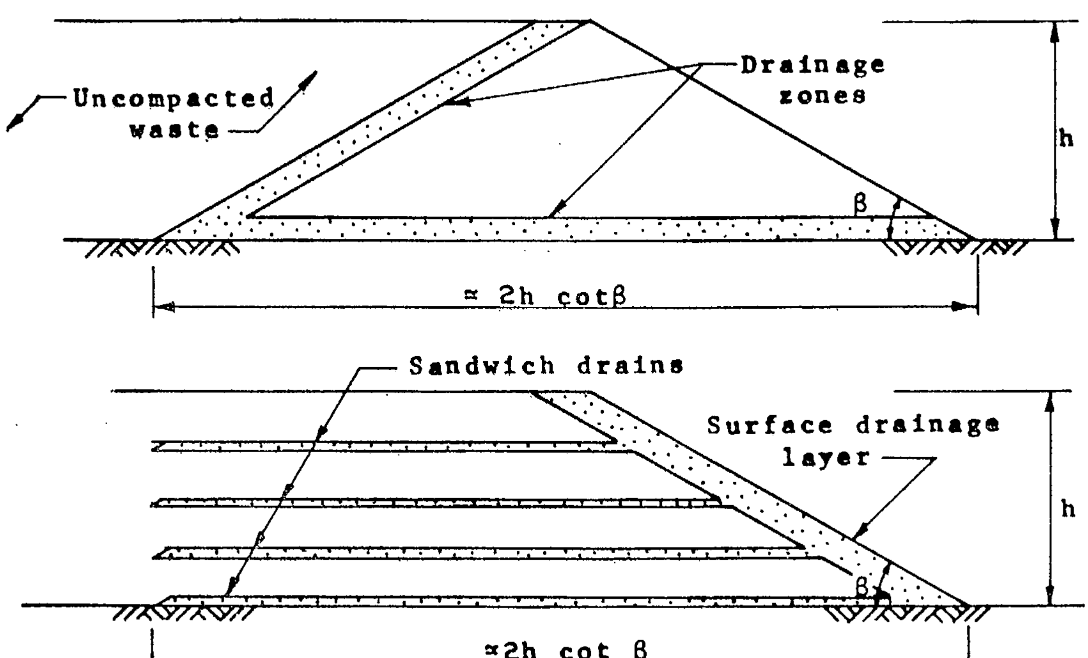
DESIGN PROCEDURE
FOR WASTE PILES

FIGURE 5-15

PERIMETER ZONES FOR WASTE PILES

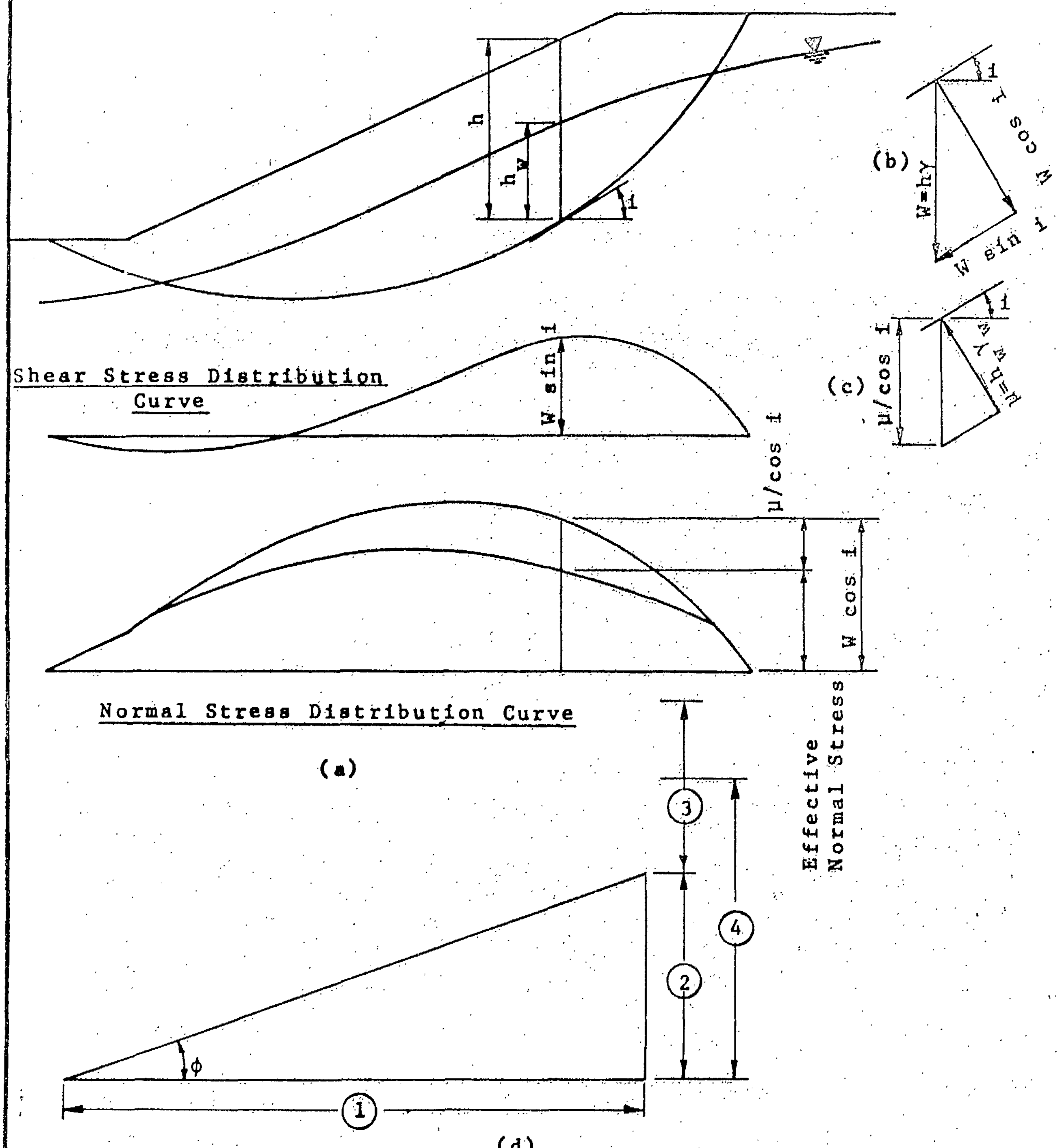


Compacted perimeter zone serves as a dyke to retain lower strength, uncompacted material in the central portions of the waste pile



Granular drainage zones maintain drained conditions within the perimeter of the pile. Perimeter zone acts as a dyke to retain lower strength materials within central portions of the waste pile.

FIGURE 5-16



(1) = The total effective normal force on the trial failure surface.

(2) = The available shear resistance due to friction.

(3) = The available shear resistance due to cohesion.

(4) = The total shearing force on the trial failure surface.

$$FS = \frac{(2) + (3)}{(4)}$$

PROCEDURE

1. Select trial failure surface.
2. Draw a number of approximately equally spaced vertical lines extending from trial failure surface to ground surface.
3. At each vertical line, determine the vertical stress at the failure surface due to the weight of the column of soil and water above surface.
4. Resolve the vertical stress into components normal and tangential to the trial failure surface.
5. Plot the tangential stress components as ordinates, and join plotted points to obtain the shear stress distribution curve. The area under the curve equals the total of the shearing forces along the trial failure surface.
6. Plot the normal stress components as ordinates, and join plotted points to form the total normal stress distribution curve.
7. Draw flow net to estimate neutral stress μ, (water pressure) along trial failure surface. Determine values of $\frac{\mu}{\cos i}$. A graphical method of determining $\frac{\mu}{\cos i}$ is indicated by Fig. (c).
8. Plot values of $\frac{\mu}{\cos i}$ below the total normal stress curve. The area of the hachured portion of the normal stresses distribution curve equals the total effective normal force on the trial failure surface.
9. The factor of safety can be determined graphically as indicated on (d).

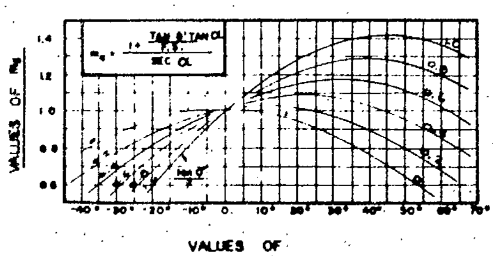
STABILITY ANALYSIS
BY METHOD OF
INFINITE SLICES

FIGURE 5 - 17

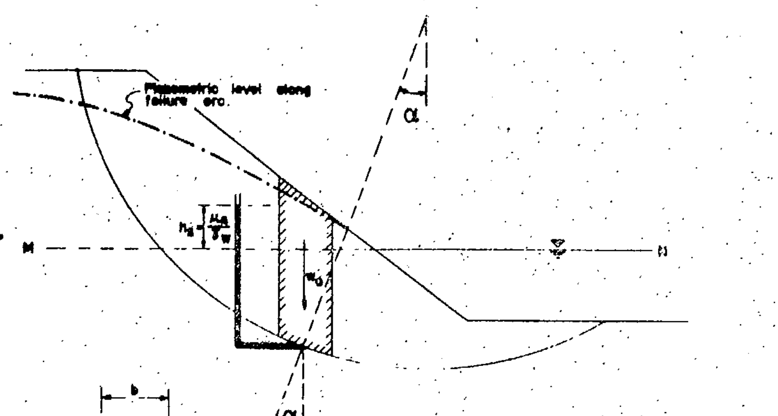
SLICE NO	OVERTURNING ELEMENTS (1)							COHESIVE ELEMENTS (2)			FRICTIONAL ELEMENTS (3)						(4)	1st Trial FS= Tan φ'/FS=		2nd Trial FS= Tan φ'/FS=	
	W OVERTURNING = W_o				α	Sin α	W_o Sin α	b	c'	c'b	h_s	$u_s = h_s\gamma_w$	$b u_s$	$W_o - b u_s$	Tan φ'	$(W_o - b u_s)$ Tan φ'	(2)+(3)	m_α	$(5) = \frac{(4)}{m_\alpha}$	m_α	$(5) = \frac{4}{m_\alpha}$
	γ	AREA																			

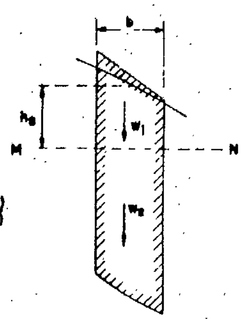
Σ(1) =

Σ(5)



NOTE α positive when failure arc slope is in the same quadrant as slope of ground.





NOTES:
1. $w_o = w_1 + w_2$
2. 'w_1' is the weight of that portion of a slice above line MN and is computed from total unit weights.
3. 'w_2' is the weight of that portion of a slice below line MN and is computed from submerged unit weights.
4. 'h_s' refers only to the height of the piezometric level above line MN. However in the case of slopes with no partial submergence, 'h' would be measured from the failure arc.

AFTER A.W. BISHOP (GEOTECHNIQUE, MARCH 1955).

$$F.S. = \frac{1}{\Sigma(w_o \sin\alpha)} \Sigma\left[\left\{c'b + (w_o - b u_s)\tan\phi'\right\}\frac{1}{m_\alpha}\right] = \frac{\Sigma(5)}{\Sigma(1)}$$

TRIAL (1)

$F.S. = \frac{\Sigma(5)}{\Sigma(1)}$

= ———

=

TRIAL (2)

$F.S. = \frac{\Sigma(5)}{\Sigma(1)}$

= ———

=

FINAL F.S =

TABULATED SOIL PROPERTIES			
γ			
γ_b			
φ'			
c'			

PROCEDURE

i. Select a trial circular arc failure surface.

ii. Divide the mass above the trial failure surface into approximately 10 vertical slices.

iii. Determine the weight (W_o) of each slice.

iv. Determine α, the angle of inclination of the trial failure surface at the base of each slice.

v. Compute W_o Sin α, the overturning element (1 on the table).

vi. From the width of the slice b, and the average cohesion c' along the base of the slice, compute the cohesive element (2 on the table).

vii. Calculate the pore pressure and the neutral force at the base of each slice. The pore pressure = the weight of water times the average piezometric head above the base of the slice ($\mu_s = h_s \gamma_w$), and the neutral force = the pore pressure times the width of the slice ($b\mu_s$).

viii. Compute the frictional element ($W_o - b\mu_s$) tan ϕ' where ϕ' is the effective angle of internal friction along the base of the slice.

ix. Assume a factor of safety F, and determine m_a for each slice using the graph. Calculate a new factor of safety (5) = $\frac{(2) + (3)}{m_a}$. The first approximation of the factor of safety then equals $\frac{\Sigma(5)}{\Sigma(1)}$.

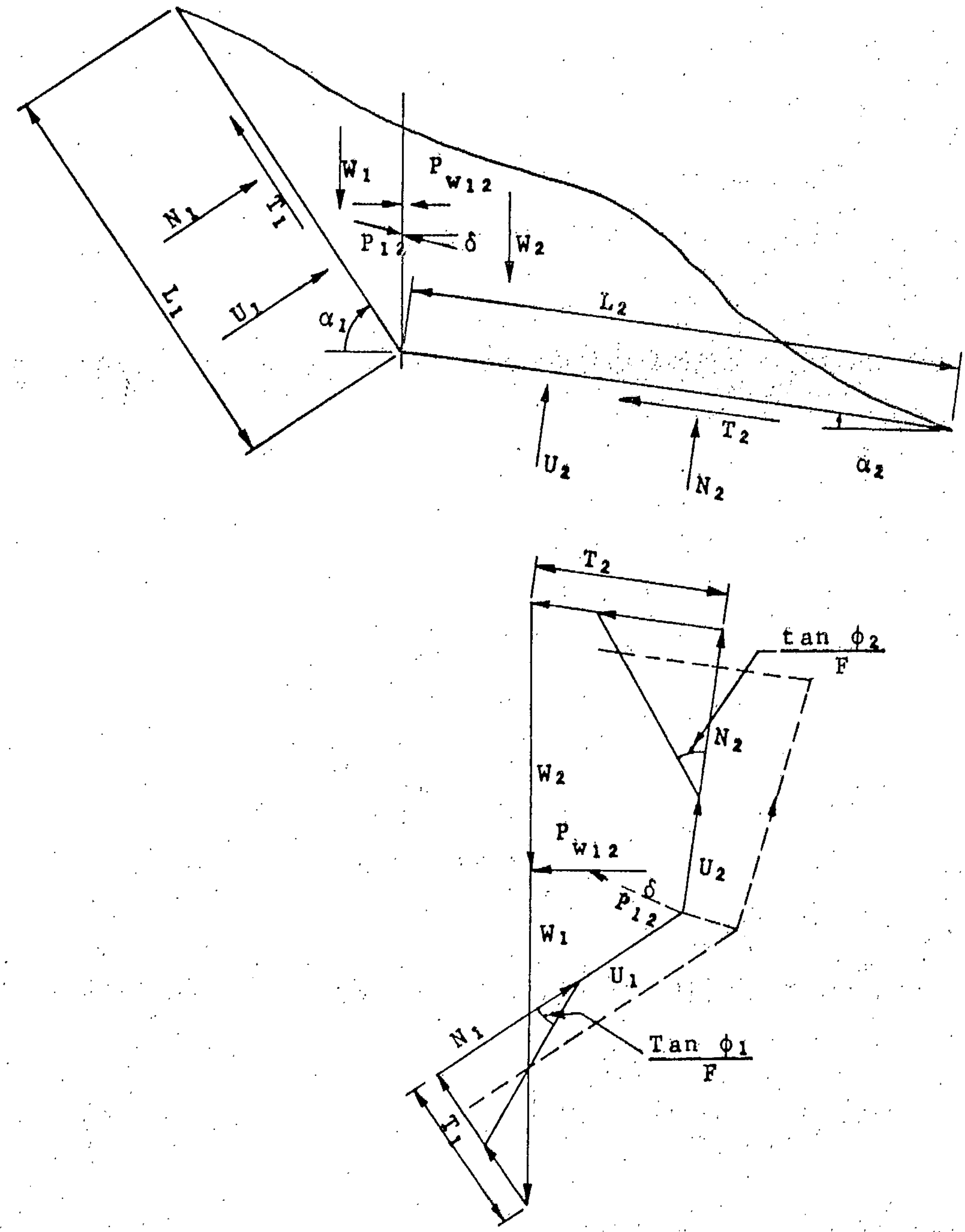
x. Using the factor of safety determined by step (ix) above, repeat step (ix) to obtain the second approximation of the factor of safety.

xi. Repeat (x) until a factor of safety F is obtained which is equal to the value of F used to determine m_a.

$$F.S. = \frac{1}{\Sigma(W_o \sin\alpha)} \Sigma\left[\{c'b + (W_o - b\mu_s)\tan\phi'\}\frac{1}{m_a}\right] = \frac{\Sigma(5)}{\Sigma(1)}$$

STABILITY ANALYSIS BY SIMPLIFIED BISHOP METHOD

FIGURE 5-18



W_1, W_2	Weight of a wedge
U_1, U_2	Resultant water pressure along the base of the wedge
N_1, N_2	Effective normal force on base of wedge
T_1, T_2	Shear force along base of wedge
L_1, L_2	Length of base of wedge
$\alpha_1\ \alpha_2$	Inclination of the base to the horizontal
P_{w12}	Resultant hydrostatic force at interface
P_{12}	Effective force at the interface
δ	Inclination of P_{12} to the horizontal
ϕ_1, ϕ_2	Effective friction angle on base of wedge
c_1, c_2	Effective cohesion on base of wedge

5-19

PROCEDURE

1. Choose a trial failure surface consisting of two or more intersecting tangents.

2. Separate the sliding mass into segments bounded by vertical lines which pass through the points of intersection of the base tangents.

3. Assume that the effective stresses across the vertical boundaries between adjacent wedges are inclined at an assumed angle δ to the horizontal. Note that the neutral (water pressure) forces across the boundaries between adjacent wedges act horizontally.

4. Choose a trial factor of safety (F).

5. Construct the force polygon for the first wedge. This gives a value for the force P_{12} which satisfies equilibrium conditions for the first wedge.

6. Using the value of P_{12} as determined in step 5 above, construct the force polygon for the second wedge. If this force polygon does not close, proceed with step 7.

7. Choose different values of F and repeat steps 5 and 6 above until a value of F is found such that the total force polygon does close.

   If δ is assumed to be zero, the shearing stresses on the vertical boundaries between adjacent wedges will also be zero, and the factor of safety will be underestimated. The maximum possible value of δ is equal to the angle of internal friction for the material through which the vertical boundary passes. Assumptions that $\delta=\phi$ leads to an overestimation of the factor of safety. The assumption that $\delta=\frac{1}{3}\phi$ usually leads to reasonable results.

