PIT SLOPE MANUAL

chapter 9

WASTE EMBANKMENTS

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PIT SLOPE PROJECT

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Energy, Mines and Resources Canada

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THE PIT SLOPE MANUAL

The Pit Slope Manual consists of ten chapters, published separately. Most chapters have supplements, also published separately. The ten chapters are:

1. Summary
2. Structural Geology
3. Mechanical Properties
4. Groundwater
5. Design
6. Mechanical Support
7. Perimeter Blasting
8. Monitoring
9. Waste Embankments
10. Environmental Planning

The chapters and supplements can be obtained from the Publications Distribution Office, CANMET, Energy, Mines and Resources Canada, 555 Booth Street, Ottawa, Ontario, K1A OG1, Canada.

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FOREWORD

Open pit mining accounts for some 70% of Canada's ore production. With the expansion of coal and tar sands operations, open pit mining will continue to increase in importance to the mineral industry. Recognizing this, CANMET embarked on a major project to produce the Pit Slope Manual, which is expected to bring substantial benefits in mining efficiency through improved slope design.

Strong interest in the project has been shown throughout its progress both in Canada and in other countries. Indeed, many of the results of the project are already being used in mine design. However, it is recognized that publication of the manual alone is not enough. Help is needed to assist engineers and planners to adopt the procedures described in the manual. This need for technology transfer will be met by a series of workshops for mine staff. These workshops will be held in various mining centres during the period 1977-81 following publication of the manual.

A noteworthy feature of the project has been its cooperative nature. Most organizations and individuals concerned with open pit planning in the country have made a contribution to the manual. It has been financed jointly by industry and the federal government.

Credit must be given to the core of staff who pursued with considerable personal devotion throughout the five-year period the objectives of the work from beginning to end. Their reward lies in knowing that they have completed a difficult job and, perhaps, in being named here: M. Gyenge, G. Herget, G. Larocque, R. Sage and M. Service.

D.F. Coates
Director-General
Canada Centre for Mineral and
Energy Technology

SUMMARY

GENERAL

The term "waste embankments" includes tailings dams, rock dumps and overburden or soil dumps. The design specifications in the manual deal primarily with stability with some consideration given to the effects on natural water courses and groundwater. Reclamation aspects are dealt with in the chapter on Environmental Planning.

Design for stability must take into account the strength of potentially weak foundations, the maximum slope angle that will be stable in the long term even after the mine has shut down, and adequate natural drainage to prevent destabilizing water effects. Provision for drainage must be explicitly designed as otherwise seeping water could cause softening of the embankment and internal erosion leading to breaching. For tailings dams, the requirements for water retention must also be considered. These include the appropriate amount of freeboard and crest width. For coal wastes, spontaneous combustion is another form of instability that must be prevented.

SITE INVESTIGATIONS

The site of a waste embankment will require some of the following ingredients of an investigation program:

a. engineering and geological surveys,
b. foundation testing,
c. borrow pit testing,
d. waste material testing, and
e. topographic mapping.

Besides the mechanical properties needed for stability analyses, other properties will be of interest. Sedimentation properties of tailings will influence the retention time and depth of water required in the tailings ponds. The oxidation potential, particularly of sulphide minerals, is important when considering the possible effects of seepage on natural water courses. The ease with which coal in waste will combust spontaneously can be tested so that the need for special design requirements can be determined. The nature of the processing reagents expelled with the tailings effluents is also important in determining potential effects on natural water courses.

Climatic and ecologic data are required for tailings dams and revegetation planning. Air temperature, precipitation, wind, solar radiation and evaporation, measured at various stations across the country, are published by the Atmospheric Environment Service of the Department of Environment, Ottawa. Maps are available showing contours of temperature, humidity, wind, rainfall and snowfall. Records of measured streamflows are published by the Water Survey of Canada, Department of Environment.

Subsurface investigations are directed toward determining the nature of soil deposits overlying the bedrock. The thickness and composition of each strata are required. Information on groundwater regime and its variation over the years are desirable. Details of any old, current or future underground mine workings are important. Similarly, it is valuable to know of any solution caverns.

DESIGN

Rotational sliding is the typical mode of instability for waste embankments. The cause might be slopes that are too steep for the type of material. Alternatively, it could result from overloading a weak foundation as the embankment is built up. The rise in water level in the embankment is often the triggering mechanism for instability.

Sliding on a sloping foundation sometimes occurs when the foundation is composed of weak clay or silt or has a thin layer of altered material on top of bedrock.

The appropriate analysis techniques to evaluate stability are described in detail in the Design chapter. Except for foundation problems, embankment design has the advantage that the nature of construction material and the construction procedure are known to the engineer. Stability calculations, therefore, have more certainty than do analyses for mine slopes.

Liquefaction occurs when tailings in an embankment or pond become temporarily suspended in water and flow like a viscous liquid. When tailings dams are breached, liquefaction of the saturated slimes often occurs. In the embankment itself, if the tailings are in a loose state and saturated, vibration from earthquakes or trucks might cause liquefaction. Design must anticipate this, and if necessary, specify the deposition procedures or sequence or require drainage to avoid it.

Overtopping of a tailings dam can lead to breaching and loss of control of the tailings pond; this is probably the most common cause of dam instability. Design requires determining the appropriate freeboard and width of crest. Using construction material that is less subject to erosion than normal tailings may be necessary.

Erosion of embankment slopes can lead to instability. In tailings dams, the concentration of water flow at reentrant corners and other changes of contour, if not controlled, can lead to piping and ultimately to breaching of the dam.

Piping is the transporting of solid particles in seepage water passing through or under the dam. When sufficient material has been carried out by the water, a pipe may be formed extending progressively under the embankment until an excessive flow of water occurs, causing breaching of the dam.

Differential settling beneath an embankment may lead to fracturing of nearby pipes. Settling in tailings dams may crack the impervious seal which must be anticipated in the design.

Coal waste piles can be ignited by spontaneous combustion. Poisonous gases can be generated and the dump become very difficult to handle. In attempting to extinguish such a fire, the generation of coal dust could lead to an explosion. Design and construction of such dumps should take this hazard into account.

Seepage through tailings embankments should be inhibited and drawn into drains where it can be controlled. In this way, stability of the slopes is maintained and any environmental impact can be controlled.

The catchment area behind a tailings dam can be quite extensive. Consequently, relocation of streams, ditches and dikes should be considered as means to isolate the tailings pond from the adjacent catchment area. The storm flood that must be designed for is considerably reduced in this way, and the quantity of seepage effluent that might have to be treated is minimized.

CONSTRUCTION AND OPERATION

Tailings dams are commonly constructed by one of three methods - downstream, upstream, or fixed centreline. Stripping of organic matter and the cutting off of seepage contribute greatly to stability of the embankment.

A maintenance program is important after an embankment has been constructed, particularly for tailings dams. The conditions under which they exist change continuously. Problems can develop very quickly under intense operating conditions, particularly in the season of high precipitation. If severe erosion is anticipated, riprap or vegetation should be used to counter the problem.

The monitoring of crest movement and groundwater, particularly in tailings dams, gives early warning of unsatisfactory conditions developing. Such measurements should be an integral part of the mine monitoring program.

PERMAFROST

Placing waste embankments on permafrost can produce results that are not experienced in other areas. The permafrost line may rise into the embankment and result in slope instability if ice lenses are created that thaw in the summer. On the other hand, the permafrost line may recede further into the foundation, depending on the thermal regime, which could give rise to excessive settling if the thawed material is clay or silt. Embankments placed on permafrost slopes can be subjected to special conditions for sliding, groundwater flow, and ice formation. Correcting these problems can be very difficult so that it is better to anticipate the difficulties and take appropriate measures to avoid them.

ACKNOWLEDGEMENTS

D.F. Coates was responsible for production of this chapter. Address enquires to him at: 555 Booth St, Ottawa K1A 0G1 Canada.

The original version of this chapter appeared as "Tentative Design Guide for Mine Waste Embankments in Canada", TB 145, 1972. It was written by Wright Engineers Limited (K.L. McRorie, A.L. Galpin, J.C. Bowling), Golder, Brawner & Associates Limited (H.Q. Golder, C.O. Brawner, D.B. Campbell) and Ripley, Klohn & Leonoff International Limited (C.F. Ripley, E.J. Klohn, M.T. Olsen). Many companies and individuals assisted in the preparation of the final report. This Technical Bulletin was then revised by B. Hoare in 1975 and consolidated into its final form by D.F. Coates. Y.S. Yu did the technical editing on the chapter.

The Pit Slope Project is the result of five years research and development, cooperatively funded by the Canadian mining industry and the Government of Canada.

The Pit Slope Group staff have been D.F. Coates*, M. Gyenge*, G. Herget, B. Hoare, G. Larocque, D. Murray and R. Sage*.

*successive Pit Slope Group Leaders

CONTENTS

FIGURES

TABLES

INTRODUCTION

PURPOSE AND SCOPE

1. The trend in Canada is towards the mining of lower grade ores than those worked in the past and consequently, waste embankments are also getting bigger and higher. Problems have been encountered with mine waste disposal and, in 1969, the Canadian Advisory Committee on Rock Mechanics recommended that:

2. "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 the 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."

3. A design guide was prepared and issued in 1972 as a result of that recommendation; this chapter constitutes the second edition.

4. Particular emphasis is placed on the ability of waste embankments to retain solid and liquid wastes. Their adequacy in preventing pollution of the environment has not been treated specifically as each mine property requires specific engineering studies for these problems. Chapter 10 of the Pit Slope Manual provides some guidance on environmental planning.

MINE WASTE EMBANKMENTS

5. Mine waste consists of various classes of materials. These include soils from strip and placer mining operations, excavated rock, ground ore in slurry form and fine wet material such as filter cake from coal treatment plants. Their characteristics will vary considerably according to geological origin, particle size, previous processing and water content. Some waste may be used for backfilling mined out spaces; the bulk of it is dumped in areas adjacent to the mine. A typical mine waste disposal system is illustrated in Fig 1.

6. As used herein, "waste embankments" include all mine waste materials placed on surface. The term does not include material placed underground as backfill.

7. "Tailings" refer to the waste product from a milling operation in which the valuable minerals have been recovered.

8. "Tailings embankments, dams or dikes" retain tailings in slurry or solid form. They may be constructed of either tailings, waste rock, natural soils or a combination of these materials.

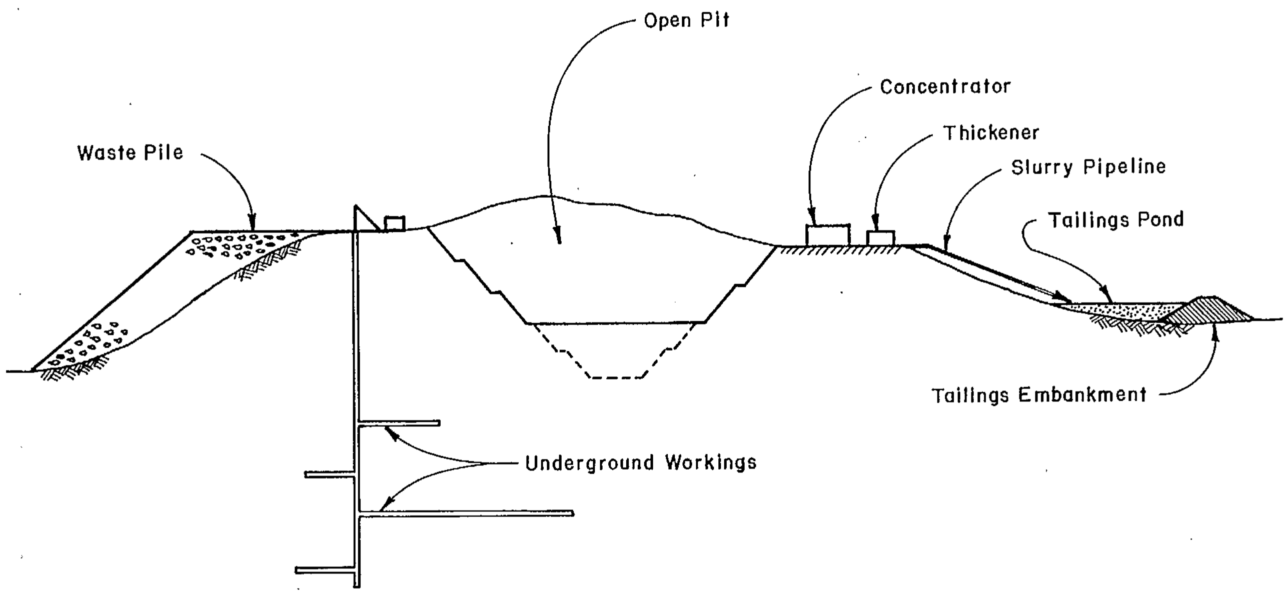


Fig 1 - Typical mine waste disposal system

9. "Waste piles" include all waste embankments other than tailings or ore resulting from a mining operation.

10. For purposes of structural design, "ore piles" are treated similarly to waste piles.

BASIC FUNCTIONS

11. The primary function of a waste embankment is to store the waste material from a mining operation in a designated area and to ensure public safety.

12. Where it is important to prevent water pollution emanating from these materials, extra design provisions will be required. For example, a tailings embankment retaining a slurry may have to include a relatively impervious seal to restrict seepage. Where the quality of the tailings effluent constitutes a serious pollutant, a system may be required to reclaim the effluent water for plant processing purposes. Alternatively, it may be necessary to treat the water prior to releasing it to natural water courses. Similarly, it may be necessary to reclaim or treat the seepage water which issues from the tailings embankment.

13. In some cases, it may be desirable to promote drainage of the tailings slimes stored behind the embankment. This might be the case when reclaiming fine coal tailings or when stabilizing tailings slimes subject to seismic activity. In such cases it will be desirable to include explicit drainage requirements in designing the tailings basin.

14. Technological advances in extractive metallurgy and variations in market prices may make it expedient to reclaim waste material. When this is stored relatively close to the mining facility it is accessible for potential reprocessing operations. Large volumes of tailings can be reclaimed economically by dredging. In the special case of reclaiming fines from settling ponds at a coal preparation plant, it may be desirable to provide internal drainage to promote rapid drainage of the water and facilitate recovery of the coal.

15. The function of the tailings pool is to provide sufficient retention time for the solid particles to settle. Therefore, the pool depth, surface area, distance between the point of tail-

ings discharge and effluent discharge, and wave action are critical parameters. Extremely small particles settle slowly, and in some instances it may be desirable to add flocculating agents to accelerate settling. The pH of the liquid will also have a significant effect on the rate of settling of fine particles. As the volume of solids accumulate in the bottom of the pool, it will be necessary to adjust the elevation of the pool surface to maintain the desired degree of clarification. Wherever possible, the effluent should be reclaimed for plant process water to conserve this valuable natural resource and to minimize the effects of the effluent on the natural water courses.

16. Initially, particles settling from the liquid in the pond will accumulate in a very loose state, having high void ratio and high water content. These loose deposits will consolidate with time as liquid is expelled from the voids between the particles. The rate of consolidation is influenced by such factors as the effective pressure of the overlying material, permeability of the deposit and surrounding soil and rock, distance the liquid must travel to drain and the time during which vertical pressure has been applied.

17. The amount of liquid expelled 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 in tailings that are finely ground, have a high clay content or are of low specific gravity. Even after consolidation under their own weight for many years, such deposits may have high void ratios and high water contents. This condition, which has an appreciable effect on the shear strength of the tailings within the basin, will continue for an extended period unless additional measures are taken to promote further drainage of liquid.

PROBLEMS ENCOUNTERED

18. There have been serious waste embankment failures. Sliding of embankment slopes can occur in waste piles and tailings embankments. Such failures can be caused by weak foundations, placing of waste material at slopes that are too steep or too high and high pore water pressure within the embankment or its foundations.

19. Breaching of tailings embankments can occur as a result of over-topping by water in the pond, or by piping of fine material under the action of seepage through the embankment or its foundation. Piping is the result of erosion which starts at the point of exit of a flow line that has passed below or around the embankment or its foundation; the erosion may progress backward aided perhaps by the presence of a flow channel. Piping into decant culverts installed under tailings embankments has been a common problem.

20. Burning of coal waste piles is a hazard because of the generation of noxious gases. Other chemical changes such as oxidation of pyrite can cause acidic runoff from waste piles.

INVESTIGATIONS

21. The extent of any investigation program will depend on possible consequences of failure, size of the embankment, and complexity of the site conditions.

22. For tailings embankments 25 ft (7.5 m) high or more, the investigations would normally include: a. engineering and geological surveys, b. foundation and borrow pit testing, c. waste material testing, and d. topographic mapping.

23. A detailed engineering and geological inspection of the site would include delineating old underground mine workings, locating streams, dwellings, and other nearby installations, identifying available sources of construction materials, determining the waste and foundation material characteristics and types of embankments suitable for the site conditions and associated mining operation. Such geological information must be known as: the location of rock outcrops in relation to the embankments; probable location of bedrock in the foundations and abutments; and dip and strike of stratified rock exposures. In some cases, determining bedrock location by geophysical methods may be advantageous.

SITE AND MATERIAL INVESTIGATIONS

GENERAL

24. An efficient design for waste embankments must consider the cost of alternative methods of waste disposal and of alternative construction material for the retaining dams. Construction procedures regarded as standard practice in producing stable highway, dam, and other embankments in civil engineering may represent a substantial item of cost when applied to mine waste embankments. However, the increasing size of embankments in current mining operations makes it important that stabilization procedures, such as compaction and seepage control, be used to the necessary extent. Procedures such as raising tailings dams with flashboards and spigotting may not be appropriate for high dams, particularly when the materials are finely ground.

25. The ultimate volume of waste material will be a principal factor in selecting disposal area. When it is necessary to build a perimeter dam on a flat area, the largest area with the lowest dam will provide the maximum ratio of storage volume/dam volume. Points of discharge can affect storage volume. If the tailings are discharged at a remote point from the dam, in some cases the tailings may be stored to heights considerably above the crest of the dam.

26. Waste piles can be built from the top down or from the bottom up depending on local topography. When waste material is dumped over the side of a cliff or mountain, a careful foundation investigation should be made and the necessary foundation preparation should be carried out to assure adequate stability. A lower waste pile may ensure greater stability; however, a higher waste pile may be more economical. The required rate of disposal may also affect design of the embankment because the strength of many foundations increases with time, being related to the rate of consolidation and permeability of the material.

27. The effects on stability of the physical and chemical properties together with handling characteristics of the material are important in the design of mine waste embankments. Some fine wastes may be so wet when discharged that they cannot be mechanically compacted and must be re-

tained by coarser wastes or by compacted embankments. Even after two-stage cycloning, the coarser underflow from the tailings cyclones may have a high content of slimes, making it unsuitable for efficient dam building.

28. The principal climatic effects on the design of a tailings dam are the short-term peak flood flows from rainfall and runoff, and the extent of possible frost damage. A tailings basin should be designed to handle peak flood flows and to maintain a minimum depth of water in the pool to settle the solids. In winter it is necessary to increase the depth of the pool by an amount slightly in excess of the expected ice thickness to maintain the necessary depth of water for clarification.

29. It is not practical to use hydraulic methods for constructing tailings dams during freezing weather conditions. The dams should be scheduled for construction during the summer with sufficient freeboard for winter operation.

30. Downstream slopes of tailings dams can freeze creating an impervious downstream toe which restricts the exit of seepage water. During this period there can be a buildup of pore water pressure within the toe, causing a toe failure or sloughing in the spring. To overcome this problem, the seepage should be carried outside the structure in drains below the frost line.