STABILITY ANALYSIS
FOR WEDGE FAILURE

FIGURE 5-19

STABILITY ANALYSIS - HORIZONTAL TRANSLATION

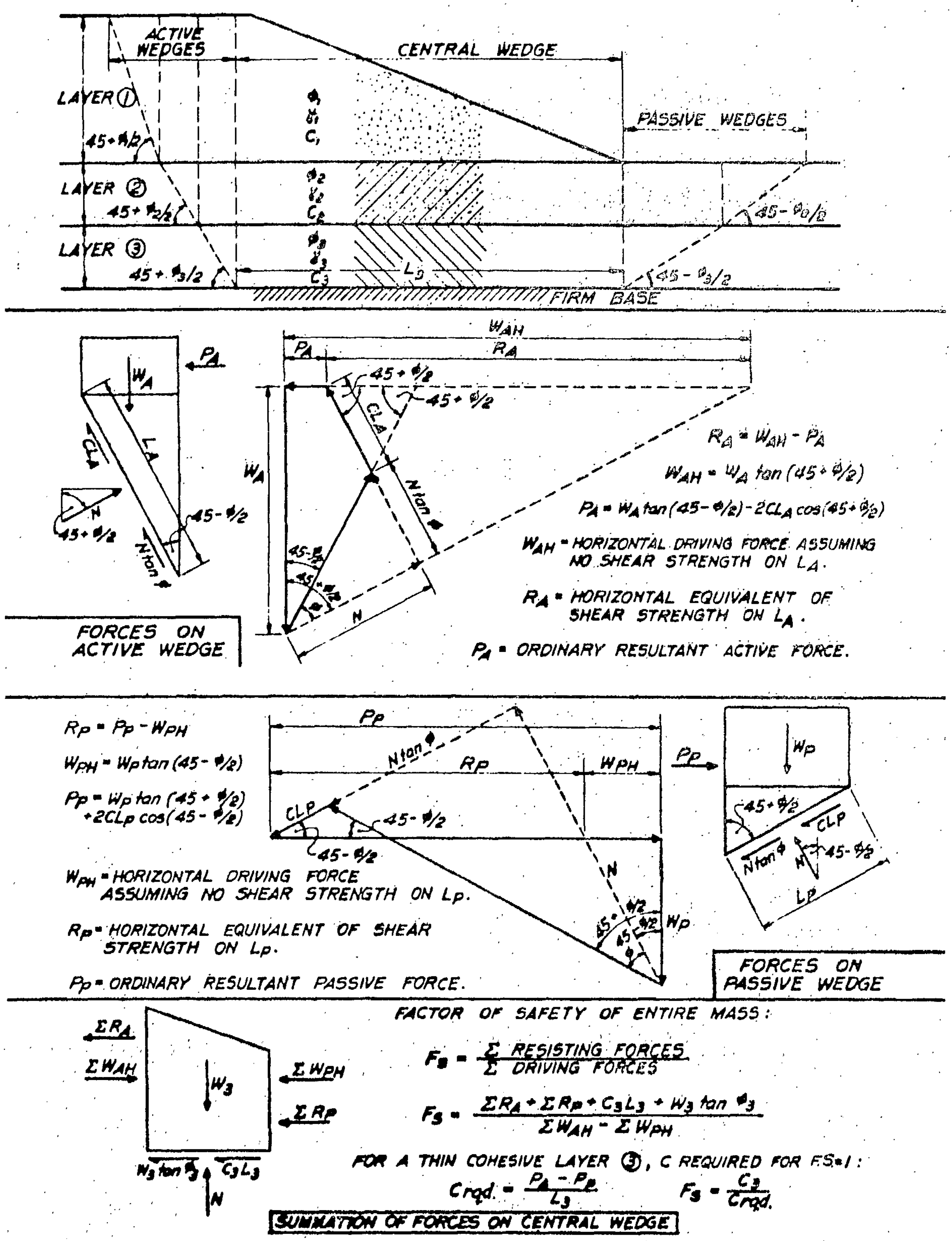
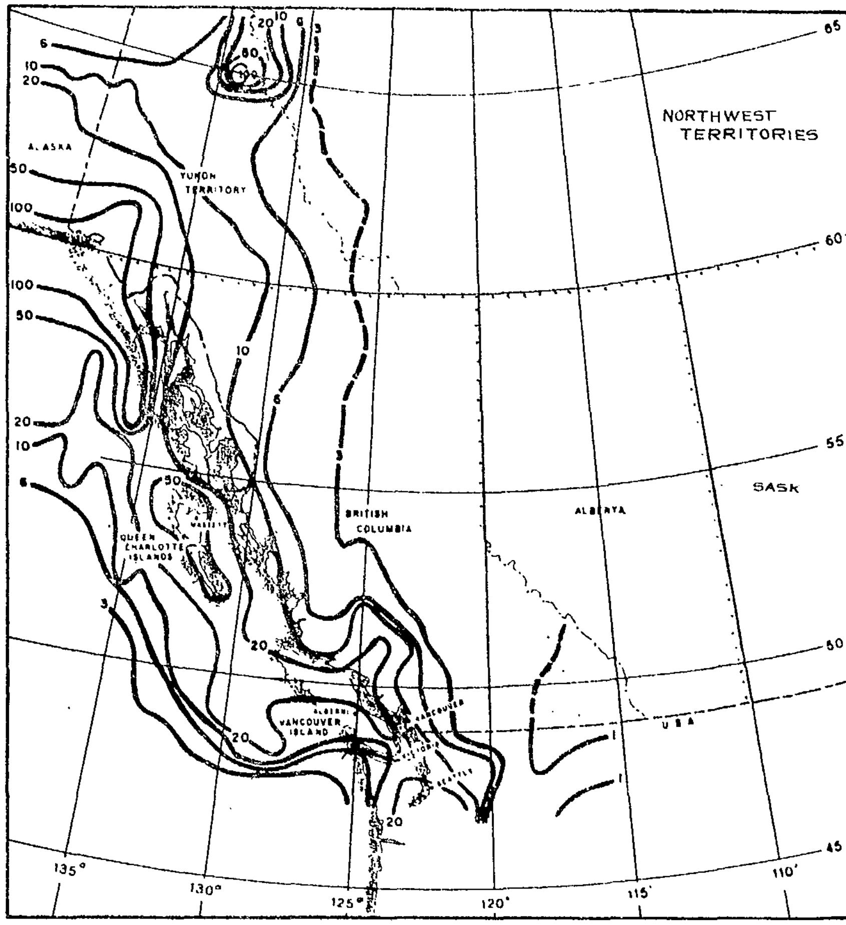
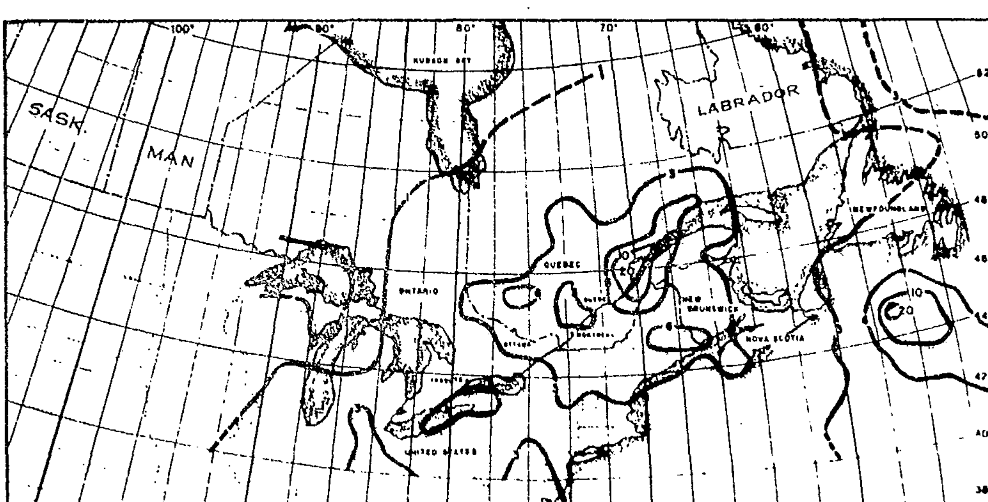


FIGURE 5-20

ONE HUNDRED YEAR RETURN PERIOD ACCELERATIONS AS A PER CENT OF g (Milne and Davenport, 1969).



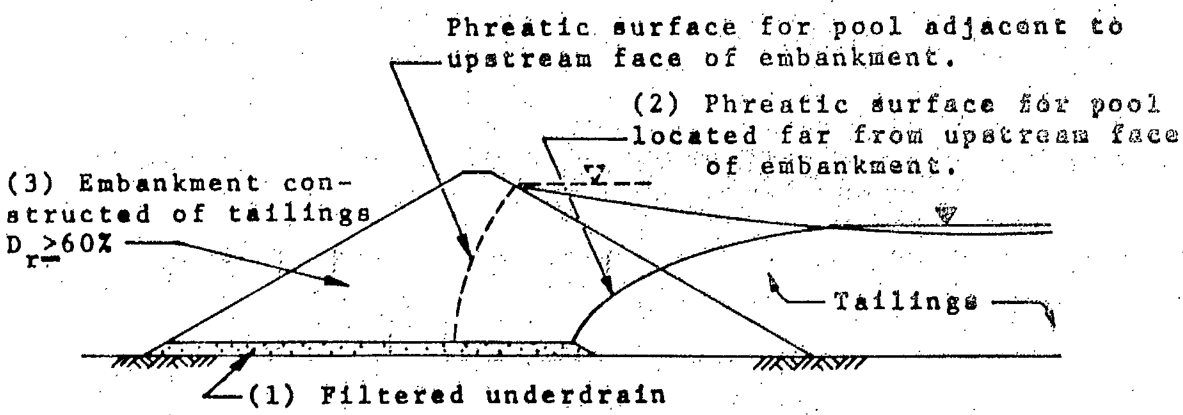
Western Canada



Eastern Canada

EARTHQUAKE ACCELERATIONS

FIGURE 5-21



Design Features to Guard Against Failure by Liquefaction

1. Provide filtered underdrain to lower phreatic surface in the embankment.

2. Maintain pool surface as far as practicable from the upstream face of the embankment to maintain low phreatic surface.

3. Compact embankment fill to achieve in situ relative density of 60 per cent or greater.

4. Mass of embankment should be large enough to provide required factor of safety against horizontal sliding along its base in the event that the shearing strength of the tailings is reduced to zero.

TAILING EMBANKMENTS
DESIGN GUARDS AGAINST
LIQUEFACTION FAILURES

FIGURE 5-22

SECTION 6

CONSTRUCTION AND OPERATION

CONTENTS | PAGE

FIGURES

SECTION 6

CONSTRUCTION AND OPERATION

TAILINGS EMBANKMENTS

Basic Construction Methods

Where a tailings embankment is constructed predominantly of sand recovered from the tailings slurry, one of three basic construction methods can be used. These three methods have been illustrated on Figure 5-1. They are commonly called:

the upstream method,

the downstream method,

the centreline method.

In the upstream method the crest of the embankment is raised by placing tailings sand in successive dykes located on the upstream side of an initial starter dyke. The initial starter dyke forms the downstream toe of the ultimate dam. Materials used in construction of the initial starter dyke should be pervious, relative to the tailings, and the gradation of the materials should conform to the requirements for filters with respect to the materials placed on the upstream side of the starter dyke.

In the downstream method, tailings sand is placed on the downstream side of the initial starter dyke. This dyke forms the upstream toe of the ultimate dam and should be impervious relative to the tailings sand.

In the centreline method, the crest of the embankment is maintailed in approximately the same horizontal position as the embankment is raised to its final height. The elevation of the crest of the dam is raised by placing successive layers of material on the crest and on the upstream and downstream fill slopes.

Influences on Design

The three basic construction methods described above lead to substantially different embankment cross-sections and produce different embankment material characteristics. Consequently, the embankment stability conditions are radically affected.

In the upstream method of construction, the stability of the final embankment is dependent, to a large degree, on the shear strength characteristics of the tailings deposited upstream of the embankment. The shear strength of the sedimented tailings is governed, in part, by the gradation and density of the solids, the consistency of the slurry and the distribution of the pore water pressures within the deposit after sedimentation. When initially deposited, the tailings have very low shear strength. The strength increases with time as drainage and consolidation takes place under the weight of overlying materials. Shear strength tests on representative samples of the tailings will give an indication of their effective angle of internal friction. However, the shear strength of the tailings also depends on the distribution of the pore water pressures within the deposit. Owing to large variations in permeability within the tailings, an accurate prediction of the distribution of pore water pressures cannot be made in advance of construction. Hence, the design of embankments constructed using the upstream method should be based on conservative assumptions regarding the rate of pore pressure dissipation within the tailings. Instrumentation is usually required to check that the pore pressures are not significantly higher than the pressures assumed during design.

For a given height and a given downstream fill slope, a tailings embankment constructed using the upstream method will generally have a lower factor of safety than a tailings embankment constructed using either the downstream or the centreline methods. If the stability of the structure is dependent on the shear strength of tailings that remain loose and saturated, the embankment may be subject to failure by liquefaction. Under these conditions, the method is generally unsuitable for the construction of tailings embankments located in areas subject to seismic activity. For high embankments (in the order of 100 feet or more in height), the method is generally unsuitable even in non-seismic areas.

In the downstream method, all of the embankment section lies outside the boundaries of the sedimented tailings. Materials incorporated in subsequent stages of the embankments may consist of the coarse fraction of the tailings separated by cycloning, or by gravity separation on the beach, coarse mine discard, or of rock or soil obtained from nearby borrow pits. The downstream method of construction permits controlled placement and compaction to achieve high shear strength characteristics, and permits the incorporation of drainage facilities to control the position of the phreatic water surface within the embankment. It is, therefore, inherently safer than the upstream method.

Sand Separation Methods

1. General

As shown in Section 3, raw tailings solids from ore processing can contain percentages of fines (-200 mesh) varying between 10 and 90 per cent by weight. Generally, it will not be practicable to construct a tailings embankment of these raw tailings by the downstream or centreline methods when they are on the fine side of this range. Even with the upstream method of construction, the possibilities of embankment instability will be greatly accentuated with such fine tailings. Usually it will be necessary to separate the coarser sand fractions from the raw tailings for placement in the embankment.

2. Spigots

Sand separation can be accomplished by several methods. With the upstream method of construction, raw tailings are often discharged into the pond through closely spaced spigots, from a header pipe installed on the embankment crest. The coarser sand fractions settle out on the beach formed just upstream of the crest. This method is illustrated on Figure 6-1. Successive embankment crests are raised by scraping sand from the beach by bulldozer or dragline.

3. Cyclones

Spigotting is unlikely to be practicable with the downstream and centreline methods of construction even when the raw tailings are very coarse. Usually, it will be more practicable with these methods to separate the coarser sand by passing the raw slurry through cyclones, or through open-channel or pipe sluices. The cyclones are usually mounted on the crest of the embankment, as shown on Figure 6-2, but they can also be mounted on an abutment at one side of the embankment. From the cyclones, the fine overflow containing the "slimes" is piped to the tailings pond; the underflow containing the coarser sands drops, or is discharged through pipes or launders, onto the embankment.

4. Sluices

Sluices can be formed by dyking the edges of the embankment crest, as shown on Figure 6-3, to form a channel sloping gently downwards. Raw tailings are discharged into the upper end of this channel, the coarser fractions settling in the channel while the finer fractions are carried in suspension to openings discharging through the dykes into the pond. The coarse sands are periodically removed from the channel by bulldozers or draglines, to be spread and compacted on the shoulders of the embankment. A variation of the channel sluicing method would be to use pipe sluices, consisting of half-pipe sections incorporating baffles and openings in the bottom of the pipe. Coarser sands are retained by the baffles and drop through the pipe openings onto the embankment; the finer "slimes" flow on to the pond.

The sluicing method of construction requires flat fill slopes for operation of the spreading and compacting equipment. Slopes are normally required to be 4:1 or flatter. Because of the relatively large volumes of sand required in the resulting embankment cross-sections, they are usually suitable only when the raw tailings contain high percentages of medium to coarse sand particles.

Sand Characteristics

The nature of the sand incorporated in the embankment cross-section is one of the most critical factors influencing the design, and the construction, of those embankments which utilize tailings sand as the prime embankment construction material.

Whatever the method of sand separation, if the sands deposited on the embankment have to be spread and compacted, they must be relatively free-draining. As deposited, they will generally have water contents considerably higher than the optimum for compaction. The underflow slurry from cyclones, for example, can have a maximum concentration of solids of 70-75 per cent by weight. The equivalent soil water content, expressed as a percentage of water to solids weight, is 43-33 per cent. In comparison, the optimum water content for compaction is usually in the range of 10-20 per cent.

If the sands deposited on the embankment have a relatively high content of clay-like "slimes", they will not only be slow in draining to optimum water content, but they will be difficult to handle. Coming from the cyclones with a high water content and a high slimes content, they will flow on very flat slopes, even to the extent of spreading beyond the design boundaries of the embankment. In contrast, if the slimes content is low, water will drain rapidly from the sands, limiting the distance they flow on the embankment and allowing spreading and compaction by mechanical equipment a short time after their deposition.

The most suitable fines content for sands to be compacted in the embankment will depend, to some extent, on the mineralogical nature of the slimes. The reduction in fines content of the raw tailings that can be accomplished on the embankment will depend on the method of sand separation used and on the gradation of the raw tailings. For example, with raw tailings solids containing 70-80 per cent by weight of fines (-200 mesh), even two-stage cycloning may not reduce the fines content to less than 10-15 per cent. As indicated by Figure 3-12, this could still result in a cyclone underflow of relatively low permeability, particularly if the slimes contain clay minerals. Lower fines contents would improve the drainage and consequent handling characteristics of the sands on the embankment. However, they would also increase the permeability, which would increase the seepage rates through the embankment, if the sand zone formed the main water barrier.

Variations in sand characteristics will occur on the embankment even with cyclone separation. Fluctuations in the concentration of the raw tailings, and in feed flow rates and pressures, will affect the gradation and water content of the sand in the cyclone underflow. Only a limited amount of cyclone adjustment is possible to counter these fluctuations. For this reason, gravity feed to the cyclones is preferable to pumped feed.

In general, the most suitable gradation for embankment sand should be studied, during the design phase, by making laboratory tests on samples of tailings. If necessary, trial batches of raw tailings should be produced for this purpose. Practicable methods of producing a suitable sand at the site can then be developed. Discussions with manufacturers will usually be necessary to decide on the types and number of cyclone stages required to provide a suitable underflow. A knowledge of the probable range of permeability of the sand from laboratory tests will enable suitable seepage control provisions to be included in the design.

Because of the variations which can occur in actual sand characteristics on the embankment, these should be checked by sampling and testing during the early stages of embankment construction. The sand separation and placement procedures should be supervised continuously.

Sand Yield

A second critical factor affecting the design and construction of embankments built of tailings sand is the yield of suitable sand obtained in separating the coarser fractions from the raw tailings. Variations in sand yield have a dual effect - a decrease in yield will slow the rate of rise of the embankment crest and increase the rate of rise of the tailings in the pond. Particularly with finely ground tailings, the yield may be too low to fill the complete embankment section with sand at a rate sufficiently high to keep the crest above the pond surface. This may require replacement of some of the sand in the section with borrow or dry waste materials as shown on Figure 5-1.

The sand yield from cyclones can be computed from information on the gradation of the raw tailings and the characteristics of the cyclones. For sand separated by sluicing methods, actual field tests may be the only means of estimating what the yield may be.

Estimates of the long-term rate of embankment construction (in particular the rate of rise of the embankment crest) should make adequate allowance for losses in sand production time. Significant losses can include time during which the mill is shut down, winter periods when it is not practicable to operate spigots, cyclones or sluices, time spent in replacing tailings disposal pipelines and cyclones (and moving them from one crest level to another) and, sometimes, time during which tailings must be dumped into the pond at points other than on the embankment to ensure full utilization of the pond capacity for tailings storage. These losses can substantially reduce the long-term sand production rate.

Starter Dams

Construction of a "starter dam" will usually be the first step in building a tailings embankment. The primary purpose of this dam is to provide a pond of sufficient size to ensure effective water clarification when tailings disposal begins. This requirement will establish the minimum permissible crest elevation. Notes on pond size have been included in Section 5.

There may be advantages in constructing the starter dam to a higher elevation than that required simply to create a pond sufficient for reclaim clarification. For an embankment constructed of tailings sand by the downstream method, a relatively large volume of sand usually has to be placed downstream of a starter dam before the crest can be raised. Meanwhile, tailings and water are rising in the pond at a relatively rapid rate, because the storage capacity of the pond at low levels is small. A higher starter dam can provide more time for the placement of sands in the embankment before the pond reaches the crest of the starter dam. It may thus eliminate a need for the placement of other borrow materials in the base of the embankment. It may also eliminate a need for a temporary spillway over the starter dam, by providing more storage for runoff water. Determination of the most suitable starter dam crest elevation, therefore, should consider these three principal factors - water clarification, the most economical overall embankment cross-section, and runoff control.

The importance of providing a relatively impermeable starter dam when it is located at the upstream toe of the main embankment, and a relatively permeable dam at the downstream toe, has been stressed previously.

Construction Stages

Usually, tailings embankments are raised in stages as required to keep the crest above the pond surface. The most practicable method of staging will depend on the methods of sand production and distribution used and on the volumes of sand required in the embankment cross-section. Methods of laying tailings pipelines along berms on the embankment, the pipelines being moved successively from one berm to another, are shown on Figures 6-1 and 6-2. Particularly with the downstream method of construction, it may be necessary to mount tailings lines and/or cyclones on trestles or towers for construction of successive stages of the embankment. This may be required to keep the lines above the pond surface, or to provide temporary storage for cyclone-underflow sand prior to it being spread and compacted. These trestles and towers are often left embedded in the embankment.

Alternatively, an embankment cross-section and construction stages may be established so that tailings lines can be laid on successive berms, with cyclones mounted on raised, movable platforms for construction of the next stage. Such an arrangement is illustrated on Figure 6-4, which shows an embankment constructed by the downstream method and incorporating a zone of rockfill in its base. The embankment construction schedule, as illustrated on Figure 6-5, is related to the rate-of-rise of the pond. The elevation of the crest of the starter dam, the volume of rock fill in the embankment and the stage boundaries are established so that successive berms are ready for the tailings pipelines before the tailings rise to the level of the berm on which they are operating.

Foundation Treatment

For dams designed to retain water, the treatment of the foundation surfaces under the dam can be critical to its stability. Where starter dams must retain considerable depths of water unassisted by any blanketting of the foundation by tailings, this becomes an important consideration.

Foundation defects such as open fissures in the bedrock, or coarse pervious foundation soils, can be particularly dangerous. Each significant defect should have custom treatment, which could include:

- excavation of pockets and zones of weak and pervious soil and shattered bedrock,
- sealing with slush concrete those fissures that are exposed under the upstream shoulder of the embankment and providing drains and filter material over those fissures exposed under the downstream shoulder,
- consolidation grouting and/or a grout curtain (usually for high dams or unusually difficult foundations only),
- scarifying and compacting soil foundation surfaces,
- special compaction of the initial layers of fill placed over the foundation.

Foundation treatment to prevent high seepage and piping becomes less important at greater distances from the core of the dam and where foundations are blanketted by low-permeability tailings. Final decisions on foundation treatment depend upon actual conditions exposed during construction and, therefore, actual treatment is often determined in the field. Minor geological details can be important in determining the nature and extent of the treatment. For stability against sliding failures, the necessary extent of removal of weak foundation materials should always be considered.

Fill Compaction

Compaction of a soil is usually accomplished by spreading the soil in layers and compacting it with a mechanical compactor. Compaction may be specified by procedure (type of compactor, layer thickness, number of coverages and placement water content to be used) or by end product (minimum in-place density required). The purpose of compaction may be to increase the fill shear strength or to decrease the fill permeability.

Variables affecting compaction are:

- the type of compactor (non-vibratory pneumatic, steel-wheel, sheep's foot and grid rollers; vibratory steel-wheel rollers, plate compactors and track-type tractors),
- the weight and energy of the compactor,
- the thickness of layers,
- the placement water content.