31. The principal climatic effects influencing the stability of mine waste piles are the magnitude of precipitation and frost conditions. Runoff from rainfall or snowmelt may cause surface erosion or a buildup of pore pressure in waste piles of relatively fine-grained material. The freezing of fine wet wastes during winter placement will have an effect on stability when they thaw. Snow layers buried in the waste pile will produce local surfaces of weakness along which sliding may occur.

32. Topography can have a major influence on operating costs and methods of waste disposal. Waste piles can be constructed from the bottom up or from the top down depending on topography. In some cases a high escarpment will allow dumping of wastes downhill to form a high pile of relatively limited extent. Provided the foundation material is competent, coarse pervious waste piles will remain stable at the angle of repose. In some instances the piles may become impervious by weathering and the buildup of pore water pressure could create an unstable condition. When material is dumped from the top of an embankment, it settles beyond the natural angle of repose and a series of minor slip failures may occur along the crest. Safety precautions must therefore be exercised in the method of dumping to prevent accidents.

33. The ratio storage volume/dam volume is an indication of the efficiency of a tailings disposal system. If only a small flat area is available and a complete perimeter dam is required, the ratio will be low. Alternatively, in rugged terrain where it is only necessary to construct a dam across one end of a valley, the ratio can be very high.

34. The geological features of principal interest in the design of waste piles and tailings embankments are those affecting shear strength, compressibility and permeability of the foundation. Critical features include organic materials, soft strata or seams, sources of seepage, and pore water pressures.

35. Earthquakes can cause distortions in earth embankments (1,2). A succession of slides of limited displacement which could occur in an embankment of cohesive materials is indicated in Fig 2 as is also the type of sliding to be expected in dry cohesionless materials. The types of movement shown do not include the possibility of liquefaction of the embankment or foundation material which would cause flow of the embankment. Vertical cracking can be caused by differential shearing of the embankment.

36. When evaluating the design of embankments to resist earthquake, two possibilities should be considered (3,4): embankment sliding not associated with liquefaction and liquefaction of the embankment or foundation materials. Sliding can be analyzed with a reasonable degree of certainty. The assessment of liquefaction potential requires the evaluation of: degree of saturation, relative density, and degree of confinement, as well as a shear analysis. Loose, fine, saturated tailings

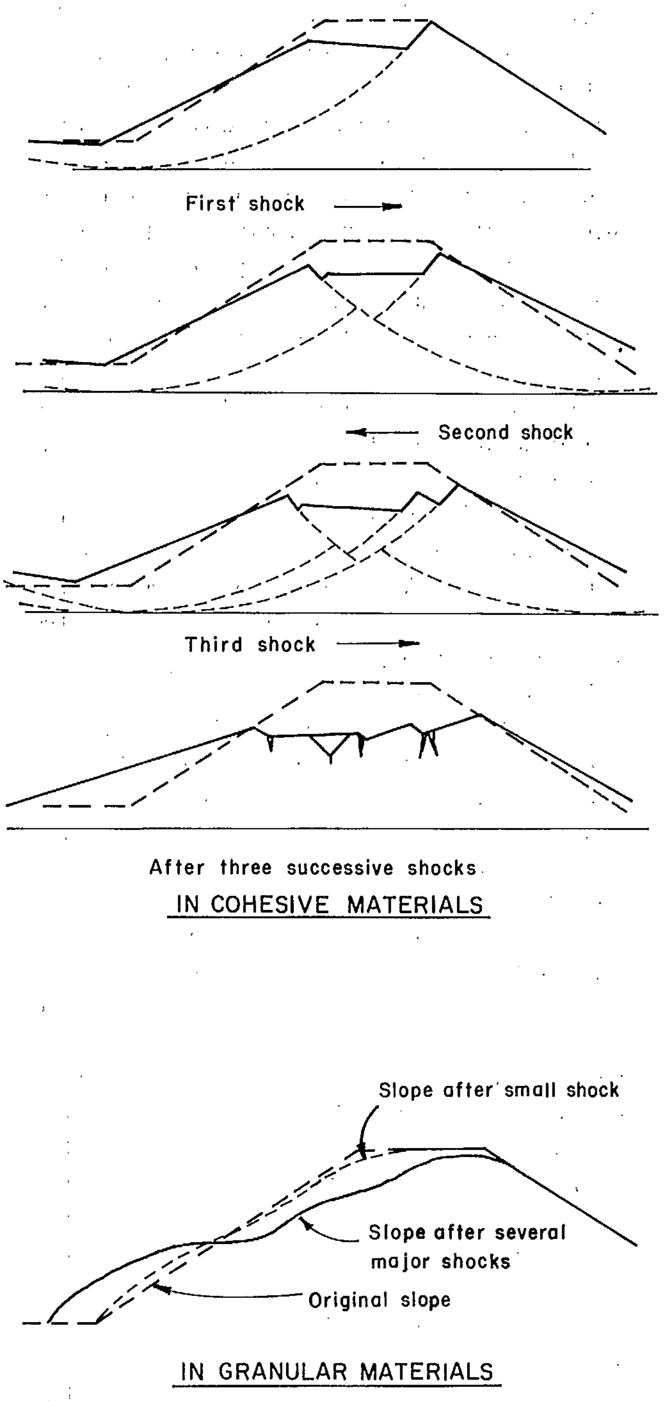


Fig 2 - Major embankment deformation due to earthquake

embankments are susceptible to liquefaction when subject to seismic activity. Unsaturated embankments with relative densities exceeding 60% offer reasonable resistance to liquefaction.

PROPERTIES

37. The shear strength of waste material is affected by the shape of its particles. Slabby pieces will tend to slide over each other, having unstable angles of repose and being susceptible to sliding during dumping. Tailings of peridotite, olivine, serpentine, sericite, mica and talc will tend to produce plate-shaped particles.

38. If very fine wet material is allowed to stand it can acquire cohesive strength. If the material is kneaded without altering the water content, its cohesion decreases considerably. This effect is known as "thixotropy". The softening and subsequent recovery of this material seems to be due to the destruction and subsequent rehabilitation of the molecular structure of the absorbed layers. Thixotropic effects occur with materials such as bentonite, clays, the insoluble portion of potash ore, shales, peridotite, olivine, serpentine, sericite, and talcy minerals. Siliceous materials are less thixotropic.

39. Soluble minerals such as halite and oxidizing sulphides will cement to other minerals. Such processes can result in rigid waste deposits.

40. Tailings from tar sands are oil-wet after processing. This oil coating on individual grains will affect the frictional component of the shear strength.

41. Sedimentation of tailings in ponds occurs in two distinct stages. Settling of the relatively coarser particles usually occurs within days; settling of the fine colloidal materials suspended in the "free" water may never occur naturally. Low specific gravity material - coal fines and tailings from platy minerals such as talc, chlorite and mica - will generally have slow sedimentation rates. Flocculating agents may be required for such minerals.

42. Oxidation of pyritic minerals within waste is determined by the rate of penetration of air which depends on the particle size distribution, the degree of compaction and the degree of

saturation. The products of pyrite oxidation are extremely acidic but may be neutralized by other alkaline minerals in the waste. Some effects of these neutralizing reactions are that: 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; shales in proximity to the pyrite may be converted into plastic clays by the action of sulphuric acid and 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.

43. Water draining from a coal waste pile may contain concentrations up to several thousand parts per million (ppm) 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 ppm may also be derived from the latter source. Much of the iron bearing water is relatively clear when it emerges from the pile but rapidly becomes ochreous on exposure to the atmosphere. In some coalfields, waste pile seepage waters may contain sodium chloride in concentrations of a few hundred ppm which have probably been derived from the waste materials, but the concentration may rise to thousands of ppm when tailings ponds are located on the waste pile. These waters will cause pollution if they are discharged directly into a stream or on adjacent land.

44. 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 of aluminum, may increase toxicity of the surface of the weathered pile and inhibit or prevent growth of vegetation.

45. Other minerals such as pyrrhotite will oxidize in waste piles, producing soluble salts and sulphuric acid. Minor amounts of arsenopyrite, tetrahedrite, and tennantite occur in some tailings which also will oxidize.

46. The processing of some minerals results in tailings effluents which are toxic to persons, animals or plants. The effluents from cyanidation residues, arsenopyrite residue, high sulphide tailings (which produce acidic effluents), potash brines (which kill plants), and uranium tailings may have to be collected and either treated or returned to the tailings pond.

COMMODITY CHARACTERISTICS

47. In coal strip mining, large volumes of waste shales, siltstones, and sandstones are produced of which the shales and siltstones tend to weather.

48. Cleaning of coal often results in waste consisting of coarse shale particles and fine coal particles with 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 or the mine output large, this procedure can require very large ponds because of the relatively slow sedimentation rate of the fine material and its low specific gravity. In other cases, the slurry containing the fine waste is filtered in the plant, and the resulting filter cake either hauled to separate waste piles or combined with the coarse wastes. The filtered material is relatively wet when it leaves the plant and can stick in haulage units.

49. Coarse and fine wastes originating from asbestos mills can be dumped separately but are usually conveyed in a dry state to a 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.

50. The excessive use of water to control dust can affect stability.

51. Potash mill tailings usually range in grain size from a maximum of about 5 mm (No. 4 mesh) down to salt in solution. The tailings 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, forming rigid deposits. However, the clay materials, which cannot normally be thickened by settling to more than 25 - 30% solids by weight, settle slowly and entrain air, often producing a frothy scum on the surface of the pond. Clear brine can be decanted from the pond if sufficient settling area is provided and baffles are used at the decant intake to hold back the scum.

52. Problems have been encountered in potash waste disposal by 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 settling of the relatively rigid deposits, or at points where unsaturated brines percolate through the deposits. Dilution from runoff water can cause unsaturated conditions.