The types of compactors and layer thicknesses suitable for the compaction of various classes of soils to 95-100 per cent of Standard Proctor Maximum Density are indicated on Figures 6-6 and 6-7. For cohesionless tailings sands, compaction by track-type tractor is normally adequate and efficient. Control of placement water content is usually unnecessary as high water contents are not detrimental when the slimes content is low. The tractor weight is limited to the support capacity of the wet sand.

For cohesionless borrow and waste materials such as rockfills, gravels and clean sands, compaction by track-type tractor and haulage units can be adequate, if properly controlled. Heavy, vibratory steel-wheel compactors are very efficient in compacting these materials, where additional compaction is required. Water content control may increase the efficiency of compaction, but is not essential for such cohesionless materials, in many cases. For stony materials, the maximum stone size permitted should be two thirds of the specified compaction layer thickness. Layer thicknesses up to 24 inches can be used, under favourable conditions, with heavy, vibratory steel-wheel compactors.

Cohesive soils are most efficiently compacted by heavy pneumatic or sheepsfoot rollers. Control of the placement water content is important to the efficiency of compaction. Where the clay content of the fill material is significant, compaction by pneumatic tyred or steel drum rollers results in smooth surfaces at the top of each layer. These smooth surfaces should be scarified prior to placement of the overlying layer.

Additional poins to consider when specifying spreading and compacting procedures are:

> a particular type of compactor has its own optimum water content; this may not be the same as that determined by standard laboratory tests; it can be determined, most accurately, by full-scale field tests with the compactor; usually, however, laboratory tests give optimum water contents within a few percentage points of those applicable to most compactors; generally, the heavier the compactor (of a particular type), the lower will be the optimum moisture content;
>
> in tailings embankment starter dams, or impervious cores, low fill permeability may be an equally important consideration to shear strength; with very stony soils, such as some glacial tills, compaction at water contents higher than the optimum will usually produce a more homogenous fill, with consequently reduced permeability;
>
> because of the high water contents generally existing in tailings sands, tractor compaction will often be the most practicable method for the sand zones of tailings embankments;
>
> use of compaction layers which are too thick for a particular roller will result in under-compaction at the base of the layers, with consequent increases in horizontal permeability of the zone.

Water Reclaim Systems

1. Types

One of three basic systems is generally used for reclaiming or discharging clarified water from a tailings pond. A common system in the past has used "decant" culverts through the embankment. In this system, water near the surface of the pond flows through openings spaced over the height of a tower, or riser pipe, constructed at some location in the pond. Each of these openings, in its turn, is plugged as the tailings in the pond reaches its level. Collected water flows down the tower and through a culvert constructed under the embankment. Downstream of the embankment the water can be allowed to drain away (if permitted by pollution control requirements) or it can be collected and pumped back to the ore processing plant. Some types of decant and other reclaim systems are illustrated on Figure 6-8.

A second water reclaim system now commonly used is to mount pumps on a floating barge or in a pumphouse capable of being moved up a ramp. These pumps draw water at shallow depths and discharge it into pipelines for return to the mill.

A third system uses siphon pipes installed over the crest of the embankment to draw water from the pond and discharge it to the downstream toe of the embankment as shown on Figure 6-9.

2. Comparison of Reclaim Systems

In relation to barge-pump reclaim systems, the decant system can have the following advantages:

mechanical or electrical failures do not interrupt the discharge of water from the pond,

if installed to cover the whole range of pond surface levels (from empty to full), the only recurring operation required to keep them in service is to successively plug the water intake openings as the tailings rise,

if they have sufficient flow capacity, they can serve as permanent drains to handle runoff and keep the pond empty after the tailings operation has been abandoned.

They have the following disadvantages:

where pond water is reclaimed from a point downstream of the embankment, rather than from the pond, the average pumping head to the mill is greater (assuming the pond is below the elevation of the plant);

high decant towers are particularly susceptible to damage by movements of the tailings solids by which they are surrounded; destructive movements can be caused by slumping of the tailings, particularly when they are deposited to substantially higher elevations along the edges of the pond than at points near the decant tower;

the actual pressures to which decant culverts are subjected by overlying tailings are uncertain; in design, the conservative assumption, therefore, should be adopted that they are subjected to the full hydrostatic pressure of the saturated tailings above them;

foundation settlements are very likely to crack and/or open joints in decant culverts, often leading to serious piping of tailings solids into and through the culvert pipes;

even if large enough to permit a man's entrance, collapsed sections of culverts located beneath an embankment are very difficult to repair, and piping occurring into them is practically impossible to control.

The principal disadvantage of barge-pump reclaim systems is the necessity to move the barge (or pumphouse) periodically as the pond water surface rises. In order to keep these moves to a minimum the tailings disposal-reclaim system should be considered as a whole. Tailings sloping down from the disposal points should force the free water towards the barge-pump location. Where possible, the barge-pump should be located adjacent to a relatively steep portion of the pond perimeter so that long pontoon-supported flexible pipes and access ways can be avoided. Preferably, the barge should be moved away from the main embankment as the pond surface rises and expands in area. If practicable, this will permit the pond surface to be kept smaller, and farther from the embankment, for the same depth of water at the barge.

If used, decant culverts should be conservatively designed. Suitable design methods are described in the references. In general, they should be avoided where foundation settlements are expected. Barge-pump reclaim systems are better, in principle, from the point of view of embankment safety.

Siphon reclaim systems have been used on some tailings embankments. The main advantage of the siphon is its ability to pass full-capacity discharges with narrow limits of water surface rise in the pond. Constructed over the crest of the embankment, it is subject to cracking by settlement of the embankment. Other disadvantages are:

its inability to pass ice and debris;

the possiblity of water freezing in the inlet legs and air vents before the pond rises sufficiently to prime the siphon;

the occurrence of surges and stoppages as a result of erratic make-and-break action of the siphon;

the siphon pipes are subject to cavitation and low absolute pressures; for this reason, they are usually limited to a total drop of 20 feet maximum on earth dams.

WASTE PILES

General

The necessary control procedures for the placement of "dry" mine wastes will depend upon the:

location of the waste pile in relation to the mine and the ore processing plant,

topography of the waste pile area,

nature of the waste materials,

type of haulage method used.

Simple dumping from conveyors or haulage units will usually be the preferred method. However, placement methods, including spreading and compaction, should be selected with respect to the waste materials as well as costs. Important factors to be observed during construction are that:

weak foundation materials are removed;

surface drainage is adequate to prevent undesirable wetting of the fill and the foundation by surface runoff. (drainage ditches should be maintained);

drainage material is suitable and is properly placed. (pipes should be protected to prevent breakage):

the outlets of drainage layers are not sealed by fill dumped over them;

in waste piles on steep slopes, materials are placed in horizontal layers, starting at the lowest point in the area.

Trial Embankments

In the design of mine waste piles, the shear strength and other characteristics of the wastes are often not known, to a sufficiently reliable degree, prior to actual production of the wastes. Where necessary, therefore, for adequate assessment of the stability of the pile, trial sections of the pile should be constructed in the field, sampled and tests made in the laboratory to determine their characteristics. Improvements in shear strength produced by compaction can also be investigated on trial embankments.

Suitable methods of field and laboratory testing have been described in Section 4.

CONSTRUCTION PORE PRESSURES

The development of high excess pore water pressures in foundations or embankments consisting of fine-grained soils may necessitate limitations on the rate of rise of the fill, in order to prevent instability. The rate of pore water pressure dissipation is related to the soil permeability. Measures that may be used to control fill and foundation pressures include:

drainage layers within the embankment and sand drains in the foundations,

control of the rate of filling based on monitored observations of pore water pressures and embankment deformations.

EMBANKMENT INSTRUMENTATION

Types

Many factors affecting the stability of mine waste embankments can change during the active life of the embankment. This is further discussed in Section 7. Where changes may be critical to stability, instrumentation should be installed in the embankment and/or its foundation to monitor those changes actually occurring.

Instruments can be installed to measure:

piezometric levels (water levels),

seepage flows,

embankment movements,

total pressures.

Piezometers

A simple and effective piezometer is the Casagrande type illustrated on Figure 6-10. It is installed in a hole drilled into the embankment or its foundation and water levels are measured by a probe lowered down the hole. In large diameter holes, several piezometer tips can be installed in the one hole, provided effective seals are placed to isolate each pervious tip from the other.

The Casagrande type piezometer is non-metallic and is designed to reflect piezometric changes with a minimum of water volume change. Similar, slower reacting types can be installed using perforated steel casing, or steel casing and well points. Alternatively, hydraulic or electrical piezometers are available which can be installed horizontally at various levels in the embankment. These are described in the references. Generally, they avoid the necessity for drill holes but their operation is more complicated and their reliability over long periods requires great care in fabrication and installation.

Seepage Flow Measurement

Records of seepage flows can indicate significant changes occurring in an embankment or its foundation. The seepage water emerging downstream of the embankment is collected in drains, set as low as practicable, which lead to a measuring weir or flume. Such weirs and flumes are described in the references. The measurement of turbidity and chemical content at this point could provide useful information. Since runoff often enters the seepage collection drains, a record should be kept of precipitation affecting the flow measurements.

Movement Indicators

Various devices can be installed to measure embankment movements. Markers can be installed on the surface for periodic surveys; aligning these in a straight line-of-sight permits rapid detection of horizontal movements. Levelling of temporary bench marks provides measurement of surface settlements. Successive measurements between two pegs spaced on either side of cracks will show if these cracks are widening and if the opening rate is accelerating.

The United States Bureau of Reclamation has used telescoping settlement rods (with cross-arms mounted at intervals on the rods) for many years. These devices are detailed in the references. They are installed vertically in the embankment as its surface rises; measurements are made with a latching device lowered inside the rods.

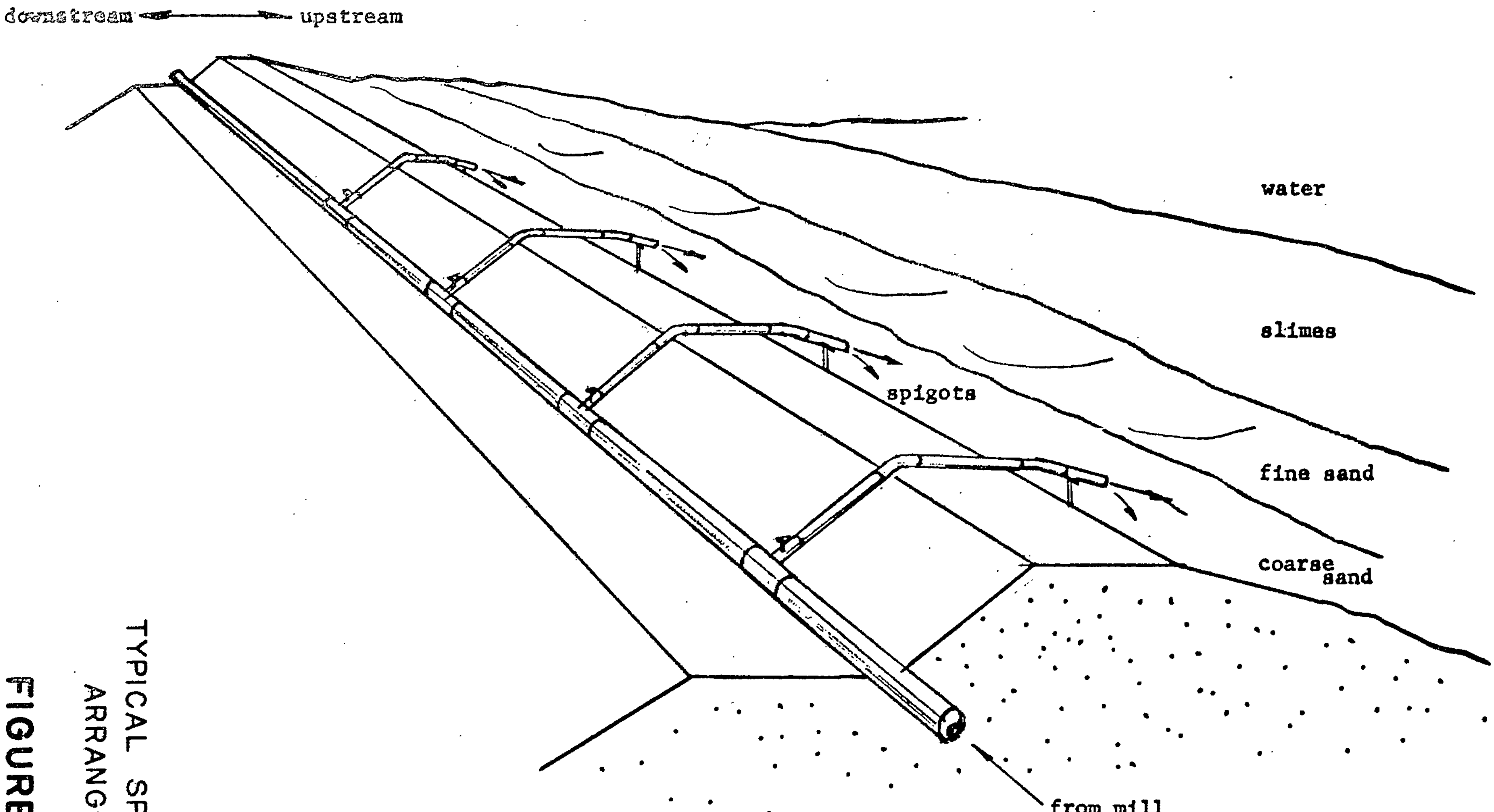
Another device for measuring both vertical and horizontal movements is the "slope-indicator". For this device, telescoping cylindrical casing is installed in the embankment as it rises. The instrument is lowered down grooves inside this casing and measures the slope of the casing in two directions at right angles, and the vertical settlement. From the measured slopes, the horizontal movements occurring over the height of the casing can be calculated. These instruments can be supplied by various companies.

Pressure Cells

Various types of devices are available for measuring the combined pressure of soil and water against plane surfaces. They could be useful in checking actual pressures on decant or other culverts installed under embankments.

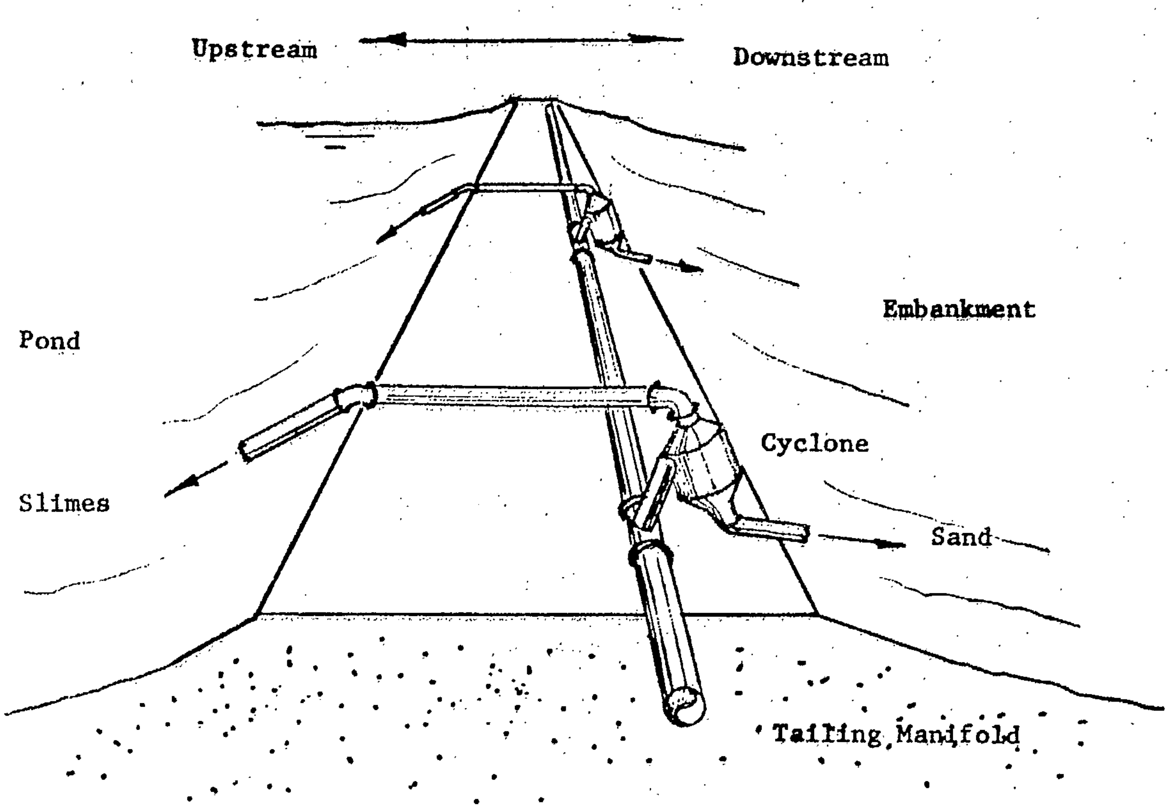
Records

Measurements made at instruments installed in an embankment or its foundation should be recorded. The frequency of the measurements should be consistent with experienced variations of the quantities measured. Initially, measurements of piezometric levels, seepage flows and precipitation should be made daily. Records should also be kept of any changes in construction and waste placement procedures, and of waste material characteristics, which may change the distribution and properties of the materials in the embankment. The data obtained should be presented in graphical form, so that variations and trends can be readily noticed. Regardless of the quantity and quality of observational data obtained, it is of little value unless reviewed on some regular basis by someone qualified to assess its meaning. Normally, this review should be done by the designer of the embankment.



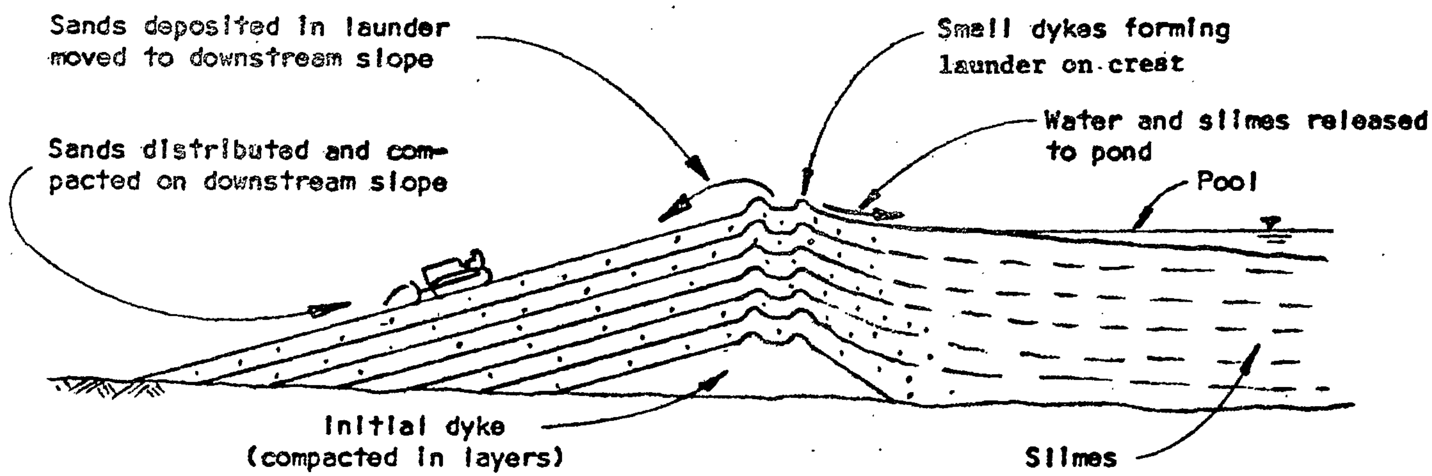
TYPICAL SPIGOTTING ARRANGEMENT

FIGURE 6-I



TYPICAL CYCLONING ARRANGEMENT.

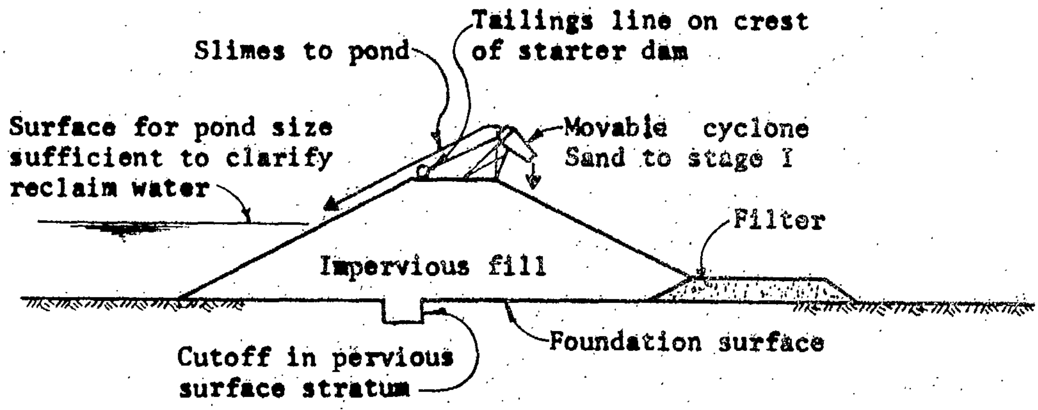
FIGURE 6-2



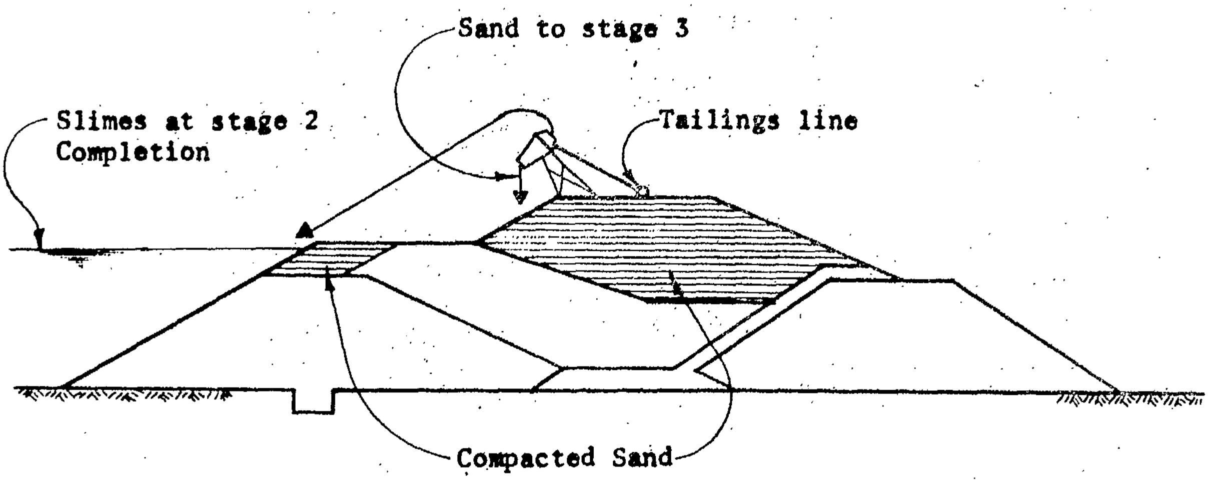
(After Casagrande, 1970)

TYPICAL SLUICING ARRANGEMENT

FIGURE 6-3



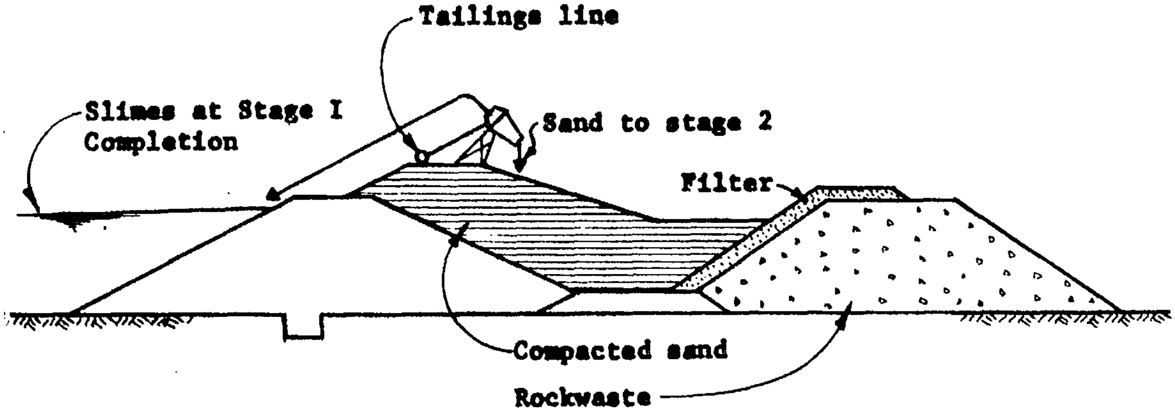
STARTER DAM



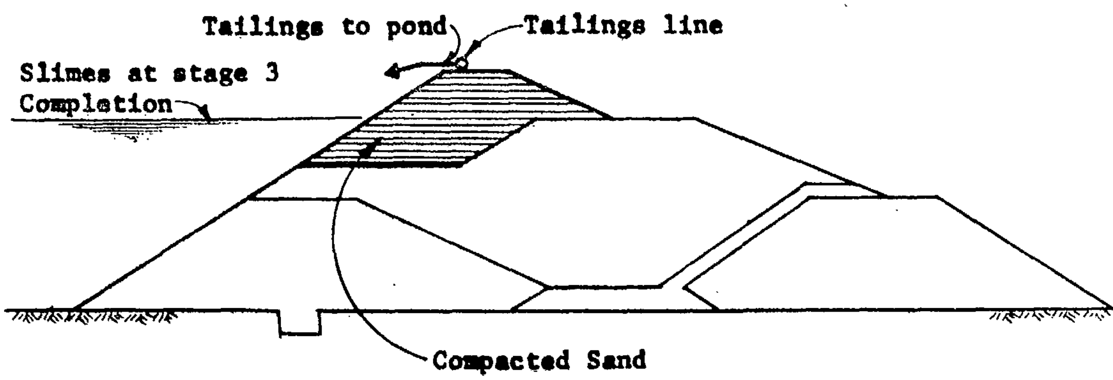
STAGE 2

Notes on Basis of Design

1. Raw tailings fines content (-200 mesh by weight) = 70%
2. Sand fines content after two stage cycloning = 11%
3. Sand yield as percentage of raw tailings solids = 34%
4. Sand dry density in embankment = 100 lbs./cu.ft.
5. Slimes dry density in pond = 85 lbs./cu.ft. average
6. Impervious fill and filter material obtained from borrow pits; rock waste obtained from ore processing plant.
7. Volume of rock waste in embankment, starter dam crest elevation and sand stage boundaries established to permit completion of stages before tailings line operating berms overtopped by slimes - see figure 6-5.
8. Cyclones operate 60% of time following plant start.



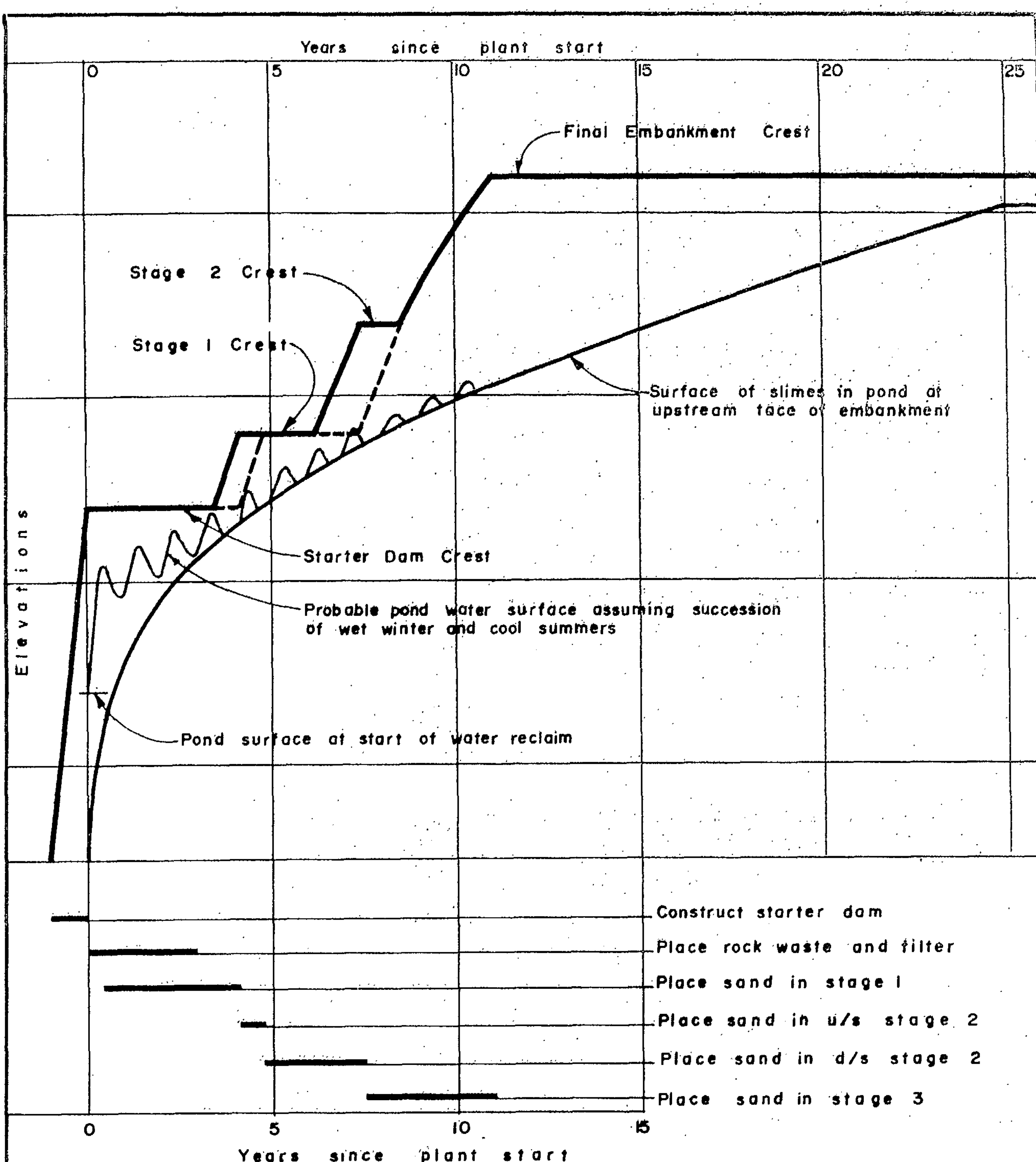
STAGE I



STAGE 3

TAILINGS EMBANKMENT
CONSTRUCTION STAGES

FIGURE 6-4



Notes:

I. Schedule is for embankment shown on Figure 6-4

TAILINGS EMBANKMENT CONSTRUCTION SCHEDULE

FIGURE 6-5

Compaction Equipment and Methods

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Sheepsfoot Rollers.	For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing the No. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.	6	4 to 6 passes for fine-grained soil. 6 to 8 passes for coarse-grained soil.	Soil Type / Foot Contact Area sq. in. / Foot Contact Pressures p.s.i. Fine-grained soil PI > 30 / 5 to 12 / 250 to 500 Fine-grained soil PI < 30 / 7 to 14 / 200 to 400 Coarse-grained soil / 10 to 14 / 150 to 250 Effecient compaction of soils wet of optimum requires less contact pressures than the same soils at lower moisture contents.	For earth dam, highway and airfield work, drum of 60 in. dia., loaded to 1.5 to 3 tons per lineal ft. of drum generally is utilized. For smaller projects 40 in. dia. drum, loaded to 0.75 to 1.75 tons per lineal ft. of drum is used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber Tire Rollers.	For clean, coarse-grained soils with 4 to 8 percent passing the No. 200 sieve.	10	3 to 5 coverages.	Tire inflation pressures of 60 to 80 p.s.i. for clean granular material or base course and subgrade compaction. Wheel load 18,000 to 25,000 lb.	Wide variety of rubber tire compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For cohesionless soils, large-size tires are desirable to avoid shear and rutting.
Do	For fine-grained soils or well-graded, dirty coarse-grained soils with more than 8 percent passing the No. 200 sieve.	6 to 8	4 to 6 coverages.	Tire inflation pressures in excess of 65 p.s.i. for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use large size tires with pressures of 40 to 50 p.s.i.	
Smooth Wheel Rollers.	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures.	8 to 12	4 coverages.	Tandem type rollers for base course or subgrade compaction, 10 to 15 ton weight, 300 to 500 lb. per lineal in. of width of rear roller.	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are available in the range of 1 to 20 ton weight. 3-axle tandem rollers are generally used in the range of 10 to 20 ton weight. Very heavy rollers are used for proof rolling of subgrade or base course.
Do	May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.	6 to 8	6 coverages.	3-wheel roller for compaction of fine-grained soil; weights from 5 to 6 tons for materials of low plasticity to 10 tons for materials of high plasticity.	

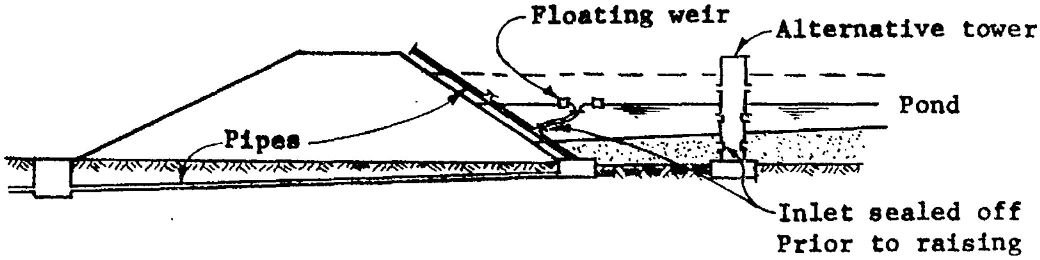
FIGURE 6-6

Compaction Equipment and Methods

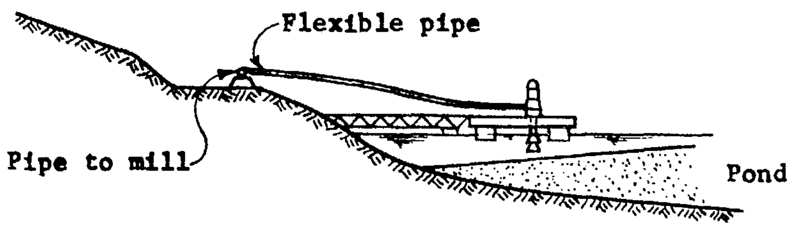
Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Vibrating Baseplate Compactors.	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200, placed thoroughly wet.	8 to 10	3 coverages.	Single pads or plates should weigh no less than 200 lb. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1,600 cycles per minute.	Vibrating pads or plates are available, hand-propelled or self-propelled, single or in gangs, with width of coverage from 1½ to 15 ft. Various types of vibrating-drum equipment should be considered for compaction in large areas.
Crawler Tractor.	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	10 to 12	3 to 4 coverages.	No smaller than D8 tractor with blade, 34,500 lb. weight, for high compaction.	Tractor weights up to 60,000 lb.
Power Tamper or Rammer.	For difficult access, trench backfill. Suitable for all inorganic soils.	4 to 6 in. for silt or clay, 6 in. for coarse-grained soils.	2 coverages.	30-lb. minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weights up to 250 lb., foot diameter 4 to 10 in.

(After Hvorslev, 1949)

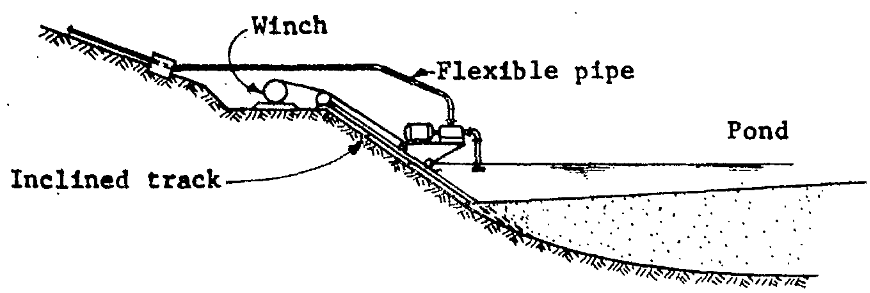
FIGURE 6-7



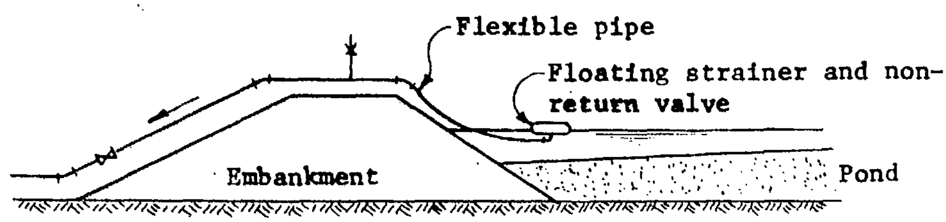
DECANT CULVERT



BARGE PUMP



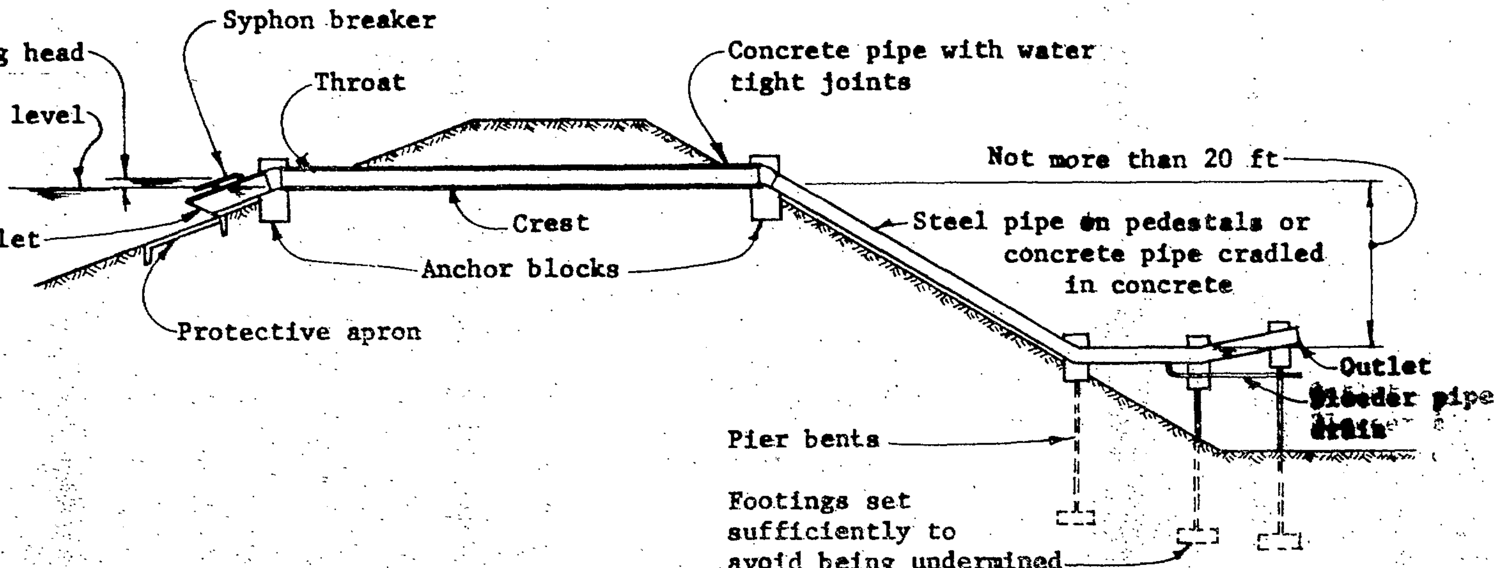
MOVABLE PUMP



SIPHON

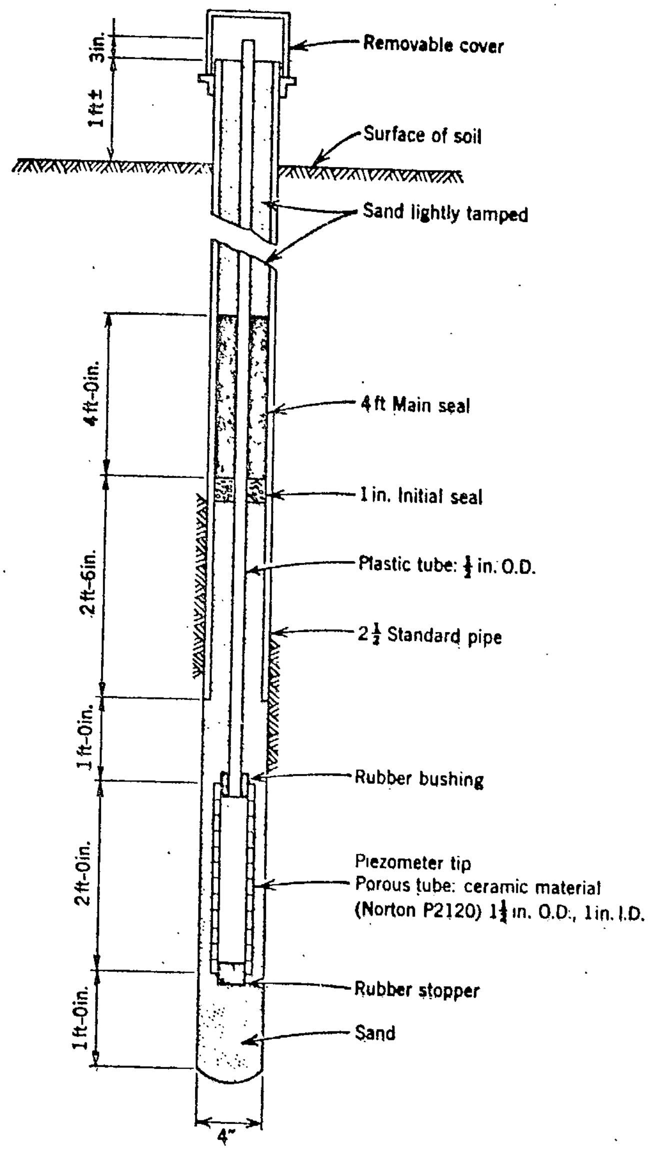
TYPES OF WATER
RECLAIM SYSTEMS

FIGURE 6-8



SMALL SIPHON SPILLWAY

FIGURE 6-9



CASAGRANDE
PIEZOMETER

FIGURE 6-10

SECTION 7

MAINTENANCE AND RECLAMATION

CONTENTS

FIGURES

SECTION 7

MAINTENANCE AND RECLAMATION

MAINTENANCE PROGRAMMES

The formation of mine waste piles and tailings embankments very often proceeds over a period of many years and many conditions can develop during these long periods to affect the stability of the embankment. In this respect, mine tailings embankments differ substantially from normal water retaining dams, the latter usually being constructed over a relatively short period with tight control over the quality of the construction materials and methods.