53. Because of the danger of brine escaping from the main settling pond, it is desirable to use natural basins to retain potash wastes, or construct retaining embankments of borrow materials, or use separate ponds for final settling of the clay portion of the waste. With 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 by seepage from the primary pond. The supplementary ponds are retained by relatively impervious embankments.

54. The need to limit seepage of brines into the sub-soil requires that the soils underlying potash waste ponds be of low permeability or that impervious membranes be placed over permeable soils prior to the disposal of tailings.

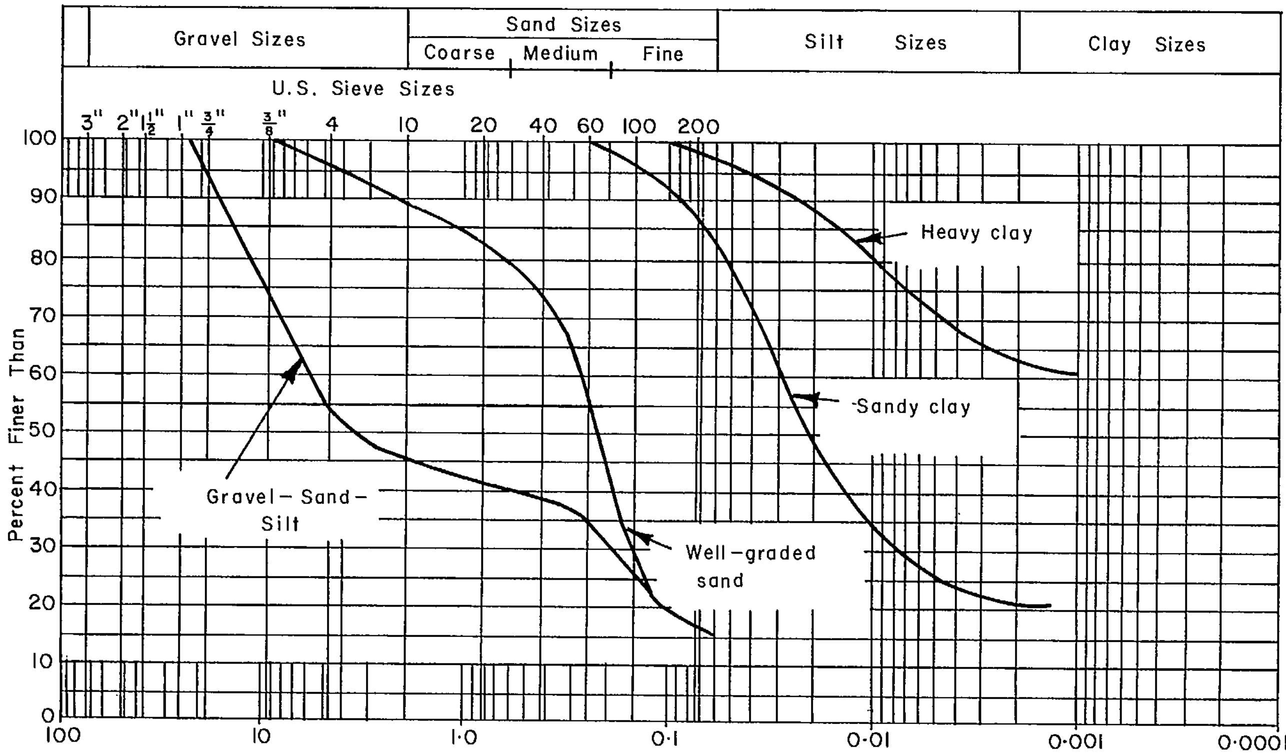
55. Tailings low in sulphides but high in quartz and silicates often result from the mining of porphyry-copper ores. Such tailings which are low in clay minerals have poor binding properties. They can usually be used in the construction of retaining embankments. When dry, they are susceptible to erosion by wind and water. These waste disposal areas may have to be stabilized by vegetation.

56. The mining of massive sulphide orebodies like copper, lead, iron and zinc sometimes results in tailings with a high sulphide content, particularly if iron sulphide is not removed. In concentrating these ores, the heavier sulphides - mainly pyrite and pyrrhotite - are more finely ground than the lighter siliceous fractions. After the tailings are deposited, these finely ground sulphides will partially oxidize and cement to form a surface crust in dormant areas of the pond. Such a crust will reduce surface erosion caused by runoff. However, if the crust forms near the downstream toe of the retaining embankment, it could restrict the exit of seepage water and reduce stability of the embankment. In this case an internal drainage system should be provided at the downstream toe.

57. If the sulphide content of the tailings is very high, it can spontaneously ignite and burn. Such burning can usually be controlled by smothering it with tailings slurry, but the permeability and stability of the deposit can be adversely affected.

58. In the extraction of gold, ores are often treated with sodium cyanide. Arsenopyrite usually requires roasting to remove the arsenic and thus allow leaching of the gold. Arsenic produced from gold-bearing arsenopyrite must be collected and stored in suitable areas. To overcome the danger of releasing toxic seepage from gold mine tailings basins, embankments should be designed to ensure minimum seepage.

59. Uranium mines using the acid leach process produce effluent wastes with very low pH and high metal content. Experience indicates that leaching will also take place within the tailings basin. Seepage from uranium tailings basins must be minimized, collected, monitored, and treated where necessary.



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

Fig 3 - Typical soil gradation curves (13)

MECHANICAL PROPERTIES

60. Grain size distribution has an important influence on natural slopes occurring in a tailings basin, affecting the height of embankment required to store a particular volume of solids. Generally, the coarser the tailings the steeper the slope, although the ratio of solids to water at the time of discharge and the velocity of discharge are also influential. Well graded material tends to have a higher density than gap graded mixtures.

61. Classification on the basis of grain size is: gravel - 2 mm to 60 mm, sand - 0.06 mm to 2 mm, silt - 0.002 mm to 0.06 mm and clay - less than 0.002 mm. Materials 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% size (60% passing) of the material to the 10% size of the material:

$$\text{ie} \quad U = \frac{D_{60}}{D_{10}} \qquad \text{eq 1}$$

Materials having U values of less than 4 are considered poorly graded. A well graded material, such as glacial till, has a U value of greater than 4.

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

63. Tailings have a wide variation of grain size distribution, depending on mill operation, but usually range from coarse sand to colloidal material (Fig 4). Generally, the particles are angular, especially when derived from hard rock sources.

64. The usual range of grain size distribution for coal waste is shown in Fig 5. Coal waste can be expected to become progressively finer after being placed in storage, because of the effects of weathering.

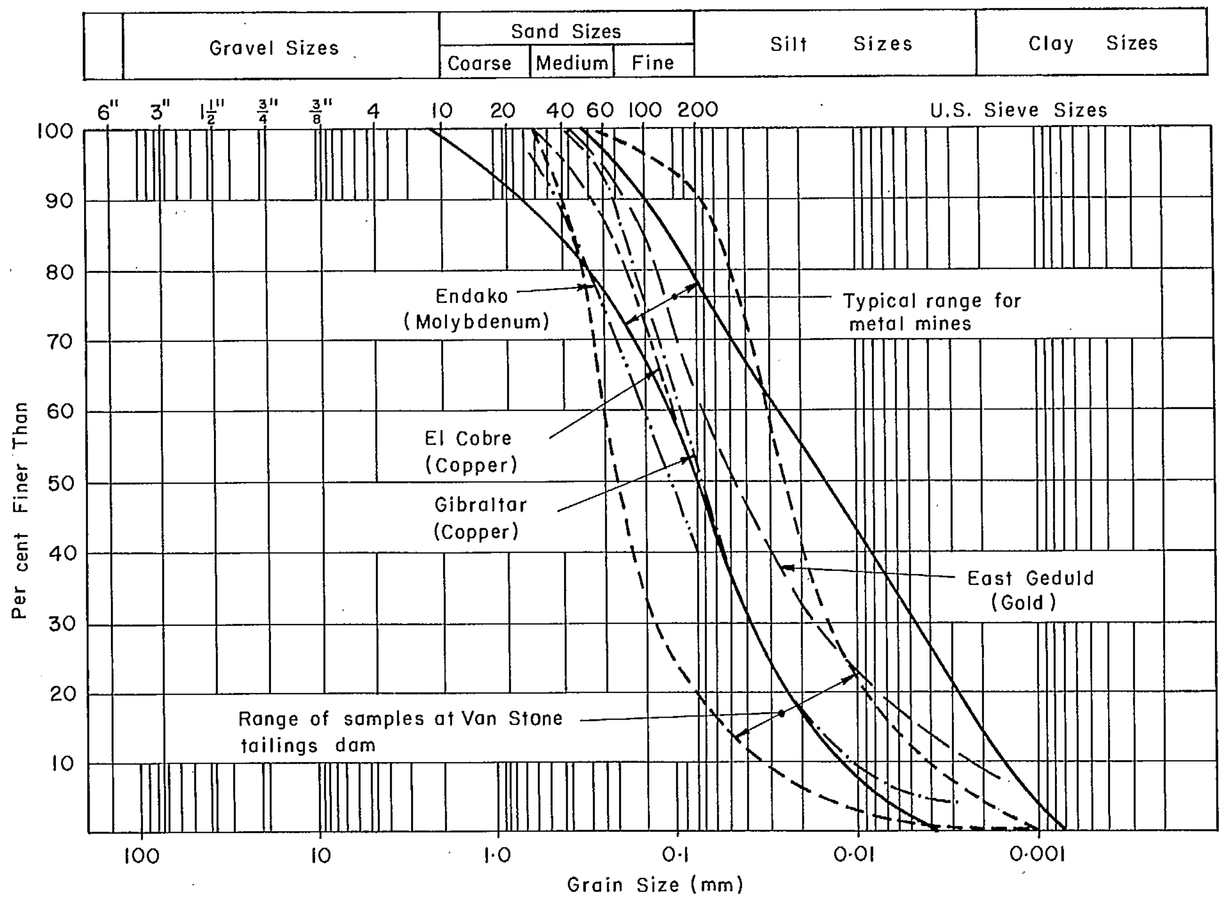


Fig 4 - Gradation of typical mine tailings

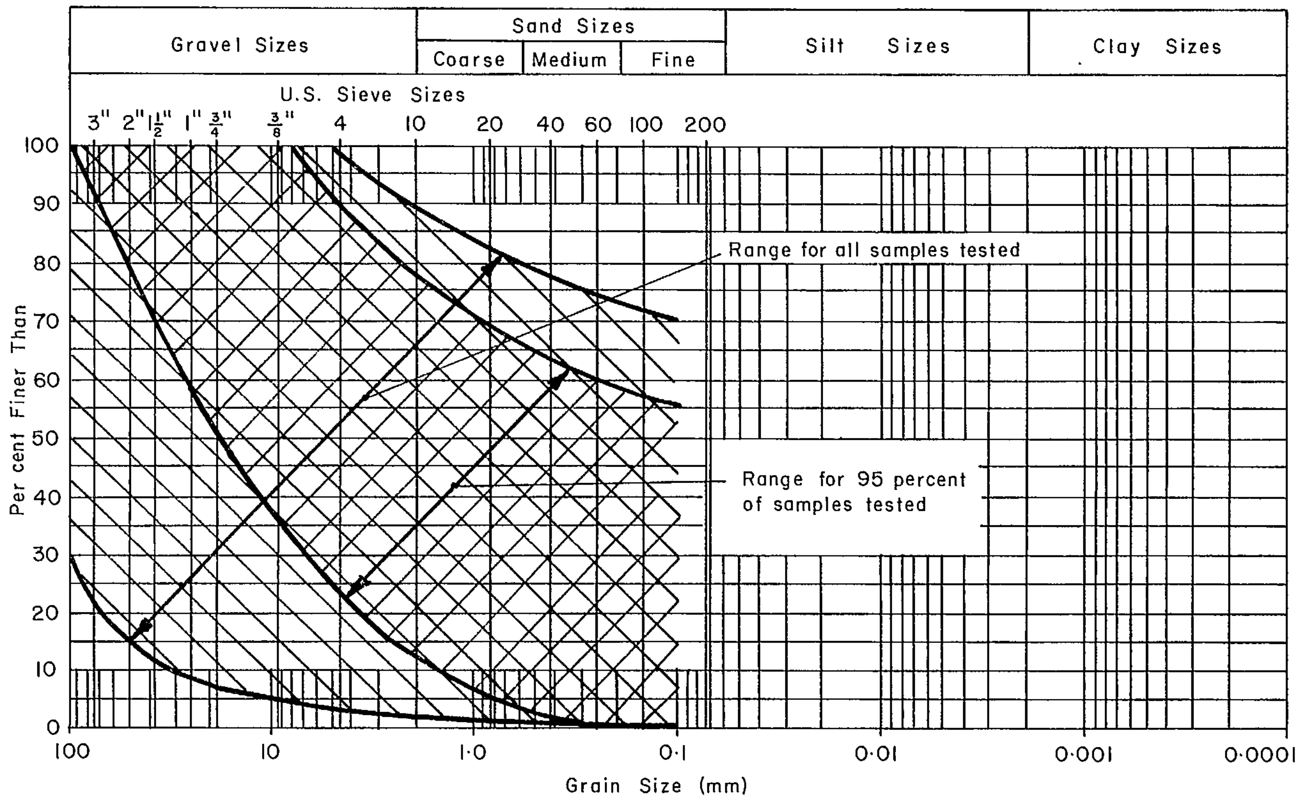


Fig 5 - Gradation of coal wastes (National Coal Board)

65. The plasticity characteristics of material are described by the liquid limit, plastic limit and shrinkage limit or Atterberg limits (5,6). In general, the plasticity of a soil is significant where the proportion of fines (-200 mesh) is high. Soils may be classified according to their plasticity, using the difference between liquid limit and plastic limit, or the plasticity index (Fig 6). Plasticity characteristics of various soils are tabulated in Table 1. Plasticity data for tailings from a mining area in central Chile are tabulated in Table 2 and the range of values for coal waste is shown in Fig 7.

Table 1: Typical plasticity characteristics

Soil	Liquid limit	Plastic limit	Plastic index	Specific gravity
Lean clay	32	18	14	2.75
Fat clay	80	28	52	2.75
Sandy clay	40	20	20	2.75
Sand	Non-plastic			2.65
Gravel-sand - silt	Non-plastic			2.68
Rock fill	Non-plastic			2.70

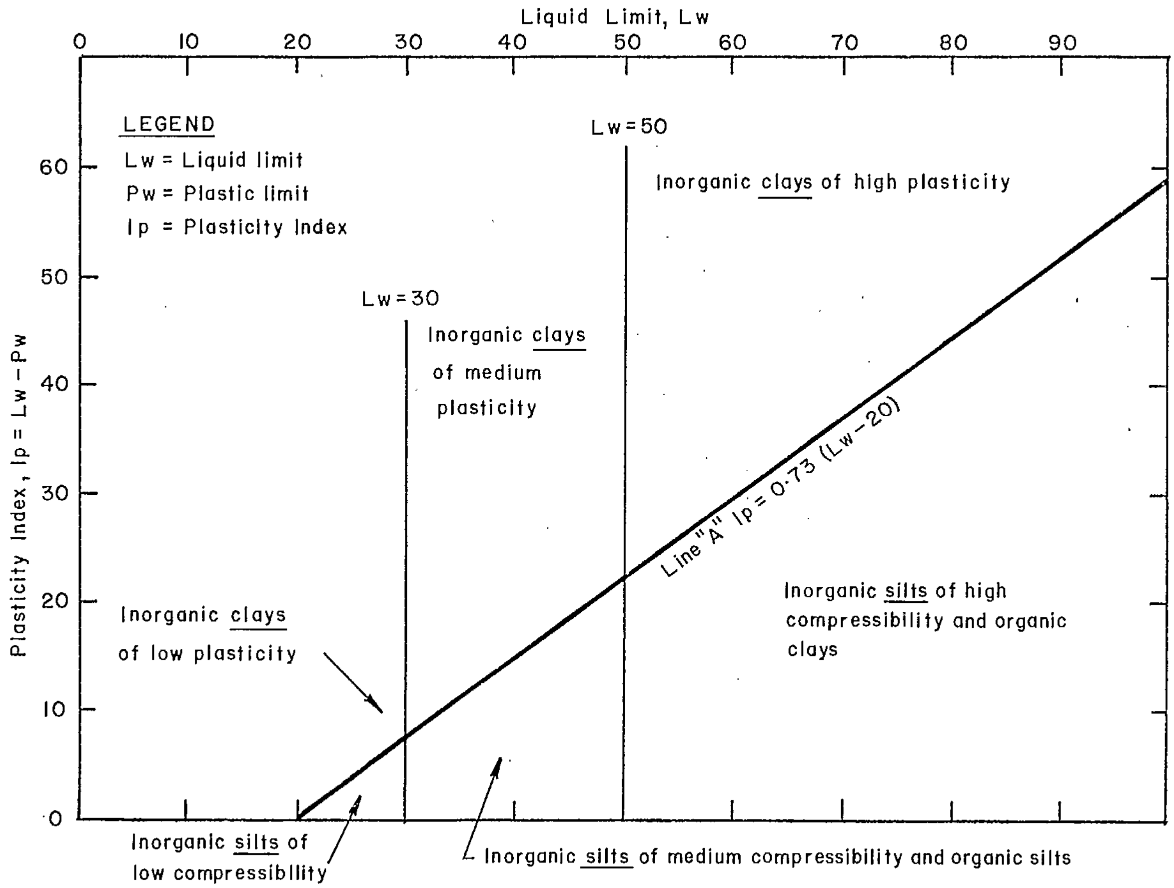


Fig 6 - Plasticity chart (5)

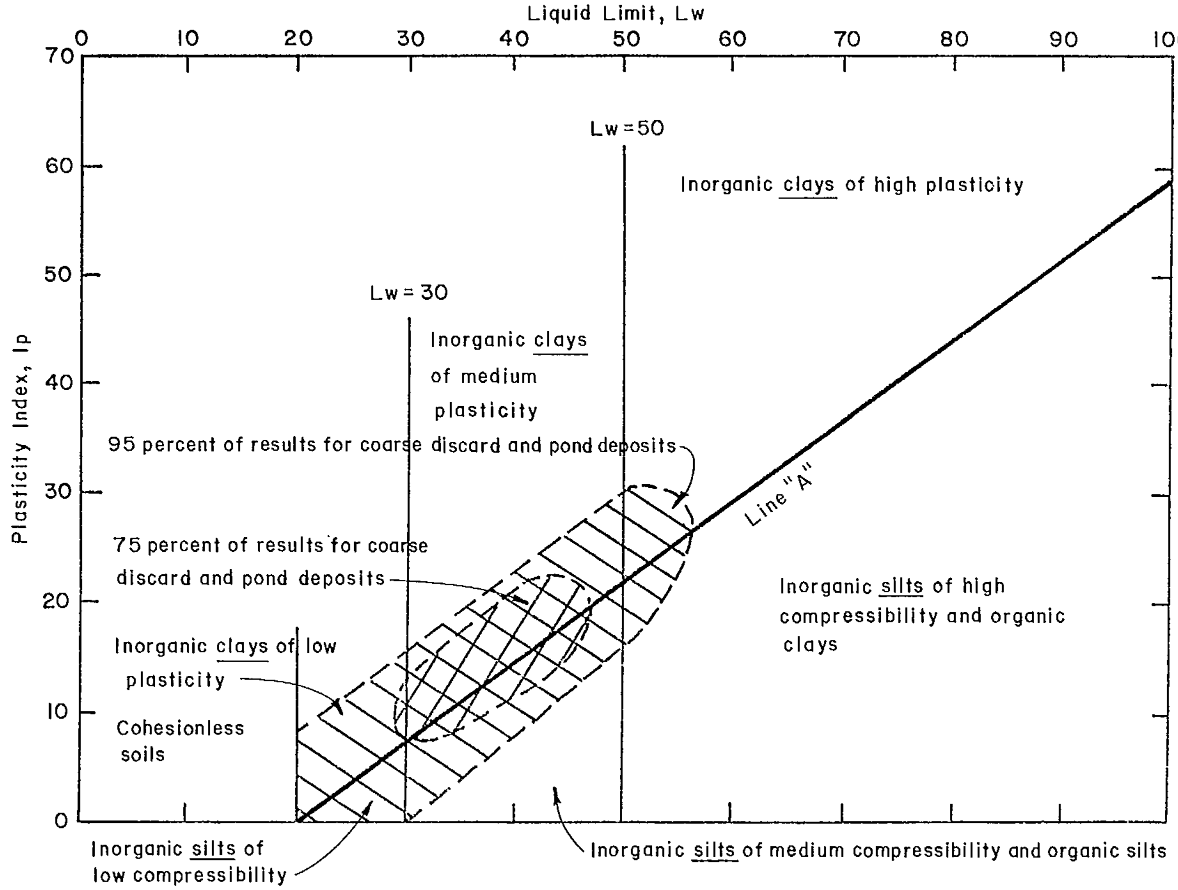


Fig 7 - Plasticity of coarse discards and pond deposits in coal waste embankments (National Coal Board)