The characteristics of the materials and methods used on mine waste embankments may change substantially over the years for many reasons. Such changes can drastically alter the conditions governing the stability of an embankment, from those provided for in the original design to others which may produce instability. In essence, different conditions usually develop year-by-year throughout the whole active life of the embankment. There are different crest levels, different water levels, different embankment slopes and cross-sections, different seepage conditions - and there can be different material characteristics. Therefore, there is a requirement for some continuous programme of inspection and maintenance of the embankment, beginning with the start of waste disposal and, sometimes, continuing after abandonment of the completed embankment. The main objectives of such a programme should be to ascertain:

whether the embankment and its foundation are behaving in the manner anticipated in the design; are they moving, settling, cracking, eroding, sloughing, or leaking more than anticipated?

whether the waste and borrow materials being placed in the embankment have the characteristics assumed in the design; are they changing and, if so, how will these changes affect the embankment stability?

whether the distribution of materials in the embankment and tailings pond is similar to that assumed in the design; are changed waste disposal procedures affecting material distribution and characteristics?

whether tailings pond levels are rising in the manner anticipated; are the slimes or liquids in the pond threatening to overtop the crest of the embankment; is the rate of embankment construction sufficiently rapid to keep the crest above the rising pond; are actual runoff and evaporation rates of the order forecast?

whether embankment drainage is adequate; is the capacity of culverts adequate to pass experienced and anticipated runoff and seepage; have culverts collapsed or become blocked; is embankment material becoming saturated by seepage or runoff; is piping or subsurface erosion occurring in tailings embankments or into decant culverts passing through them ?

the water table levels within the embankment; how do they vary with pond levels and precipitation; have there been increases in level, or seepage flow, that cannot be explained by increases in pond level or precipitation; if so what caused them ?

The answers to these questions can be significant in detecting incipient or developing instability in the embankment.

INSPECTIONS

Besides daily examination of certain areas, such as decant lines, and special inspections after heavy rains, detailed inspections of high embankments, and of those whose failure would entail serious consequences, should be made at least twice each year during the waste disposal period. Such inspections should include not only on-site inspections of the embankment itself but also reviews of the records of instrumentation installed in or near the embankment.

Particular points to be checked during inspections are:

the presence of longitudinal and transverse cracks on the crest or slopes of the embankment;

the extent and rate of horizontal and vertical movements indicated by surface reference markers, settlement rods and slope indicators; (are the movements acceleration?);

any signs of heaving of the foundation near the toes of the embankment slopes;

variations in piezometric water levels and in seepage flows; the development of any springs or wet areas on the embankment and foundation surfaces;

conditions at seepage exit points and at culvert or decant pipe outlets; is there sloughing at these points; is water flowing along the outside surfaces of the pipes; is the seepage water clear or does it contain sediments; are there any signs of sink-holes appearing on the upstream faces of tailings embankments; the latter two points could indicate the occurrence of sub-surface erosion;

the nature of the materials being placed in waste retaining embankments (and of the wastes themselves), particularly of spigotted and cycloned sands being placed in tailings embankments; the most important characteristics will be their basic mineralogical character, (siliceous or clay minerals), their gradation and their moisture content;

- the geometry of the embankment; is it being filled to the heights, widths and slopes specified in the design: have any unanticipated excavations been made in the embankment foundations, abutments or slopes that could threaten its stability?

- spillway and diversion structures; are they in good repair and capable of performing as required by the design?

- the extent of any erosion occurring on the embankment and abutment slopes and in the channels of stream diversions or spillways.

Careful attention to these points can often provide forewarning of developing instability.

REMEDIAL MEASURES

General

The extent and nature of remedial measures required to maintain or improve the stability of mine waste embankments will vary with circumstances. Some developments may require extensive work addition to that anticipated in design. For others, minor repairs may be adequate. Descriptions of the principal types of measures effective in improving stability and in combating erosion follow.

Slope Flattening

This is illustrated on Figure 7-1. By removing the weight of material near the crest of the slope, the driving force tending to produce a slide is reduced. If the material removed from the crest is dumped over the toe of the slope, measures should be taken to ensure that it does not impede drainage from the embankment. (A drain could be required if the material has a permeability low in comparison to that of the material in the base of the embankment.)

Berms

This is a special case of slope flattening. It is also illustrated on Figure 7-1. Generally, a berm can improve foundation stability but may not be very effective in increasing the factor of safety against slope failure unless it is at least one third to one half the height of the embankment. Its effectiveness will also depend on the shear strength of the material in the berm. Compaction may be necessary.

If the embankment is to be raised, it may be necessary, for adequate stability, to flatten the overall slope by stepping the slope forward, thus forming a berm at the original crest level.

Height Reduction

This could be in the form of excavation to form a berm, as shown on Figure 7-2, thus flattening the overall slope.

Seepage Control

Inverted filters can be used to prevent erosion and sloughing caused by seepage from embankment or foundation surfaces, as shown on Figure 7-2. Such filters can prevent the development of piping. However, if piping has developed to the point where sink-holes are forming on the upstream face of the embankment, it may be necessary to blanket this upstream face with impervious fill or to seal the leaks by grouting. Where sub-surface erosion is occurring into culverts or pipes buried under the embankment, these methods may be the only ones possible also.

Relief wells can sometimes be used to lower the water table under embankment slopes, as shown on Figure 7-2. Such systems can be expanded as the need arises. Generally, they are not very effective where the holes are located outside of the embankment fill. Inclined holes can be drilled under the embankment; however, drilling at angles more than about 30° off vertical usually involves additional costs. Relief well systems are most effective when installed under the embankment before the start of waste disposal.

Surface Drainage

Generally, less erosion of waste pile slopes will occur when the upper surface of the embankment is graded down towards the hillside or towards the centre, as shown on Figure 7-3, and the runoff led away through drainage ditches or pipes. A drainage system of this type is preferable to one where drains are located close to the top of the slope, as seepage from such drains can affect the stability of the slope. Erosion of long slopes can be reduced by breaking the length of the slope by berms and leading the drainage water to drop pipes. Broken rock, coarse gravel or grass (if it will grow on the slope) can also be used to limit slope erosion.

Maintenance of embankment drainage systems should be directed to preventing unnecessary entry of water into the embankments. The drainage system should be inspected at invervals, particularly after heavy rain, with a veiw to keeping the system free of obstruction. Arrangements should be made for any work required to be carried out such as:

clearance of vegetation, sediments and refuse from trash screens and drainage ditches,

rodding of pipe drains to clear sediments or salt deposits; cleaning of silt traps,

attention to the outlets of any drainage zones, or other seepage outlets.

repairs as required,

the diversion of any flow or accumulation of water into the permanent drainage system,

attention to filters.

Tailings settlement ponds may require adjustment of inlet and decant arrangements, the clearance of any blockages, and rectification of any undercutting of the embankment slopes brought about by wave action.

For completed tailings embankments it may be necessary to maintain the decant or overflow systems to prevent the accumulation of rainwater or to control surface run-off; the clearance of silt from these systems is likely to be necessary from time to time. Breaching a tailings embankment is not generally an acceptable means of preventing an accumulation of water; this method should only be used if erosion caused by the outflow will not endanger the embankment, block the drainage system or cause nuisance in other ways.

Toe Embankments

Where waste piles are located on relatively steep hillsides they may start to move downhill, particularly if the materials are fine and become saturated, or they are affected by weathering. Such movement can sometimes be stopped by constructing an embankment of compacted material at the toe of the pile.

PREVENTION AND CONTROL OF COAL WASTE FIRES

Factors Influencing Spontaneous Combustion

1. Temperature

Spontaneous combustion of carbonaceous materials is an oxidation process in which the material combines with oxygen from the air, with the evolution of heat. Oxidation usually proceeds very slowly at ambient temperatures but increases rapidly and progressively as the temperature rises. Neither oxidation, spontaneous combustion, nor burning, will occur in the absence of air. For burning to occur with visible flame it is normally necessary for the material to be combustible, to reach its specific ignition temperature, and for sufficient oxygen to be supplied from the air.

The combination of atmospheric oxygen with carbonaceous material releases heat. If the heat generated is not dissipated, the oxidizing and heating effects become cumulative and the temperature rises more rapidly, thus increasing the rate of oxidation still further. Coal and carbonaceous materials may oxidize in the presence of air at ordinary temperatures, far below their ignition point. Materials containing cellulose, such as wood, straw, jute, paper and cardboard do not react appreciably with oxygen until approaching their ignition temperatures, within the range of about 260°/300°C. However, at ambient temperature and in the presence of moisture, these materials are liable to spontaneous heating by the action of certain micro-organisms.

2. Coal Rank

In general, lower rank coals are more reactive and hence more susceptible to self heating than higher rank coals.

3. Presence of Pyrite

Iron pyrite (FeS_2) is oxidized at normal temperatures by moist air to form ferrous sulphate and sulphuric acid, and this reaction also is strongly exothermic.

Pyrite, if present in sufficient proportion, and particularly when finely divided and associated with carbonaceous matter, increases the tendency towards spontaneous combustion in waste piles. Where heating occurs, the oxidation of pyrite and organic sulphur in the coal will form sulphur dioxide, and where there is insufficient air for complete oxidation, hydrogen sulphide will be given off. The characteristic smell of these gases sometimes provides a means of detecting heating.

4. Moisture

At relatively low temperatures an increase in free moisture increases the rate of spontaneous heating. High inherent moisture, a feature of low rank coals, is indicative of a tendency to spontaneous heating. The presence of free moisture is essential for the oxidation of pyrite and, in the presence of pyrite, moisture accelerates oxidation and contributes to heating.

5. Void Ratio and Specific Surface

The ease with which air passes through waste containing carbonaceous material determines the rate at which heat generated by oxidation is carried away. With large-size material and large air voids, the movement of air is usually sufficient to carry away any heat generated by oxidation and to cool the material. With well graded or fine material having small air voids the air remains stagnant and the heat generated is retained in the mass but when the available oxygen is consumed the heating stops. With intermediate gradings and voids the conditions for spontaneous heating are ideal, and the heated parts may form hot spots and eventually break into flame. Another important factor in the oxidation process is the specific surface of the carbonaceous materials exposed to air. The rate of oxidation generally increases as the specific surface increases, that is as the size of particles decreases.

Prevention

There is no inexpensive method available for preventing spontaneous combustion of existing coal waste piles which are inclined to catch fire. Removal of all carbonaceous material and extinguishing of burning ground, quenching and rebanking in compacted layers are almost the only ways of guaranteeing that fires are eliminated. However, the fire risk can be reduced by close attention to the following points:

removing all vegetation in front of an extending pile;

compacting the waste material and streamlining the outside surfaces of the pile;

prohibiting the placing of flammable materials on the pile (coal, wood, sawdust, sacking, cardboard, paper, waste oil, oily rags, and containers for oil, grease, paint, etc.);

ensuring that boiler ashes are properly quenched before deposition on the pile and prohibiting or controlling fires on the pile;

backfilling and sealing all excavations and boreholes,

inspecting the pile regularly to detect fumes, etc. from hot spots.

Control

1. Methods

The method of controlling a fire on a waste pile will depend upon the size and nature of the pile, the local conditions and circumstances, including the location, extent, and progress of the fire and the availability of suitable extinguishing materials. Experience has shown that the following methods can be used: excavation; trenching; blanketing with inert material; or injection of a slurry of incombustible matter and water.

Water sprays should not be used, except in special circumstances, because of the danger of explosions.

2. Excavation

If the fire is in the initial stage the burning material may be excavated, removed and spread out to cool. This has the disadvantage, however, of disturbing the pile and exposing fresh surfaces to the air and may increase the concentration of fumes. After the burning material has been excavated, the area should be regraded, compacted, covered with inert material and compacted again.

3. Trenching

A trench may be dug into the waste pile to isolate and retard the progress of the fire. The trench may be left open and inspections made to ensure that caving does not occur thereby allowing conduction of heat across the trench. Alternatively, a slurry of limestone dust and water can be pumped into the trench. Even for small fires, the trench should be at least 6 feet wide and 6 feet deep. Other inert materials such as water-softener sludge, or spent lime may also be used. Sand should not be used to fill the trench as it will admit air.

4. Blanketing

Covering the burning area with inert material such as limestone dust, clay, fine sand, or finely-ground shale, is often effective if the blanket is sufficiently thick, completely covers the affected area (particularly the lower parts of the surrounding slopes) and is maintained airtight. Constant inspection and maintenance are required in order to ensure a continuous seal.

5. Injection

Injection of water alone into a burning pile is rarely effective in extinguishing the fire permanently and may lead to explosion, but slurries of incombustible material have been used successfully to control waste pile fires. Clay, shale or limestone dust may be used: however, limestone dust is preferred as on heating it produces carbon dioxide, which does not support combustion. This effect, together with the water in the slurry, may so reduce the temperature that heat is dissipated faster than it is generated. This method has been commonly used in cases where temperatures have been less than 100°C. It has also been used successfully for fires at red heat, but there may be some risk of explosion in such cases.

6. Water Sprays

Water sprays applied continuously to a fire area can be effective as a temporary expedient for damping down waste pile fires, and may thus enable other work to proceed. Spraying may be effective in extinguishing very shallow fires but is not likely to be suitable for deep-seated fires. The use of a jet of water is usually less effective than a spray, and is more likely to cause formation of water-gas with danger of explosion.

Some of the disadvantages of using water sprays are:

the steam formed is inert and therefore is only useful in that it temporarily displaces air; steam has a high latent heat which is released on condensation, bringing the adjacent cooler material to a temperature of about 100°C and producing a condition suitable for the start of further active heating; the condensed water may also be absorbed into the surfaces of neighbouring carbonaceous waste, and the heat of absorption may thus increase the temperature of the waste;

any explosion of water-gas and air may aggravate the problem by opening the waste pile to air and causing the fire to spread;

if the sprays are stopped for any reason, or moved to a new position, air may be drawn into the pile as the water drains away;

the products formed by the oxidation of pyrite may be leached out by water, exposing new surfaces of pyrite;

the use of large quantities of water may adversely affect the stability of the pile; It may also produce a large volume of acidic drainage.

EXISTING EMBANKMENTS

Mine waste embankments may become a threat to lives or property at some time after their abandonment, or during periods when mines are inactive. This may be because of progressive deterioration of the embankment under the actions of water and weather, or because of further urban or industrial development close to the embankments.

Generally, embankments should be put into such a condition before their abandonment that only occasional maintenance will be required to prevent their deterioration to the point where instability can develop. This will usually place emphasis on:

Providing reliable drainage systems, capable of functioning adequately for many years with little maintenance;

protecting embankment slopes against surface erosion and weathering;

preventing spontaneous combustion of coal waste piles.

If adequate provisions are made: to prevent saturation, softening, weathering and chemical change of the materials in the waste embankment; to limit surface erosion; and to prevent the rise of water tables within the embankment; there will be no reason why its stability should decrease below that existing at the time of its completion. Such provisions should be included in the design of the embankments, as already described.

The most appropriate remedial measures to be taken with an existing embankment that is showing signs of instability, or which has become a threat because of adjacent new development, will depend on an evaluation of its existing stability condition and of the cause of the developing instability, if any. Such evaluations require, for both the embankment and its foundation: determination of the existing geometry, both surface and internal; assessment of the existing characteristics of the various materials; and determination of water levels. Surveys can readily determine the surface geometry of the embankment and its foundation. If adequate records have been kept during the waste disposal period, it should be possible to determine from these records, at least the approximate distribution of the various classes of waste or borrow materials within the embankment, and so reconstruct its internal geometry. If such records have not been kept, it may be necessary to drill, sample and test the embankment and its foundation as described in Section 4, to determine material properties and water levels. With sufficient information obtained, the most probable cause of instability can usually be deduced and suitable analyses made to confirm the existing stability condition of the embankment. The most appropriate remedial measures necessary to improve its stability can then be decided.

RECLAMATION

Landscaping

Reclamation of completed mine waste embankments may be required to obtain an acceptable aesthetic relationship between them and their surroundings. This usually involves landscaping of the embankment. Reclamation may provide additional benefits, such as:

- allowing wasted land to be used for agricultural, forestry or other purposes,
- more effective control of erosion and the rapid establishment of equilibrium of the embankment surface,
- reducing the risk of combustion of coal waste pile materials,
- improving stability above minimum requirements,
- facilitating maintenance.

The requirements for reclamation should be taken into account in the design of waste embankments.

To ensure the effectiveness of any landscape treatment given to either new or existing waste embankments, their levels, profiles and surface treatment should be such that they blend unobtrusively with their surroundings or become acceptable elements in the scenery.

In developing schemes for the reclamation of waste embankments, the following points should be considered:

- the reclamation of existing embankments to allow their use for agriculture or forestry, or to provide land for building or industry, may be economically advantageous in the long term;
- reclamation, in the long term, reduces river pollution problems; however, during the shaping of an existing embankment as part of a landscaping scheme, pollution by run-off water from the embankment may temporarily increase;

the removal of material from the side slopes of an embankment may result in a reduction in stability in some circumstances; this should be investigated before the design of any reclamation scheme involving major earthworks in finalized;

minor re-shaping such as the removal of sharp edges, shoulders and corners, and the smoothing of irregular lines, features and surfaces, can considerably reduce obtrusiveness; low embankments with shallow slopes are less obtrusive than high embankments with steep slopes; this is particularly so where the natural topography is flat or gently undulating; curved embankment surfaces are less obtrusive than plane surfaces;

embankment surfaces on which there is a substantial growth of vegetation are already at or near equilibrium; the disturbance of such surfaces should be avoided wherever possible; and test plots are important to determine the limiting factors of growth on specific embankments.

Surface Preparation

Mines are located in areas having widely different climatic conditions and waste embankments are composed of materials of widely different chemical and physical composition. Surface treatments will thus vary considerably and, to ensure success of any proposed reclamation scheme, expert advice should be sought on local soils, vegetation and growing conditions.

Preparation of the surface will depend on whether any agriculture or forestry is intended or possible. If so, it will be necessary to achieve a surface which is loose enough to sustain plant life. Where it is intended to sow grass or plant trees on a waste embankment, the surface layer should be given only minimum compaction; loosening can then be done with a cultivator when and if necessary. If considerable time may elapse between completion of the embankment and sowing or planting, it may be necessary to compact the surface and subsequently scarify it to a depth of about 18 inches, in two directions at right angles, with a heavy rooter immediately prior to planting. An alternative treatment for tree planting is to ridge and furrow the surface at about 6-foot intervals. It has been found in practice that scarified surfaces soon lose their property of absorption after grass seed has been sown, although the sub-surface structure remains loose. Furthermore, the indications are that after the establishment of a sward the increase in permeability of the surface is only marginal.

Slopes for reforestation should neither be steeper than 35 per cent (the maximum slope convenient for men planting forestry transplants) nor flatter than 3 per cent. If trees are to be planted on extensive flat areas where reshaping is impracticable, waste should be tipped over the area with dump trucks. Individual loads should be tipped close together, avoiding compaction, so that the trees have a sufficiently loose material in which to grow and so that there is no standing water. Terraces for tree planting do not encourage satisfactory growth due to lack of drainage and the tendency for their surfaces to become heavily compacted.

Grass can be grown successfully on some waste materials. Conventional agricultural methods of sowing grass can only be reasonably and economically used on slopes flatter than 15 per cent. Hydraulic seeding, however, allows the grassing of much steeper slopes, although, in practice, more satisfactory results are likely to be attained on slopes flatter than 35 per cent. The hydraulic seeding methods permit seeding at a distance above the level of the equipment vehicles and, in order to accommodate these, it is advisable that berms should be included in the profile at intervals to provide slopes not exceeding about 300 feet in length. This is also a convenient length for erosion control on steep slopes. Hydraulic seeding is more expensive than sowing by conventional methods but it has the advantages of speed in places of difficult access and of rapid growth; the various methods may also incorporate protection against erosion.

Soil development occurs primarily from decaying plants in the presence of microbial activity. Seeding on the waste material of any plants that will grow, even if they do not last, will start this process. Topsoil or other suitable soil-making material can be spread on waste embankments prior to seeding if experience shows it to be necessary. Alternatively, in some cases, embankments can be fertilized to achieve desired results. Costs and benefits should influence the selection of the approach to be used.

With high sulphide tailings, a large quantity of neutralizing chemicals are required, to counteract acidity produced by sulphide oxidation, before plant life can be sustained.. Plant nutrients also will be required, as they are with siliceous tailings.

Prior to any cultivation of the surface by agricultural machinery, any stones capable of turning a plough should be removed.

Additional drainage measures for erosion control will be required on slopes before grass has become established or before sufficient growth has taken place on forestry areas; this can be effected by temporary minor drainage ditches cut with a plough, or by hand, and spaced as found necessary, as well as possibly with tile drains.

Cultivating, Fertilizing and Seeding

Fresh coal wastes are normally neutral or slightly alkaline but as weathering proceeds they may become increasingly acidic, depending upon the proportions of pyrite that are exposed to atmospheric oxidation and the amount of alkaline minerals that are available to neutralize the free acidity. High acidity, and high local concentrations of salts of manganese, iron and particularly aluminum, may increase the toxicity of the surface of the weathered material and effectively inhibit or prevent the growth of vegetation.

The process of transforming coal wastes into fertile soil involves:

- neutralizing acidity by liming;
- developing a crumb structure in the top layer by sowing grass or possibly legumes, and building up the humus content and encouraging microbiological life;

maintaining good drainage;

encouraging growth by watching for signs of lime and nutriment deficiency; trace elements should be added if the soils or leaf analysis shows this to be necessary.

The chemical composition of the coal and other waste materials will influence the choice of fertilizer and grass to be used, and advice on these items should be sought from local sources. Experimentation should be carried out, well in advance of the completion of an embankment, to find the most suitable seed for a particular locality. Test plots of typical materials should be fenced off and seeded with recommended strains and fertilized over a period to ascertain the after treatment required. An unseeded and unfertilized control plot should be maintained for comparison.

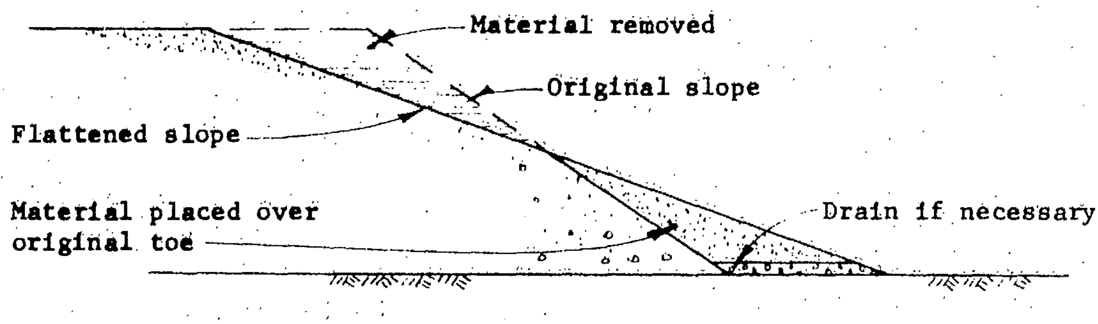
The pH value and the chamical composition of waste materials on which seeding is to take place should be determined and account taken of the effect of atmospheric pollution (if any) and the exposure to which the site is subjected.

Seeding times vary throughout the country and the use of hydraulic, as opposed to conventional, methods permits successful seeding later in the season.

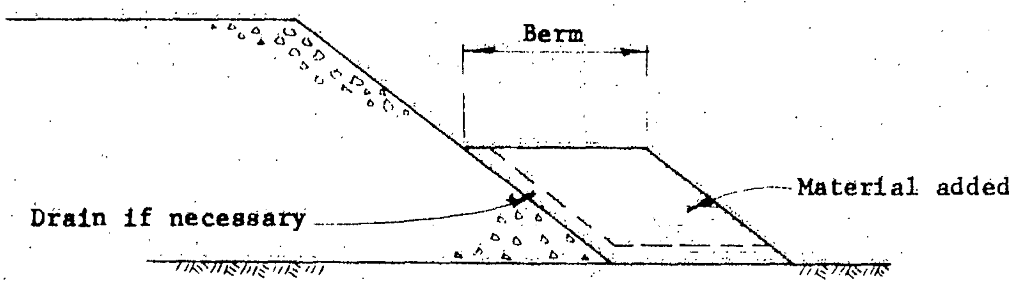
Protection and Maintenance

Protection and aftercare of newly sown waste embankments are important. Attention may be required for 5 to 10 years to ensure the development of a self-sustaining vegetation cover. Wherever necessary, fencing should be erected on the boundaries of seeded areas, even if only for a limited number of years.

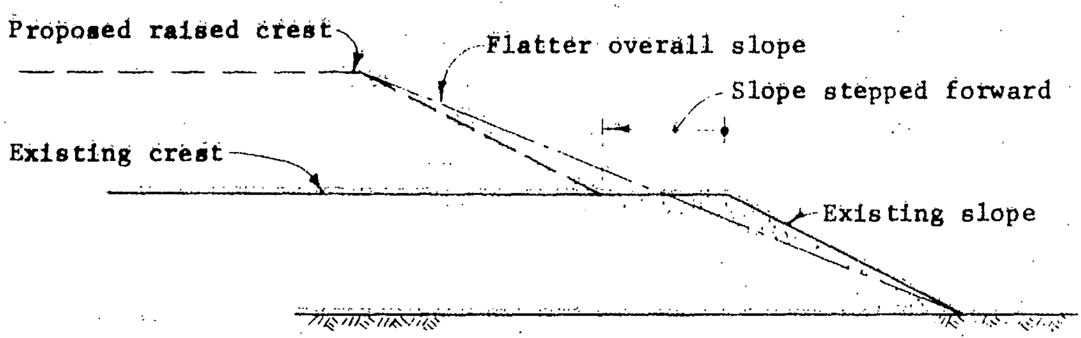
Aftercare and treatment of areas restored to agriculture should not only include further fertilizing, if necessary, but should also include careful management of the land to prevent overgrazing and disturbance of the surface.



SLOPE FLATTENING



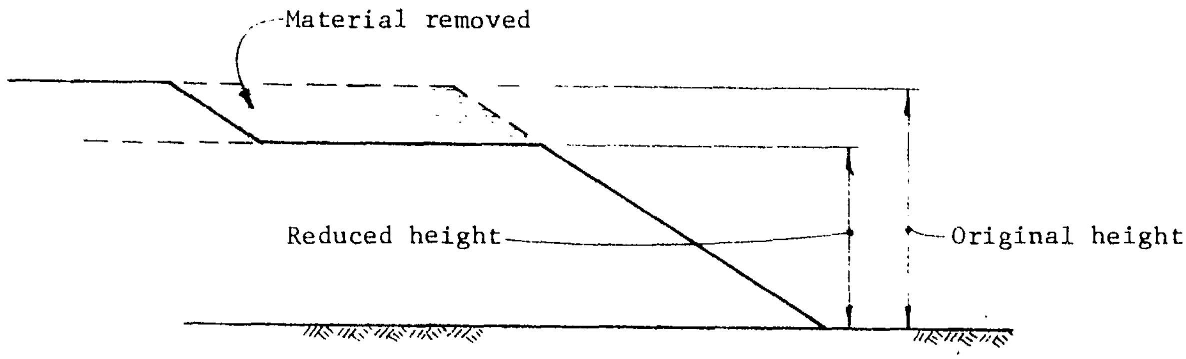
ADDITION OF BERMS



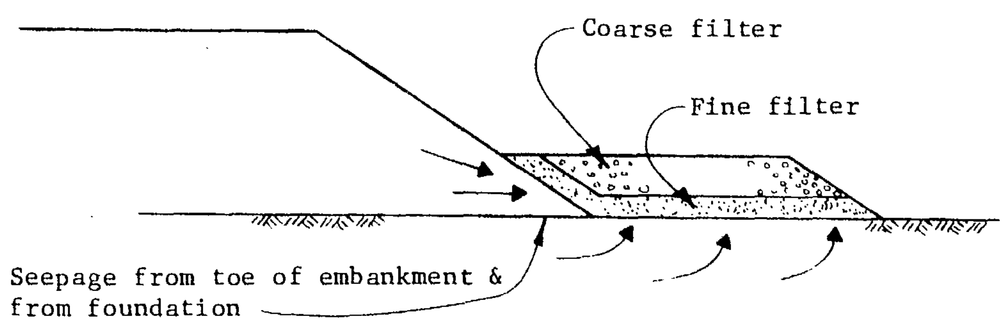
SLOPE STEPPING

EMBANKMENT SLOPE
REMEDIAL MEASURES

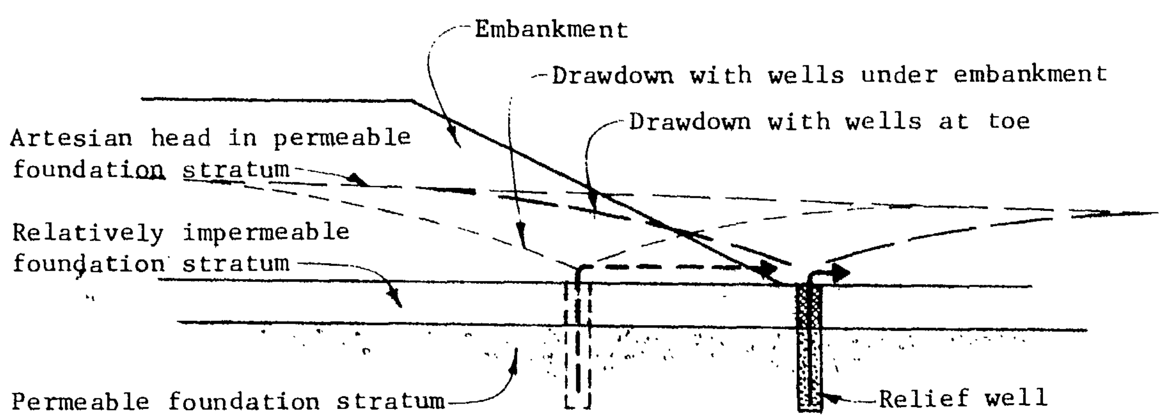
FIGURE 7-1



HEIGHT REDUCTION

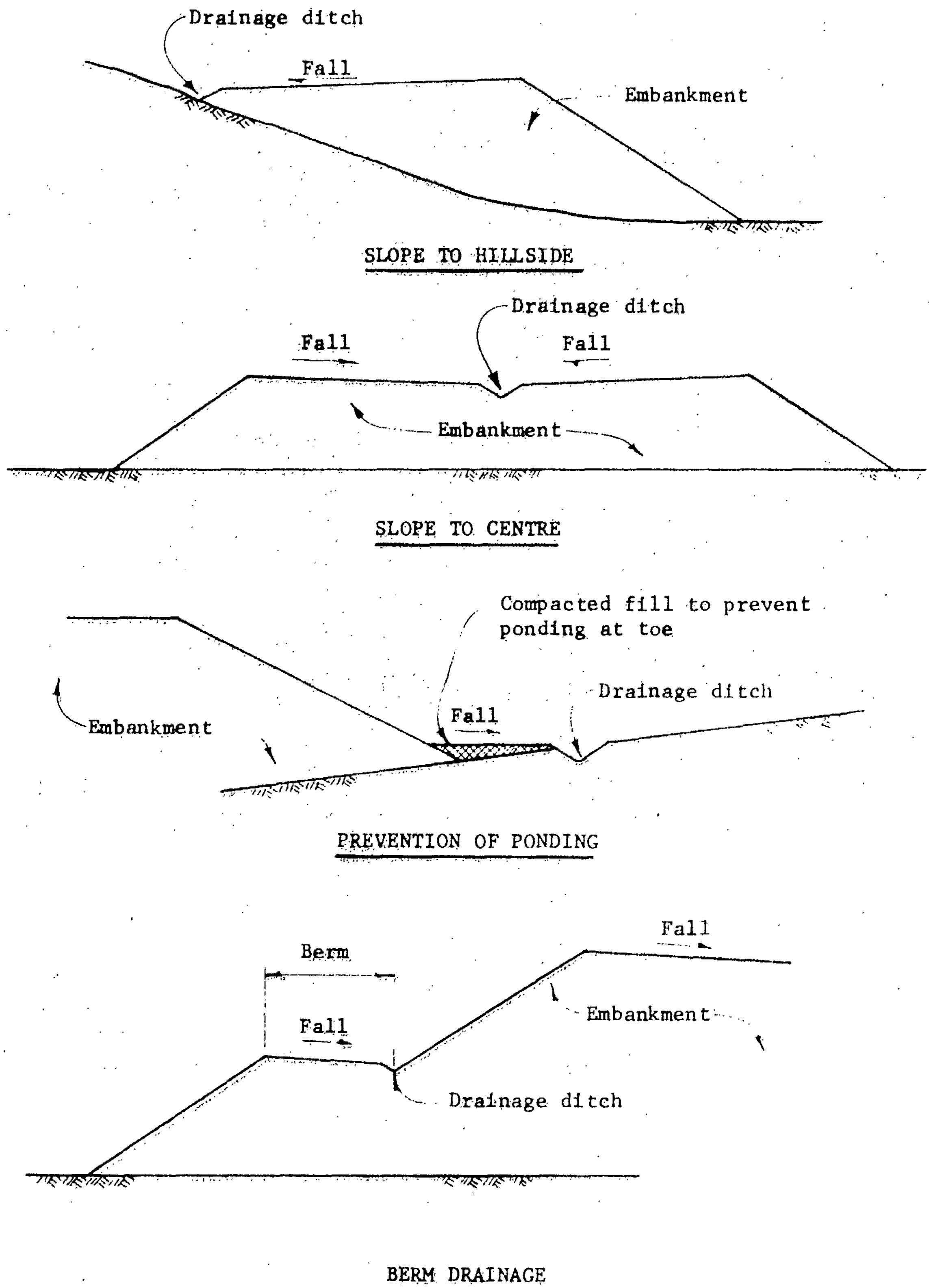


INVERTED FILTER



RELIEF WELLS

EMBANKMENT HEIGHT REDUCTION
AND SEEPAGE CONTROL
FIGURE 7-2



EMBANKMENT SURFACE DRAINAGE

FIGURE 7-3

APPENDIX A

ESTIMATING EVAPORATION

CONTENTS | PAGE

FIGURES

APPENDIX A

ESTIMATING EVAPORATION

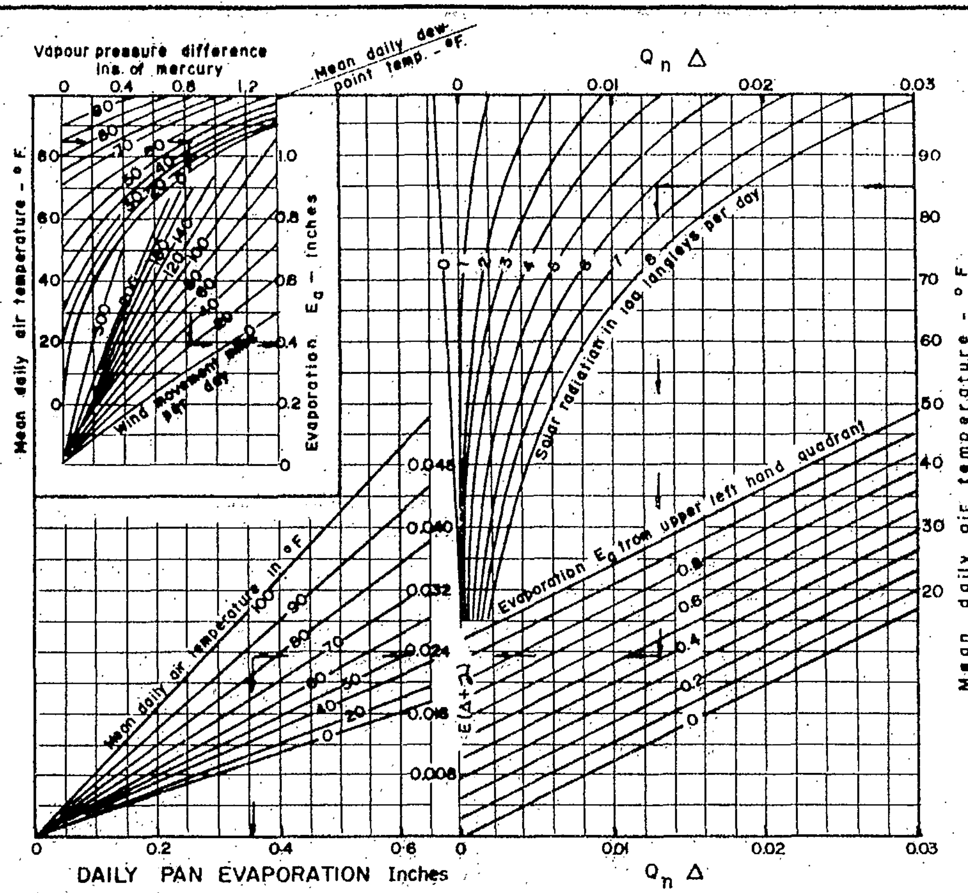
PAN EVAPORATION

The evaporation from Class A pans can be reliably estimated using data on solar radiation, air temperature, dew point and wind movement at the pan site. The basic equations, and graphs and data for their solution, are shown on Figures A-1 and A-5. Winter pan evaporation values missing because of ice cover, and snowfall, can be estimated by this method. Although checks on the reliability of the method have been based principally on daily evaporation data, experience has shown that only minor errors result where monthly evaporation (mean daily value for the month) is computed from monthly averages of the daily values of the parameters. Theoretically, air pressure variations may cause a bias in computed evaporation where the method is applied through a wide range of elevation; however, limited data indicate that it can be used, without appreciable error, for elevations up to 5,000 feet.

LAKE EVAPORATION

The method described above for estimating pan evaporation assumes that the air temperature and the temperature of the water in the pan are the same. However, observations demonstrate that the sensible heat transfer across the pan walls can be appreciable for the Class A pan and that it may flow in either direction, depending on the pan and air temperatures. Since heat transfer through the bottom of a reservoir is essentially zero, the pan data require adjustment. Similarly, the pan does not account for advection (the net heat energy content of inflowing and outflowing water) to, and energy storage in, the reservoir. Such advection could be an important factor in the evaporation from a tailings pond storing water at temperatures above the ambient air temperatures.

Not all energy advected into a pan or lake is dissipated through increased evaporation. Since the increased energy results in higher water surface temperatures, radiation loss and sensible-heat exchange are also affected. Approximate relationships based on water temperature, elevation and wind movement have been established to determine the portion of the advected energy utilized in the evaporation process. Using these relationships, allowances can be made for pan and lake advection and the corresponding pan and lake evaporation values estimated. Equations, graphs and data necessary for the calculations are shown on Figures A-2 to A-5. The procedure involves estimating pond evaporation, making allowance for pan advection (if necessary) and assuming a pan coefficient of 0.7, and then adding the evaporation due to net advection in the pond. This added evaporation must be computed from approximate water and energy budgets, considering flows into and out of the pond (including seepage) and the temperatures of the respective water volumes.



<u>BASIC EQUATIONS</u>

$E \quad = \quad (Q_n \Delta + E_a \gamma) \;/\; (\Delta + \gamma)$

$E_a \quad = \quad (e_s - e_a)^{0.88}\,(0.37 + 0.0041V)$

Where:

E = Daily pan evaporation - inches

E_a = Daily pan evaporation assuming $T_s = T_a$ - inches.

Δ = Slope of saturation vapour pressure vs temperature curve at air temperature T_a.

= de_s/dT_a (See Figure A-5)

Q_n = Net radiant-energy exchange

γ = Coefficient depending on air pressure, taken here = 0.025

<u>NOTE</u>

1 Langley = 1 calorie/sq.cm.

e_s = Saturation vapour pressure at water temperature T_s

e_a = Vapour pressure of air at temperature T_a.