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

67. The water contents of soils commonly range from 5% to 35%. However, for soil types that contain a high percentage of clay or organic material, water content in excess of 100% may occur. In general, the water content depends on void ratio, particle size, clay mineral, organic content, and groundwater conditions. Water content data for tailings from some mines in central Chile are shown in Table 2.

Table 2: Plasticity characteristics of some Chilean tailings

Locality	El Cobre	Hierro Viejo	Los Maquis	La Pataqua	Cerro Negro	Bellavista	El Sauce	Ramayana
Permeability of foundation soil	low	low	medium	medium to high	high	medium	low	low
Percentage minus 200 mesh	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, per cent	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

68. The unit weight or density of a material is defined as the weight of material divided by its total volume, including solids, liquids and voids. The in-place density of soils is 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.

69. In general, when measuring the degree of compaction of a soil, the dry density - weight of solids divided by total volume - is used. For a given compactive effort, the maximum dry density is obtained at a particular "optimum water content" (Fig 8).

70. Typical ranges of density of various soil types are shown in Table 3. The dry density of tailings varies from 70 lb/cu ft (1120 kg/m³), to 120 lb/cu ft (1920 kg/m³). The dry density of coal tailings obtained from different deposits ranges from 80 lb/cu ft (1280 kg/m³) to 130 lb/cu ft (2080 kg/m³).

71. The relative density of a soil is the density relative to its two limiting values. A soil in its loosest state has a relative density of zero, and a soil in its densest state has a relative density of 100%. Because the density of a soil is directly related to the void ratio, the relative density, D_r, can be expressed in terms of the void ratios:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} 100\% \qquad \text{eq 2}$$

where e_{max} is the void ratio of the loosest state, e_{min} is the void ratio of the densest state and e is the void ratio of the soil as it exists. The relative density offers a convenient measure of the degree of compactness of a soil in a fill or embankment and also provides a significant indication of the susceptibility to liquefaction. A soil with a relative density greater than 60% is not susceptible to liquefaction.

72. The specific gravity of a rock or soil may be defined as the ratio between the unit weight of the substance to the unit weight of pure water at 4°C. The average specific gravity of granite is

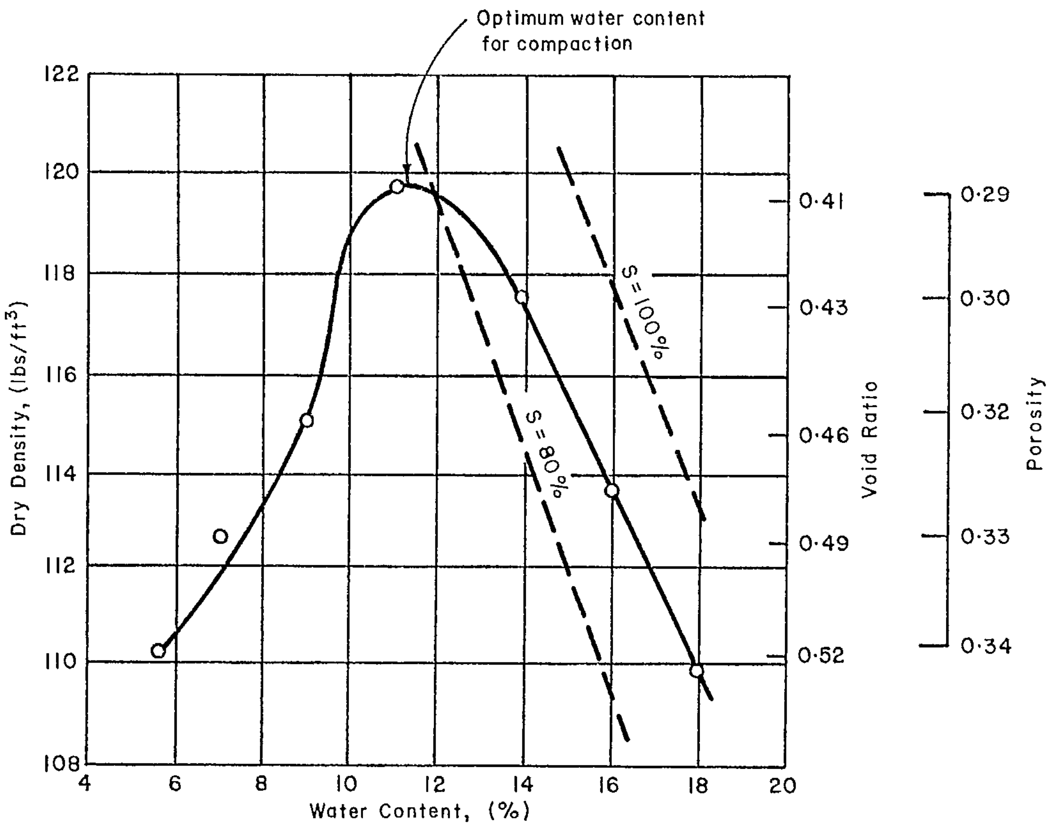


Fig 8 - Typical curve of dry density vs compaction water content

about 2.65, with a range from 2.4 to 3.6 depending on the mix of its mineral constituents. The specific gravity of tailings particles may range from about 2.5 to 3.5, depending on mineralogical composition.

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

74. Permeability may be defined as the facility with which water is able to travel through pores of the soil. 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 length of seepage path). Knowing this coefficient, the hydraulic gradient, and the total area through which seepage flow passes, the rate of seepage

Table 3: Typical densities of natural soils

Description	Porosity n (%)	Void ratio e	Water content w (%)	Unit weight γ_d lb/ft³	γ_d kg/m³	γ lb/ft³	γ kg/m³
Uniform sand, loose	46	0.85	32	90	1441	118	1890
Uniform sand, dense	34	0.51	19	109	1746	130	2082
Mixed-grained sand, loose	40	0.67	25	99	1585	124	1986
Mixed-grained sand, dense	30	0.43	16	116	1858	135	2162
Glacial till, very mixed-grained	20	0.25	9	132	2114	145	2322
Soft glacial clay	55	1.2	45	76	1217	110	1762
Stiff glacial clay	37	0.6	22	105	1681	129	2066
Soft slight organic clay	66	1.9	70	58	929	98	1570
Soft very organic clay	75	3.0	110	42	672	89	1425
Soft bentonite	84	5.2	194	27	432	80	1281
Rock fill	41	0.7	25	100	1601	125	2002

w = water content when saturated, in per cent dry weight

γ_d = unit weight in dry state

γ = unit weight in saturated state

through a homogeneous soil can be calculated.

75. Seepage through a natural soil or rock mass depends 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, and gravel. The voids in a homogeneous soil, without fissures, can be measured in fractions of a millimetre. The dimensions of open fissures which exist in natural soil or rock masses and in embankments can often amount to several centimetres. 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, which is the case with tailings embankments, the existence of such fissures should be considered. They can 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 segregated seams of stony material between compacted layers, and at contact points between conduits and walls incorporated in the fill.

76. The coefficients of permeability for various soils and tailings are indicated in Table 4. Typical relationships between permeability and density for various types of soils and tailings are illustrated in Fig 9. The influence on permeability of soil gradation and the nature and amount of the fines contained in the soil are illustrated in Fig 10. Soils may be classified according to their coefficients of permeability, as shown in Table 5.