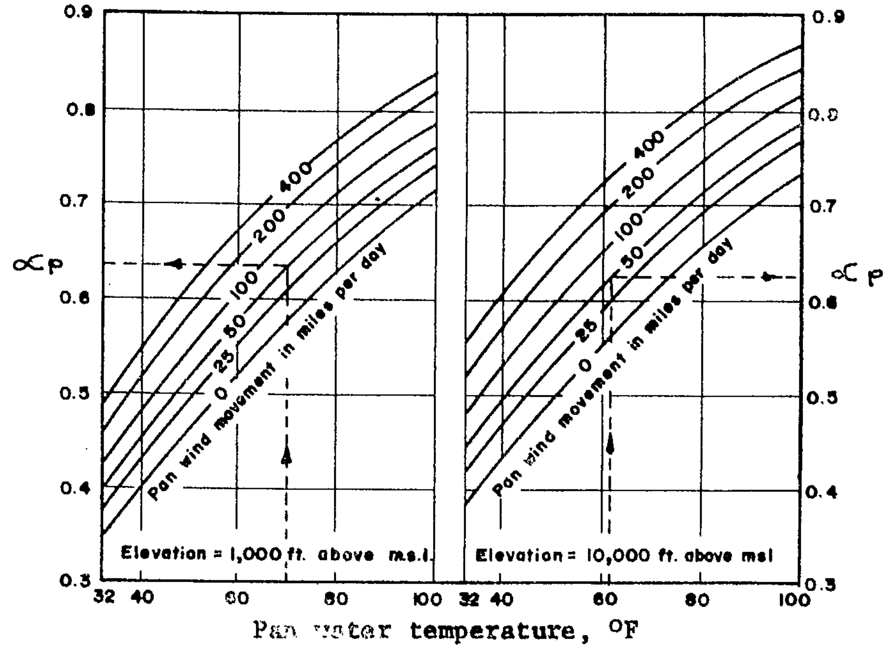
= Relative humidity x saturation vapour pressure

V = Wind movement-miles per day

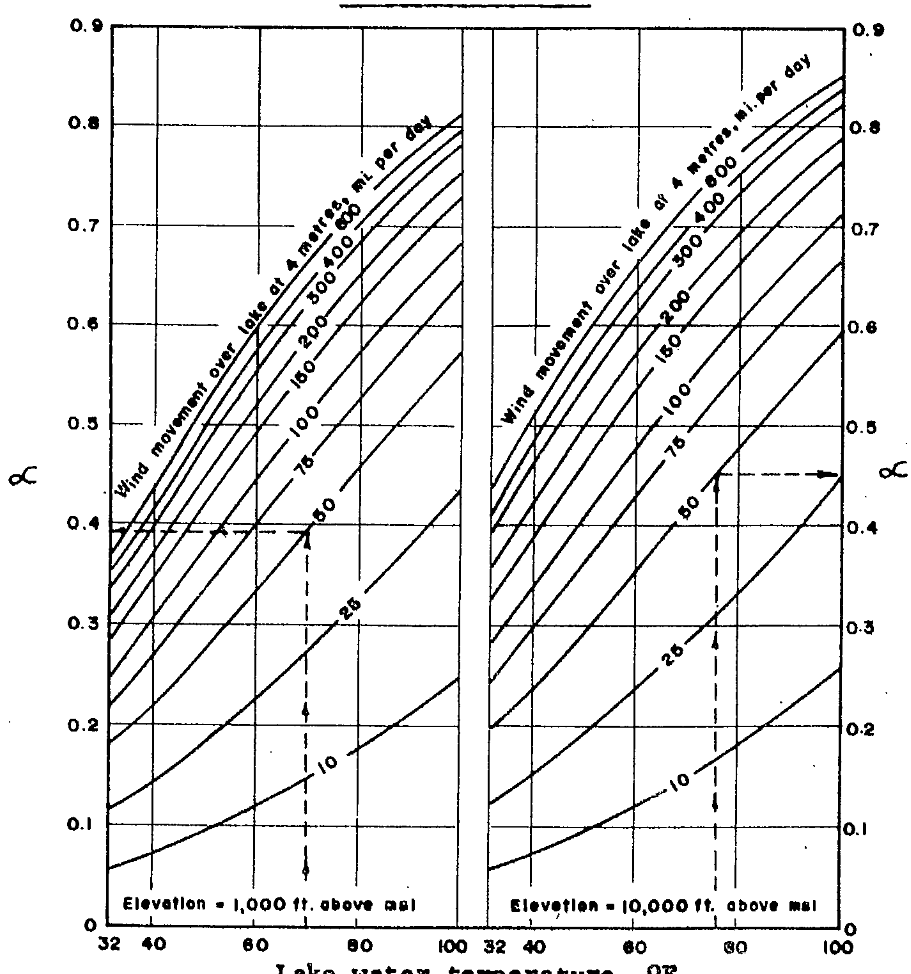
CLASS A PAN EVAPORATION

FIGURE A-1

(U.S. Weather Bureau)



INTO CLASS A PAN

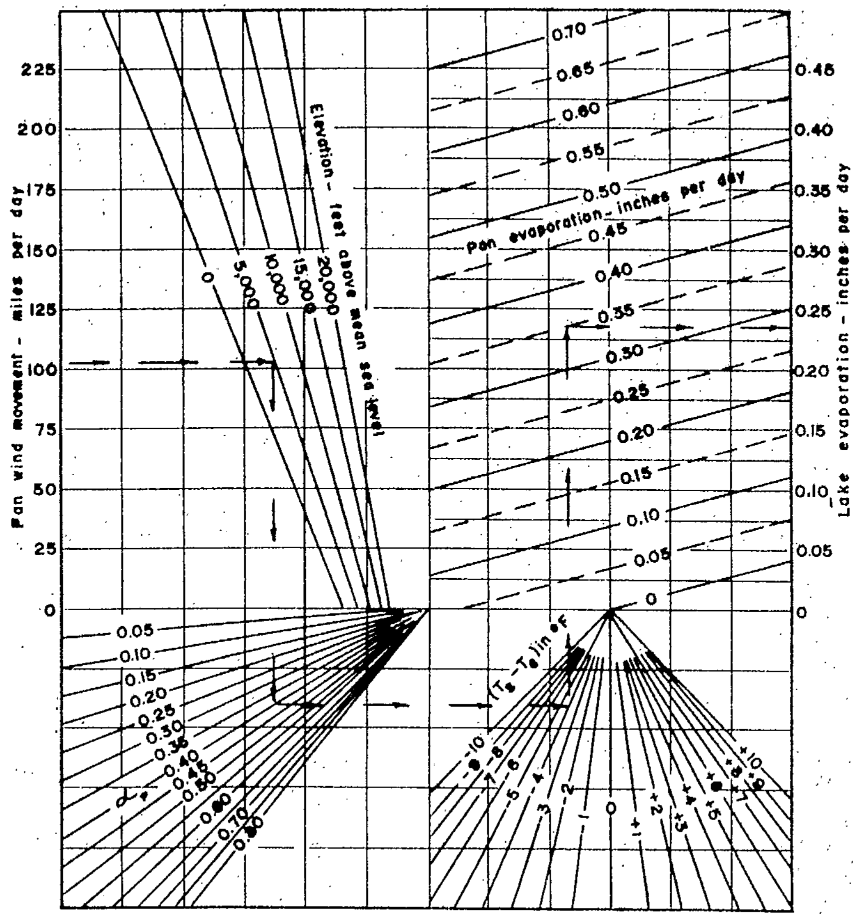


INTO LAKE

PORTION OF ADVECTED ENERGY UTILIZED FOR EVAPORATION

FIGURE A-2

(U.S. Weather Bureau)



(U.S. Weather Bureau)

LEGEND:

T_s = Pan water temperature - ^{o}F.

T_a = Air temperature - ^{o}F.

α_p = Portion of advected energy utilized for evporation from a Class A pan.

LAKE EVAPORATION
EXCLUDING ADVECTION
INTO LAKE

FIGURE A-3

WATER BUDGET EQUATION

$$S_2 - S_1 = I + P - O - O_g - E$$

Where:

S	=	Lake storage volumes
I	=	Surface inflow volume
O	=	Surface outflow volume
O_g	=	Sub-surface seepage volume
P	=	Precipitation volume
E	=	Lake evaporation volume

ENERGY BUDGET EQUATION

$$Q_v - Q_e = (IT_1 + PT_p - OT_o - O_gT_g - ET_e + S_1T_1 - S_2T_2) / A$$

Where:

Q_v	=	Net energy advected into the water - calories per sq.cm.
Q_e	=	Increase in energy stored in the water - calories per sq.cm.
A	=	Lake surface area - sq.cm .
T_1, T_2, etc.	=	Temperatures - ^oC.
Tp	=	Wet-bulb temperature
Tg	=	Water temperature at bottom of lake
Te	=	Lake-surface water temperature

EVAPORATION DUE TO ADVECTION

$$E_e = \alpha (Q_v - Q_e) / H$$

Where:

E_e	=	Lake evaporation - cm .
α	=	Portion of advected energy utilized for evaporation
H	=	Latent heat of vaporization
	=	585 cal ./gm. approx.

LAKE EVAPORATION
DUE TO ADVECTION
INTO LAKE

FIGURE A-3

VARIATION OF DEWPOINT WITH TEMPERATURE AND WET-BULB DEPRESSION AND OF SATURATION VAPOR PRESSURE WITH TEMPERATURE
(PRESSURE = 30 in.)

Air temp. °F.	Saturation vapor pressure Milli-bars	Saturation vapor pressure In.Hg	Wet-bulb depression, °F 1	2	3	4	6	8	10	12	14	16	18	20	25	30
0	1.29	0.038	-7	-20												
5	1.66	0.049	-1	-9	-24											
10	2.13	0.063	5	-2	-10	-27										
15	2.74	0.081	11	6	0	-9										
20	3.49	0.103	16	12	8	2	-21									
25	4.40	0.130	22	19	15	10	-3	-15								
30	5.55	0.164	27	25	21	18	8	-7								
35	6.87	0.203	33	30	28	25	17	7	-11							
40	8.36	0.247	38	35	33	30	25	18	7	-14						
45	10.09	0.298	43	41	38	36	31	25	18	7	-14					
50	12.19	0.360	48	46	44	42	37	32	26	18	8	-13				
55	14.63	0.432	53	51	50	48	43	38	33	27	20	9	-12			
60	17.51	0.517	58	57	55	53	49	45	40	35	29	21	11	-8		
65	20.86	0.616	63	62	60	59	55	51	47	42	37	31	24	14		
70	24.79	0.732	69	67	65	64	61	57	53	49	44	39	33	26	-11	
75	29.32	0.866	74	72	71	69	66	63	59	55	51	47	42	36	15	
80	34.61	1.022	79	77	76	74	72	68	65	62	58	54	50	44	28	-7
85	40.67	1.201	84	82	81	80	77	74	71	68	64	61	57	52	39	19
90	47.68	1.408	89	87	86	85	82	79	76	73	70	67	63	59	48	32
95	55.71	1.645	94	93	91	90	87	85	82	79	76	73	70	66	56	43
100	64.88	1.916	99	98	96	95	93	90	87	85	82	79	76	72	63	52

FIGURE A-5

VARIATION OF RELATIVE HUMIDITY (PER CENT) WITH TEMPERATURE AND
WET-BULB DEPRESSION
(PRESSURE = 30 in.)

Air temp. °F.	Wet-bulb depression, °F.													
	1	2	3	4	6	8	10	12	14	16	18	20	25	30
0	67	33	1											
5	73	46	20											
10	78	56	34	13										
15	82	64	46	29										
20	85	70	55	40	12									
25	87	74	62	49	25	1								
30	89	78	67	56	36	16								
35	91	81	72	63	45	27	10							
40	92	83	75	68	52	37	22	7						
45	93	86	78	71	57	44	31	18	6					
50	93	87	80	74	61	49	38	27	16	5				
55	94	88	82	76	65	54	43	33	23	14	5			
60	94	89	83	78	68	58	48	39	30	21	13	5		
65	95	90	85	80	70	61	52	44	35	27	20	12		
70	95	90	86	81	72	64	55	48	40	33	25	19	3	
75	96	91	86	82	74	66	58	51	44	37	30	24	9	
80	96	91	87	83	75	68	61	54	47	41	35	29	15	3
85	96	92	88	84	76	70	63	56	50	44	38	32	20	8
90	96	92	89	85	78	71	65	58	52	47	41	36	24	13
95	96	93	89	86	79	72	66	60	54	49	44	38	27	17
100	96	93	89	86	80	73	68	62	56	51	46	41	30	21

FIGURE A-6

APPENDIX B

ESTIMATING RUNOFF

CONTENTS PAGE

FIGURES

APPENDIX B

ESTIMATING RUNOFF

ANNUAL AND SEASONAL RUNOFF VOLUMES

Seasonal or annual forecasts of runoff volumes are feasible where snow accumulates during the winter and is followed by a characteristic spring thaw. These conditions exist in many parts of Canada and methods for estimating runoff from snow melt are included in this Appendix. It is feasible also, to estimate the probable return period of high annual or seasonal precipitation values which could occur, by statistical frequency analysis, where long-term precipitation records are available. Methods of making such frequency analyses are described in the references.

A simple plotting of annual precipitation versus annual runoff will often display a high degree of correlation, particularly in areas where the major portion of the precipitation falls in the winter months. In order to be reliable such a precipitation runoff relationship would have to be determined for a catchment basin having characteristics similar to that considered, or a correlation would have to be established between short-term records in the catchment and long-term records available in an adjacent catchment.

Snow surveys in the catchment basin are sometimes used for forecasting the seasonal runoff from snowmelt. Although a good correlation can be obtained between snow-survey data and seasonal runoff, on large catchments it has been found that because of drifting, variations in winter melt from point to point and variations in ground water storage and precipitation during the runoff period, snow cover is usually less dependable than precipitation in providing an index to seasonal runoff.

FLOOD RUNOFF FROM RAINFALL

Maximum Rainfall

The quantity of runoff produced by a stream depends on the moisture deficiency of the basin at the onset of rain and on the storm characteristics, such as rainfall amount, intensity and duration. For small catchment areas of a few square miles, the areal distribution of storm rainfall can be assumed to be uniform, without appreciable error. Often, the intensity of storm rainfall to be expected over the catchment area can be obtained from precipitation records in catchments in the same general area. Climatic maps have been published, also, showing recorded maximum 24-hour precipitation values across Canada. Where records of annual maximum rainfall amounts for a specified duration (say 24 hours) are available, the probable maximum rainfall can be estimated by statistical analysis of the records, being approximately equal to the mean of the series plus fifteen times the standard deviation. This method has been found to yield reliable values in Canada.

Initial Losses

Studies have indicated that a fairly definite quantity of water loss by infiltration is required to satisfy initial field moisture deficiencies before runoff will occur, the amount of loss depending upon antecedent rainfall conditions. Allowances for such "initial losses" are ordinarily made in estimating runoff volumes. They may range from a minimum value of a few tenths of an inch during relatively wet seasons to approximately 2 inches during dry summer and autumn months. The initial loss for conditions usually preceding major floods in humid regions normally ranges from about 0.2 to 0.5 inch and is relatively small in comparison with the flood runoff volume. Consequently, in computing infiltration indices from records of major floods, allowances for initial loss may be neglected or estimated approximately.

Infiltration Indices

The infiltration index is usually defined as an average rate of loss such that the volume of rainfall in excess of that rate will equal the volume of direct runoff. Values computed for a number of drainage basins in the United States, and considered to indicate approximately the minimum indices to be expected during major storms, were within the ranges tabulated on Figure B-1.

Synthetic Unit Hydrographs

A unit hydrograph is one representing one inch of direct runoff from a rainfall of some unit duration (say over 6 hours) and specific areal distribution. The basic premise implies that rainfall excess of two inches within the unit of duration will produce a runoff hydrograph having ordinates twice as great as those of the unit hydrograph. The term "unit-rainfall duration" refers to the runoff producing rainfall, or rainfall excess, that results in a unit-hydrograph. The unit hydrograph resulting from a 6-hour unit-rainfall duration is termed a "6-hour unit hydrograph". The term "lag" is the length of time from the midpoint of the unit rainfall duration to the peak of the unit hydrograph. For small drainage areas the value of the unit-rainfall duration selected for determination of the unit hydrograph should not exceed about one half the lag.

Two basic methods are used for developing unit hydrographs as follows:

- Analysis of rainfall-runoff records for isolated storms occuring over the catchment basin. (This requires rainfall and stream-flow records for the actual catchment basin.)

- Computation of synthetic unit hydrographs from direct analogy with basins of similar characteristics or from indirect analogy with a large number of other basins through the application of empirical relationships. The empirical relations presented by Franklin F. Snyder have proven to be particularly useful in the study of runoff characteristics of drainage areas where stream flow records are not available. The basic equations for deriving a synthetic unit hydrograph by this method are shown on Figure B-2.

The general procedure should be to:

- Analyze such hydrological data as are available for the drainage area to determine approximately the peak discharge, lag and general shape of unit hydrographs (in many instances, fragmentary hydrological data that are not adequate for unit hydrograph derivation in the usual manner may be very useful in connection with synthetic analyses).

- If adequate hydrological records are available, evaluate the coefficients required for the basic equations and use these values in estimating the peak discharge of a synthetic unit hydrograph for the given drainage area. Lacking such records, adopt values based on records for adjacent streams with similar characteristics.

- By a general comparison of the runoff characteristics involved, estimate whether the unit hydrograph peak discharge values computed for the particular area are consistent with values for comparable basins.

Studies have been made to determine the probable degree of accuracy inherent in the use of unit hydrographs derived from records of minor floods, in estimating critical rates of runoff from maximum probable storms. These studies indicated that unit hydrographs required to reproduce the major flood hydrographs had peak discharge ordinates consistently higher than those computed from records of minor floods in which the areal distribution of rainfall was approximately uniform. In the majority of basins considered, the peak ordinates of unit hydrographs derived from major flood hydrographs (representing runoff volumes of water greater than about 5 inches depth from the drainage area) were 25 to 50 per cent higher than values computed from records of minor floods (runoff 1-2 inches). Particularly for small catchment areas, the computed peak discharge should be increased by a percentage of this order, if the unit hydrograph has been based on precipitation values that are small in relation to the probable maximum rainfall in the area.

Methods of adjusting the derived unit hydrograph to accommodate differences in the peak discharge (using empirical relationships relating the widths and heights of the peak of the unit-hydrograph as shown on Figure B-3) are given in the references.

RUNOFF FROM SNOWMELT

General

Unlike rainfall, snowmelt is not a quantity that can be measured directly and, therefore, it must be estimated indirectly from meteorological parameters. In relation to flood hydrograph analysis, this involves primarily the determination of snowmelt rates under various conditions of terrain, vegetal cover and weather. Secondly, it involves evaluation of the effect of the snowpack on runoff. Evaluation of the total volume of runoff during the melt season involves determination of the water equivalent of the snowpack.

Snowmelt During Rain-Free Periods

Estimating snowmelt on a theoretical basis is a problem of heat transfer involving radiation, convection and conduction. The relative importance of each of these processes is highly variable, depending upon conditions of weather and local environment.

The natural sources of heat in melting snow are:

- Absorbed solar radiation
- Net longwave (terrestial) radiation
- Convection heat transfer from the air
- Latent heat of vaporization by condensation from the air
- Conduction of heat from the ground (usually negligible)
- Heat content of rain water.

Solar (shortwave) radiation is the prime source of all energy at the earth's surface. The amount of heat transferred to the snow pack by solar radiation varies with latitude, season, time of day, atmospheric conditions, forest cover and reflectivity (albedo) of the snow. By far the largest variations in the portion of solar radiation transmitted by the atmosphere are caused by clouds, and direct measurement of solar radiation principally reflects the effect of depletion by clouds. Since such radiation measurements are available for relatively few meteorological stations in Canada, it is usually necessary to estimate incoming radiation indirectly from data on the duration of sunshine, observations of cloud cover or diurnal air temperature fluctuations. Methods for making these estimates are described in the references. A nomograph for estimating incoming solar radiation at latitudes below 50°N is shown on Figure B-4. Graphs showing the seasonal and latitudinal variation of solar radiation outside the earth's atmosphere, and the seasonal variation of incident radiation on north-south facing slopes, are included on Figure B-5.

The albedo, expressed as the per cent of reflected shortwave radiation to that incident on the snow surface, is important in estimating the amount of solar energy absorbed by the pack. It may range from more than 80 per cent for new fallen snow to as little as 40 per cent for melting late season snow.

Generalized equations have been developed for snowmelt during rain-free periods, on the basis of various assumptions and requirements for varying conditions of forest environment. These equations are tabulated on Figure B-6. The melt coefficients represent the actual melt of the snowpack, expressed as daily ablation in inches of water equivalent over the snow covered area. The coefficients also express melt for a ripe snowpack. (Isothermal at 32°F and with a 3 per cent free water content.)

Snowmelt During Rain

For clear-weather, springtime snowmelt, energy exchange by the process of turbulent exchange from the atmosphere is of secondary importance compared with radiation. In winter rain-on-snow conditions, however, turbulent exchange is the dominant heat exchange process. It involves the transfer of sensible heat from warm air advected over the snowfield (convection), and also the heat of condensation of water vapour from the atmosphere condensed on the snow surfaces (condensation).

For this reason, solar radiation melt during rainstorms is relatively small and the basic equations can be simplified by assuming an average rate. The simplified equations for estimating snowmelt during rain are tabulated on Figure B-7. Values of wind speed and temperature used in the equations should representative of average conditions over the snow covered area of the basin. (The reductions in wind speed in the forested portions of the basin are accounted for in selection of the basin convection - condensation melt coefficient.)

Effect of Snowpack Condition on Runoff

Runoff analysis for winter or early spring periods requires consideration of the storage effect of the snowpack. This is determined primarily by the conditions of temperature and liquid water within the pack at a given time. Generally, these have to be estimated indirectly from information on changes that have been known to occur. During the natural spring snowmelt period, the snowpack is conditioned to produce runoff early in the period, and there is generally little storage effect after the initial priming has taken place. A "ripe" snowpack is said to be "primed" when its liquid-water-holding capacity has been reached. At this point, the only storage effect is that of "transitory" storage, resulting in temporary delay of liquid water in transit through the pack. There is no restriction on the time of year that the snowpack may yield water to the underlying ground surface. Mid-winter rainfall or snowmelt may satisfy the "cold content" (the heat required to raise the temperature of the snowpack to 0°C) and liquid-water-holding capacity of the snowpack. After those deficiencies have been met, any further input of liquid water at that time will pass through the snowpack as drainage by gravitational force.

Observations have indicated that the liquid-water-holding capacity of snowpacks generally falls within the range of 2-5 per cent of the total water equivalent. For an initially cold (sub-freezing) snowpack, the equivalent cold content (inches of water produced at the surface by rain or snowmelt which, upon freezing within the pack, will warm it to 0°C) can be calculated from the equation given on Figure B-7. This must be added to the liquid-water-deficiency (capacity minus liquid-water content) to obtain the total water required to condition the snowpack to produce runoff. Observations have indicated that there is a diurnal variation of liquid-water content of snowpacks during springtime, the content after drainage during the night ranging from 2 to 5 per cent and being as much as 10 per cent during the day. (Due to melt water in transit).

The time delay to runoff for mountainous areas is in the order of 3-4 hours for moderately deep packs. However, where horizontal drainage is limited, as on the plains, the delay may be much longer.

Runoff Losses

In snow hydrology, it is assumed that no direct runoff occurs until the soil storage is filled to its capacity. For typical mountain soils, the theoretical maximum storage capacity ranges from 4 to 8 inches of water, for the zone from which stored water may be removed by transpiration or evaporation. However, for areas of deep snow accumulation, the soil moisture deficit is satisfied early in the snowmelt period and, in many cases, it is satisfied in the autumn from rainfall or snowmelt. Also, frozen ground will inhibit infiltration.