77. The permeability of coal waste depends primarily on its gradation and degree of compaction. Typical values of the coefficient of

Table 4: Permeability coefficients for soils

Coefficient of permeability, k, in cm/sec (log scale)

10^{2} 10^{1} 1 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, eg 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

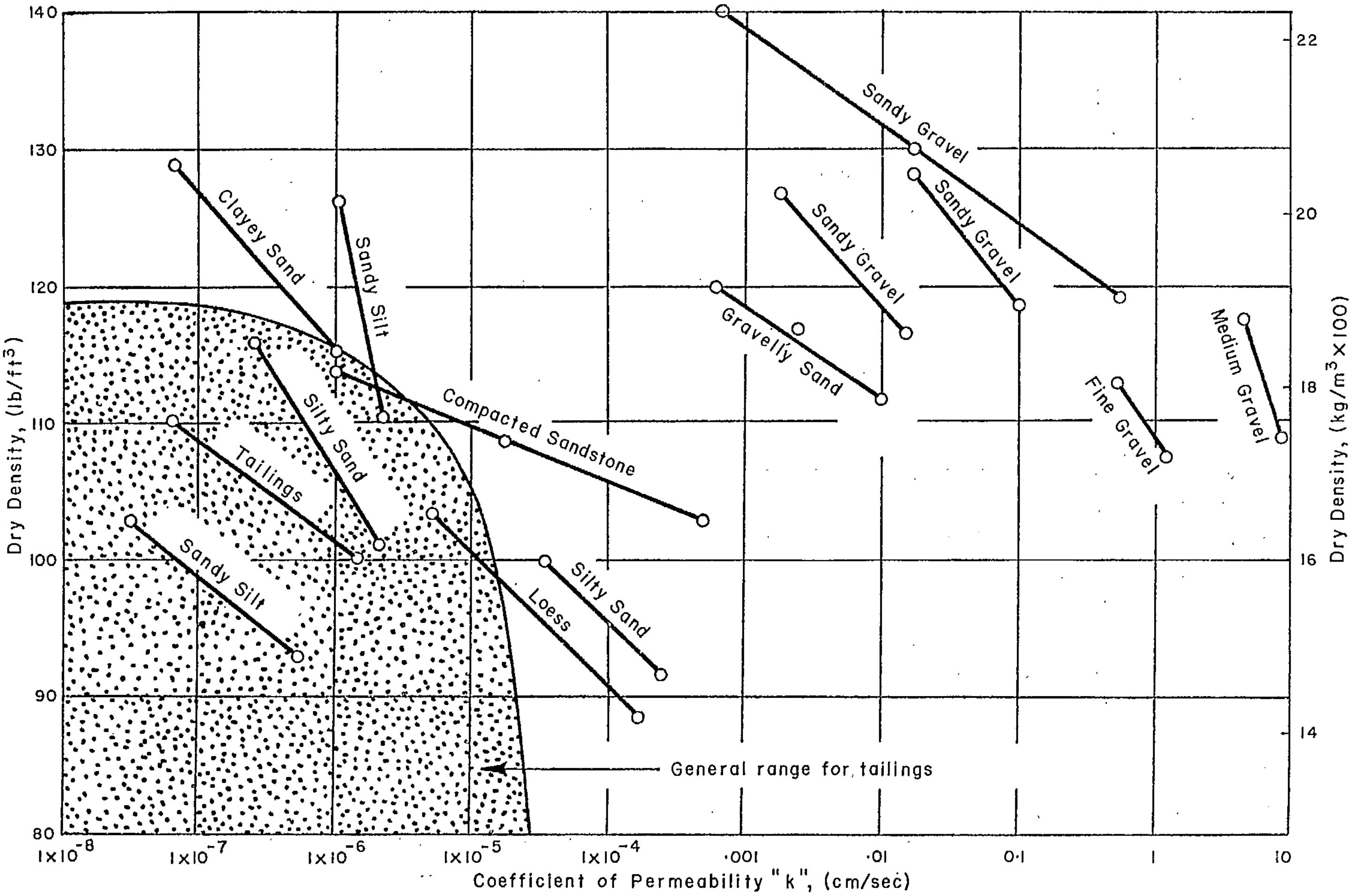
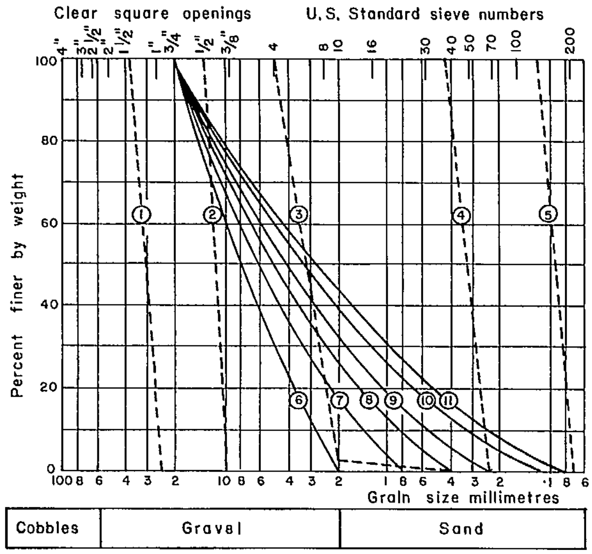


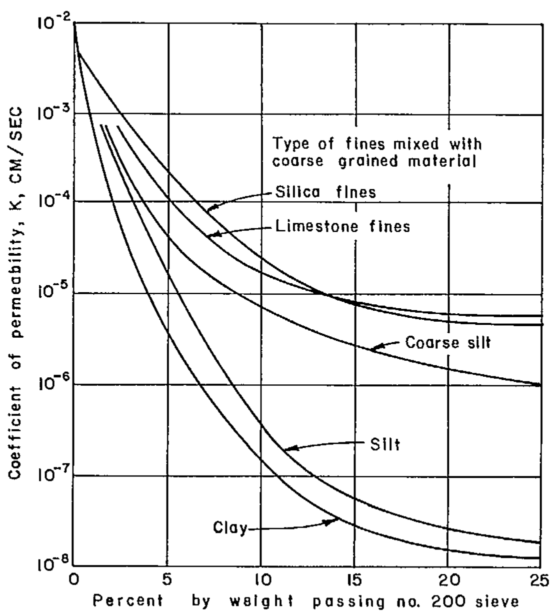
Fig 9 - Permeability vs density (21)



PERMEABILITY VS. GRADATION

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	$3{\cdot}5 \times 10^{-1}$
9	$1{\cdot}1 \times 10^{-1}$
10	$4{\cdot}0 \times 10^{-2}$
11	$5{\cdot}0 \times 10^{-3}$



PERMEABILITY VS. FINES CONTENT

Fig 10 - Coefficient of permeability vs gradation and fines content (USN - Bureau of Yards and Docks)

Table 5: Permeability classification of soils

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}

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 silty materials. As a result, the horizontal permeability tends to be much higher than the vertical permeability. The coefficient of permeability of slurries varies from 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.

Table 6: Typical values of effective cohesion and angle of internal friction for soils

Soil	Effective cohesion c' psf	Effective cohesion c' kPa	Effective angle of internal friction ϕ' degrees
Bentonite shale	300	14.3	7
Muddy sand	400	19.1	30
Shale (fill cemented)	1000	47.9	34
Sandstone (fill)	-	-	35-45
Soft clay	400	19.1	Variable depending
Very soft clay	200-370	9.5-17.7	on rate of load
Stiff clay	1500-2000	71.8-95.7	application
Silt (non-plastic) - medium dense			28-32
Silt (non-plastic) - dense			30-34
Uniform fine to medium sand - medium dense			30-34
Uniform sand - dense			30-40
Well-graded sand - medium dense			38-46*
Well-graded sand - dense			36-42*
Sand and gravel - medium dense			40-48*
Sand and gravel - dense			40-55*
Tailings sand - loose			30-36

* Higher values occur at low confining pressures and such high angles require confirmation by thorough and extensive testing.

78. The shear strength is defined as the maximum resistance of a soil to shearing stresses. The shear strength of a soil is made up of two components, cohesion and internal friction:

$$s = c' + \overline{p}\tan\phi' \qquad \text{eq 3}$$

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

Typical values of c' and ϕ' for various soil types are tabulated in Table 6.

79. 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 density of the soil and angularity of the soil particles. The shear strength of tailings is 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.

80. 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 from 0 to 1000 lb/sq ft (1488 Pa), and ϕ' varies from 22 to 34 degrees.

81. 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 permeability of the soil and 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 (7). Figure 11 shows typical curves of this relationship.

82. The consolidation characteristics of most tailings are similar to those of a normally consolidated soil. A soil element that is at equilibrium under the maximum stress it has ever experienced is normally consolidated. A consolidation curve for the tailings at the East Geduld mine is shown in Fig 12. A gradation curve for these tailings is shown in Fig 4. Consolidation curves for a very loose and low density tailings material in an undisturbed and remoulded condition are shown in Fig 13.

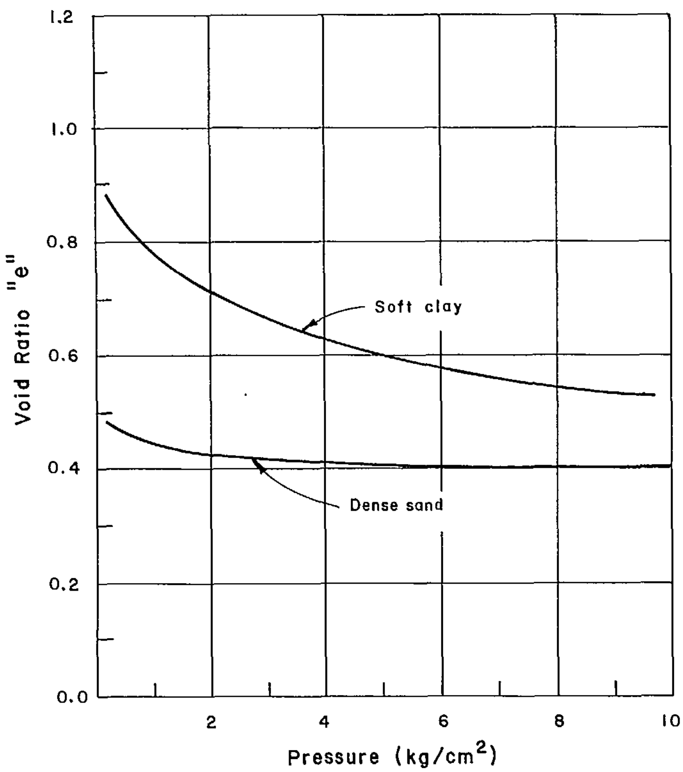


Fig 11 - Typical pressure-void ratio curves for sand and clay

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

84. The properties of rockfill depend primarily on particle shape, size distribution, mineralogy of particles, relative density and confining pressure (8). Typical values of specific gravity and density are tabulated in Tables 1 and 3. The average coefficient of permeability of a