Loss of water by evapotranspiration can be estimated by various empirical formulae. Generally, evapotranspiration losses from snowpacks in forested areas during the spring melt period can be taken as about 12 per cent of the water equivalent of the snowpack.

Loss by evaporation from the snowpack itself is usually small, observations in the United Stated indicating it to average less than 0.5 inch of water per month, during the winter and early spring. During late spring there is usually condensation on the snow surface.

Ground-water recharge, the source of base stream-flow, usually returns to the stream over a period much longer than that of the direct runoff. For typical mountainous areas of the Western United States, it accounts for about 30 per cent of the snowmelt.

Snowmelt Hydrographs

Unit hydrographs for snowmelt runoff can be derived by trial-and-error techniques through reconstitution of historical data, as described in the references. However, as snowmelt is more or less continuous over a long period of time, it is impractical to derive unit-hydrographs by analysis of isolated short periods of intense runoff.

NUMBER OF VALUES WITHIN VARIOUS RANGES COMPUTED FROM HYDROLOGIC RECORDS FOR NATURAL DRAINAGE BASINS

Range Ins./hr	Jan. Feb.	March April	May June	July Aug.	Sept. Oct.	Nov. Dec.
Northeastern U.S.A. Drainage						
0 - .02		8				
.02 - .05		10	1		6	3
.05 - .10		3	3		8	11
.10 - .15			1		5	7
.15 - .20					2	4
.20 - .25					2	
Over .25			2	1	6	6
Total No.		21	7	1	29	31
North U.S.A. Pacific Drainage						
0 - .02	3	3				1
.02 - .05	4	4	1			4
.05 - .10	2				1	2
Total No.	9	7	1		1	7

(U.S. Army Corps of Engineers)

INFILTRATION INDICES

FIGURE B-1

BASIC EQUATIONS

$t_p = C_t (LL_{ca})^{0.3}$

$t_r = t_p / 5.5$

$q_p = 640\ C_p / t_p$

$t_{pR} = t_p + 0.25 (t_R - t_r)$

$q_{pR} = 640\ C_p / t_{pR} = q_p\ t_p / t_{pr}$

$Q_p = q_p\ A$

Where:

t_p = Lag time of t_r unit hydrograph - hours

t_r = Unit rainfall duration - hours

t_R = Unit rainfall duration other than standard unit t_r - hours

t_{pR} = Lag time of t_R unit hydrograph - hours

q_p = Peak discharge rate of the t_r unit hydrograph - c.f.s./sq.mile

q_{pR} = Peak discharge rate of the t_R unit hydrograph - c.f.s./sq.mile

Q_p = Peak discharge rate of t_r unit hydrograph - c.f.s.

A = Drainage area - square miles

L_{ca} = Stream mileage from site to centre of gravity of the drainage area (to a point opposite the C. of G.)

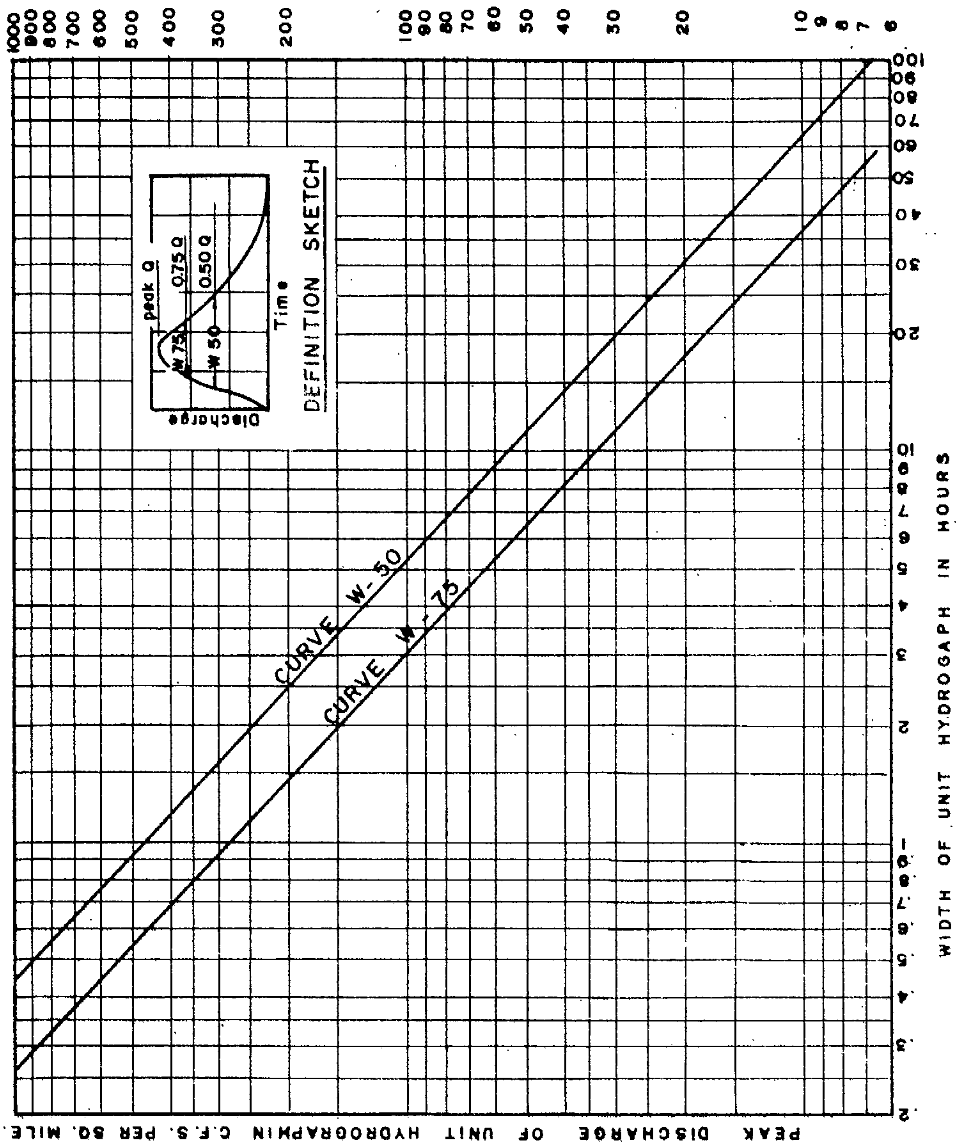
L = Stream mileage from site to upstream limits of the drainage area.

C_t & C_p = Coefficients depending upon units and basin characteristics. Corresponding values of C_t and 640 C_p:

640 C_p: Range: 200 - 600 - Average: 400
C_t: Range: 8.0 - 0.4 - Average: 2.0

SYNTHETIC UNIT HYDROGRAPH
SNYDER'S EQUATIONS

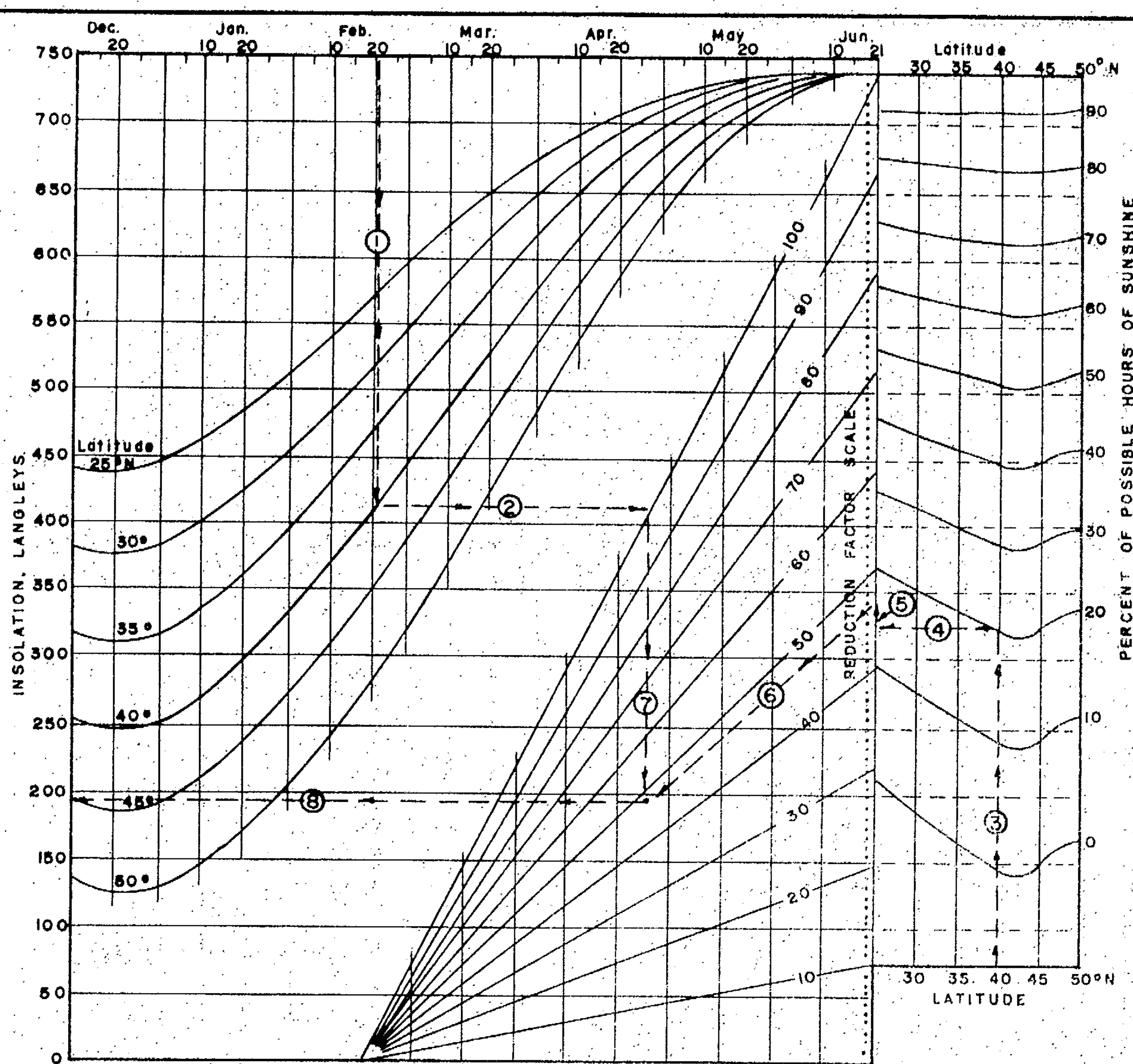
FIGURE B-2



UNIT HYDROGRAPH
PEAKS VERSUS WIDTHS

FIGURE B-3

(U.S. Army Corps of Engineers)



NOTES:

1. The sample shown by dashed lines estimates the daily total insolation at latitude 40°N on February 21, with 20 percent possible sunshine. Consecutive steps are numbered. Step 5 adds the seasonal correction (+2) read from the table. The final estimate is 195 langleys per day.

2. For use between June 21 and December 21, the curves are symmetrical about June 21.

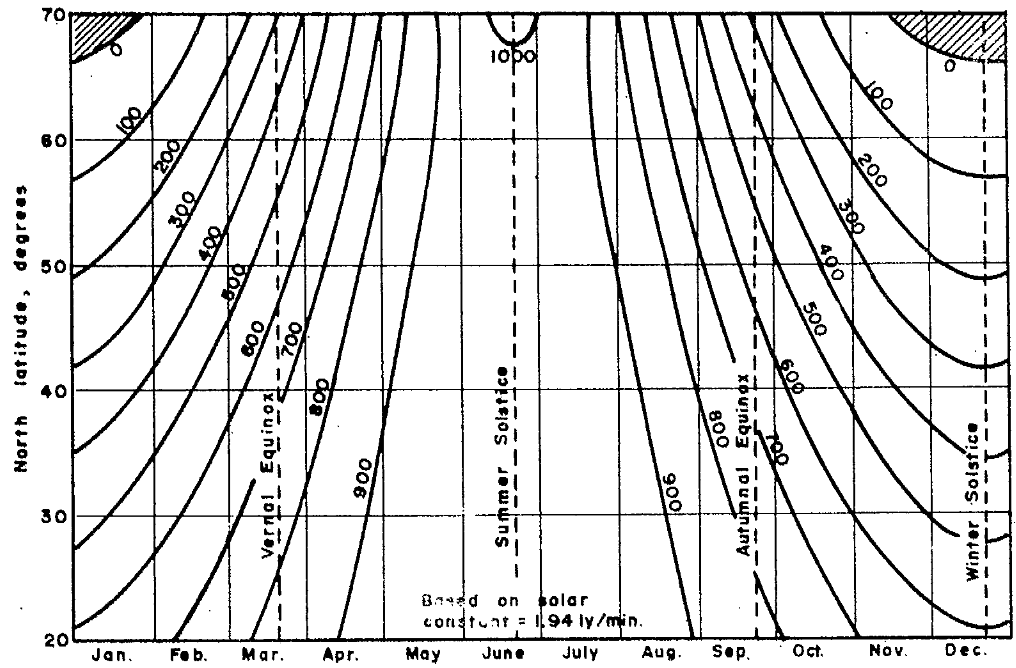
SEASONAL CORRECTION TO REDUCTION FACTOR

	Percent of Possible Sunshine							
Month	0	10	20	30	40	50	60	70
Jan.	+4	+3	+3	+2	+2	+2	+1	+1
Feb.	+3	+3	+2	+2	+2	+1	+1	+1
Mar.	-1	-1	-1	-1	-1	0	0	0
Apr.	-2	-2	-1	-1	-1	-1	-1	0
May	-4	-3	-3	-2	-2	-2	-1	-1
June	-5	-4	-4	-3	-2	-2	-2	-1
July	-5	-4	-3	-3	-2	-2	-2	-1
Aug.	-4	-3	-3	-2	-2	-2	-1	-1
Sept.	-2	-2	-1	-1	-1	-1	-1	-1
Oct.	0	0	0	0	0	0	0	0
Nov.	+2	+2	+1	+1	+1	+1	+1	0
Dec.	+4	+3	+3	+2	+2	+2	+1	+1

SOLAR RADIATION VS. LATITUDE AND SUNSHINE

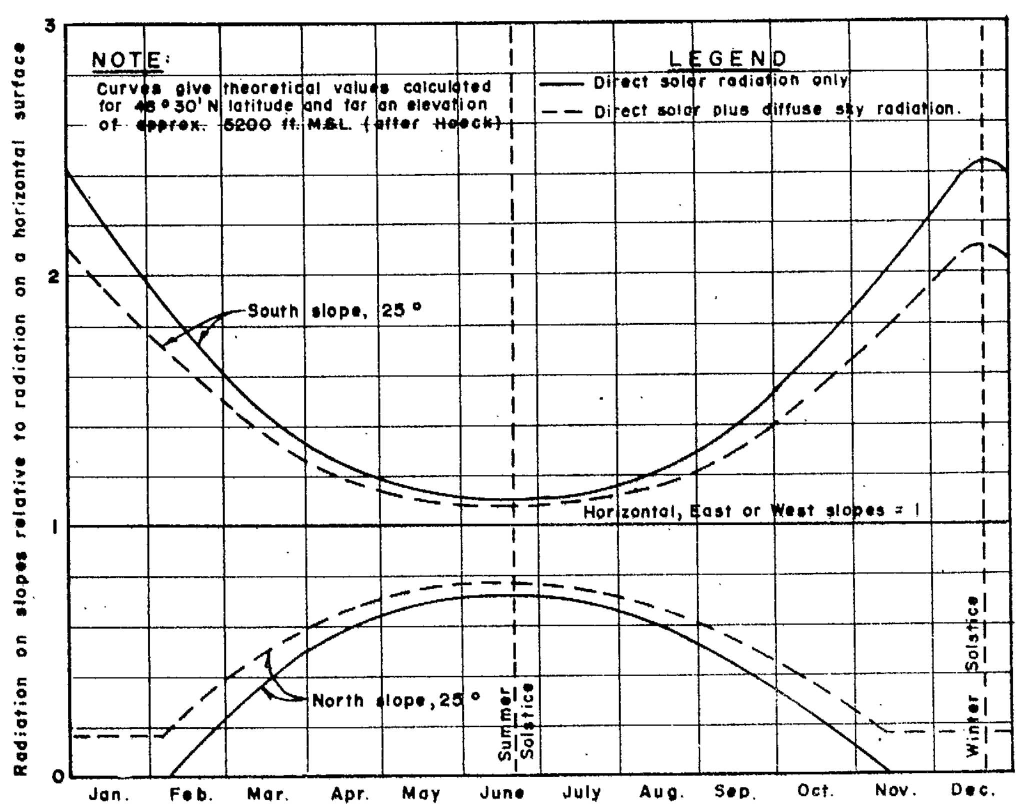
FIGURE B-4

(After Hammon, Weiss and Wilson)



DAILY RADIATION OUTSIDE EARTH'S ATMOSPHERE

(Langleys)



ON NORTH AND SOUTH SLOPES

CLEAR SKY SOLAR RADIATION

(After Hoeck)

FIGURE B-5

BASIC EQUATIONS

Heavily forested area: (>80%)

$$M = 0.074(0.53T'_a + 0.47T'_d)$$

Forested area: (60-80%)

$$M = k(0.0084v)(0.22T'_a + 0.78T'_d) + 0.029T'_a$$

Partly forested area: (10-60%)

$$M = k'(1-F)(0.0040I_i)(1-a) + k(0.0084v)(0.22T'_a + 0.78T'_d) + F(0.029T'_a)$$

Open area: (<10%)

$$M = K'(0.00508I_i)(1-a) + (1-N)(0.0212T'_a - 0.84) + N(0.029T'_c) + k(0.0084v)(0.22T'_a + 0.78T'_d)$$

Where:

M = Snowmelt rate - inches/day

T'_a = Difference between air and snow surface temperatures - ^oF

T'_d = Difference between dewpoint and surface temperatures - ^oF

v = Wind speed in open areas - miles/hour

I_i = Solar radiation on horizontal surface (insolation) - langleys

a = Average snow surface albedo

k^1 = Basin shortwave radiation melt factor - (See Fig. B-5. It would be 1.0 for a basin essentially horizontal or whose north and south slopes are areally balanced. It usually falls within the limits of 0.9 and 1.1 during spring).

F = Average basin forest cover, expressed as a decimal fraction.

T'_c = Difference between cloud base and snow surface temperatures ^oF. (Air temps. drop 3-5 ^oF/1000 feet elevation. Where cloud base is less than 1000 feet, its temperature can be assumed equal to surface air temperature).

N = Cloud cover, expressed as a decimal fraction.

k = Basin convection - condensation melt factor. (See Fig.B-7).

(U.S. Army Corps of Engineers)

SNOWMELT DURING RAIN-FREE PERIODS

FIGURE B-6

SNOWMELT EQUATIONS

Heavily forested areas: (>80%)

$$M = (0.074 + 0.007\ P_r)\ (T_a - 32) + 0.05$$

Open or partly forested areas: (<60%)

$$M = (0.029 + 0.0084\ kv + 0.007\ P_r)\ (T_a - 32) + 0.09$$

Where:

M = Snowmelt rate - inches/day

T_a = Mean temperature of saturated air - ^{o}F.

v = Mean wind speed - miles/hour. For partly forested areas, wind values should be those representative of the open portions of the basin .

P_r = Rate of rainfall - inches/day.

k = Basin convection - condensation melt factor. This allows for basin exposure to wind. It would be 1.0 for unforested plains, but could be as low as 0.3 for densely forested areas .

COLD CONTENT EQUATION

$$W_c = \rho\, DT_s/160$$

Where:

W_c = Cold content equivalent - inches of liquid water.

ρ = Snow density - grams/cubic centimetre.

D = Snowpack depth - inches.

T_s = Average snowpack temperature deficit below $0^{o}C$.

SNOWMELT DURING RAIN AND SNOWPACK COLD CONTENT

(U.S. Army Corps of Engineers) FIGURE B-7

APPENDIX C

MEASUREMENT SYSTEM CONVERSIONS

GENERAL

Differing systems are used in civil and mining engineering in defining the properties of soils and wastes. The civil engineering system has been used throughout this Guide since practically all data on soils are based on this system. Formulae showing the relationships between various common measurements in the two systems are given in this appendix.

WATER CONTENT (W)

Civil: Wc per cent = 100. weight of water/weight of solids

Mining: Wm per cent = 100. weight of water/total weight

Conversion: Wc = 100 Wm/(100-Wm)

VOID RATIO, POROSITY, SATURATION, CONCENTRATION

Civil: Void Ratio (e) = V_v / V_s

Where: V_v = Volume of voids
V_s = Volume of solids

Porosity (n) = V_v / V

Where: V = Total volume

Degree of Saturation (S per cent)
= 100 V_w / V_v
= $Wc/(\gamma w/\gamma d - 1/G_s)$

Where: Vw = Volume of water
γw = Density of water
γd = Dry density of soil
G_s = Specific gravity of solids

Mining: Volume Concentration (C per cent) = 100 Vs/V

Weight Concentration (Cw per cent)
= 100. weight of solids/total weight

Conversions: $e = n/(1-n) = G_s W_c/S$

$c = 100/(1+e)$

$Cw = CG_s/Gm$

$Gm = (1-C)/(1-Cw)$

Where: Gm = specific gravity of mixture.

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