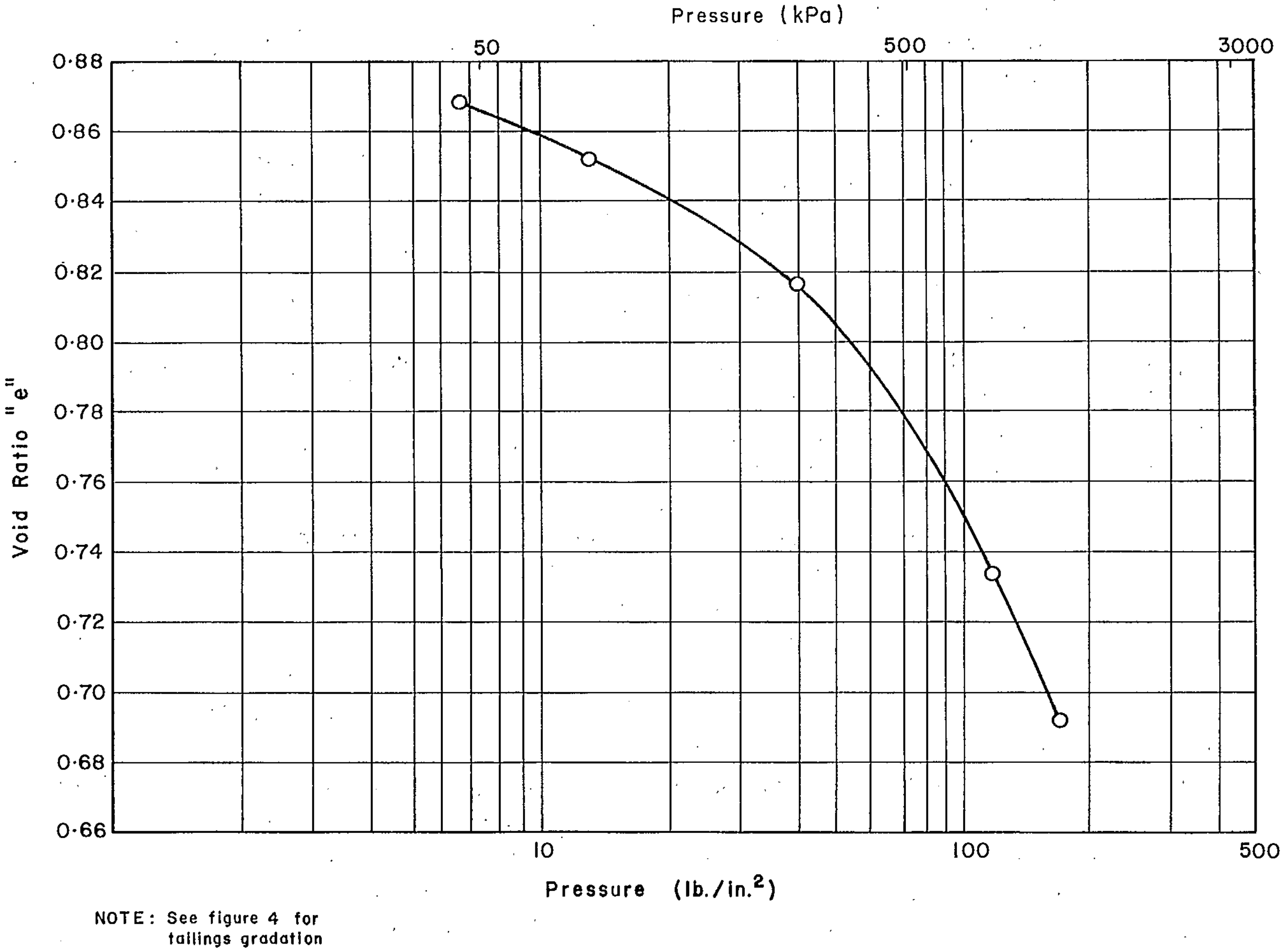


Fig 12 - Consolidation curve of East Geduld tailings (Donaldson)

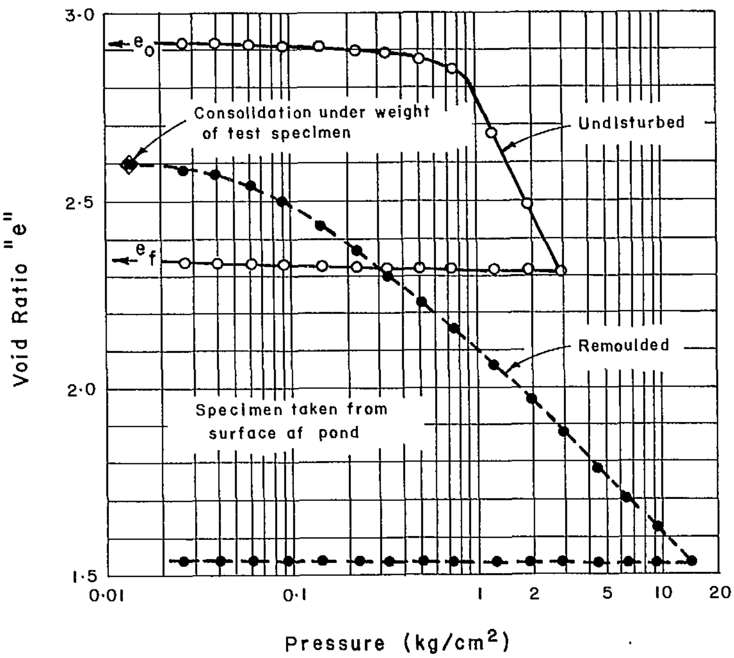


Fig 13 - Consolidation curves of undisturbed and remoulded tailings (35)

well graded rockfill normally will range between 10^{-1} 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 or smaller than the average values for the fill.

85. Some relationships determined by laboratory tests on crushed rocks between the angle of internal friction, particle size, relative density, and confining pressure are shown in Fig 14, 15 and 16. Typical values of the angle of internal friction of rockfill materials lie above 35 degrees.

86. The behaviour of sedimentary rocks is affected by inherent stratification and jointing. Typical values of some important mechanical properties of sedimentary rocks are tabulated in Table 7. These properties, in situ, are greatly influenced by such structural discontinuities as joints, faults and folds. For tailings embankments, special attention should be given to limestone foundations because of the effect of weathering on strength and permeability. Sandstones can be relatively pervious. The variable nature of the more porous sedimentary rocks should be considered if used as fill material.

87. Granite, diorite and basalt are igneous rocks frequently encountered in foundations; some of their important properties are shown in Table 7. They are generally adequate but basalt formations in particular can be very pervious because of heavy jointing.

CLIMATE AND HYDROLOGY INVESTIGATIONS

88. Records of air temperatures, precipitation, wind, solar radiation and evaporation measured at various stations in Canada are published by the Atmospheric Environment Service of the Department of the Environment, Ottawa (9). Various climatic maps and atlases showing average distribution of extremes of temperature, humidity, wind, rainfall, snowfall, etc are published also. These data are essential in estimating runoff into mine tailings ponds. A list of the most useful

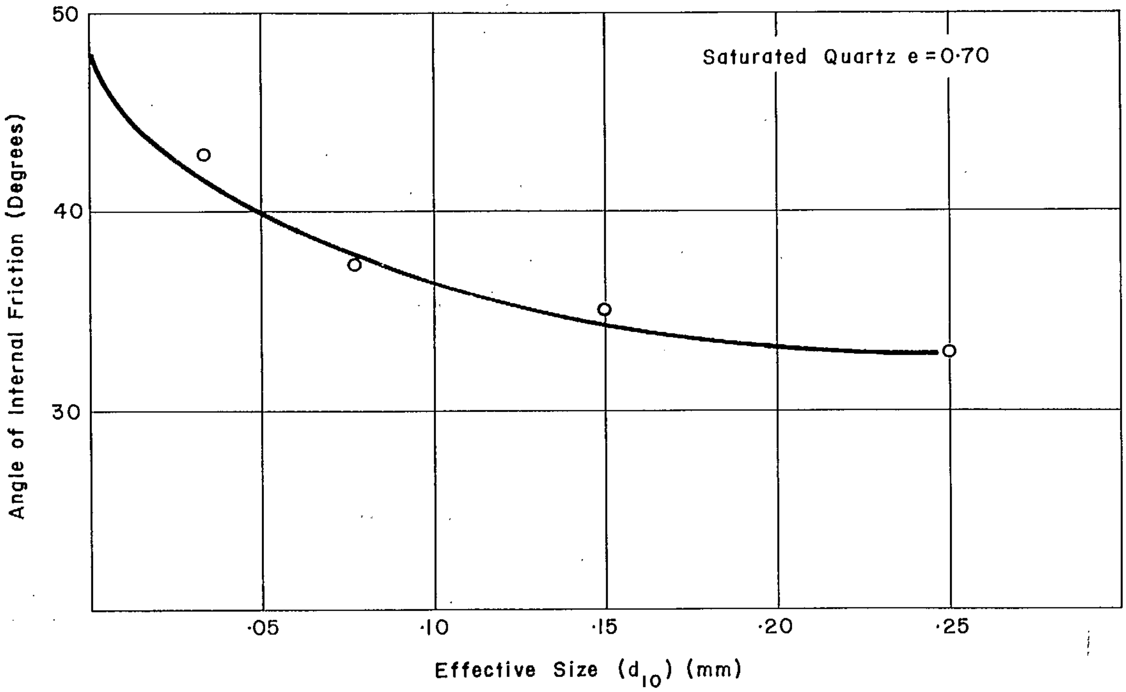


Fig 14 - Angle of internal friction vs confining pressure for crushed basalt (Vesic)

publications is included in the references (10).

89. Records of measured stream flows are published by the Water Survey of Canada, Department of Environment.

90. All natural streams and the maximum possible runoff should be diverted around the tailings basin for two principal reasons: a. to prevent contamination of the natural waters, and b. to minimize the volume of runoff into the tailings basin.

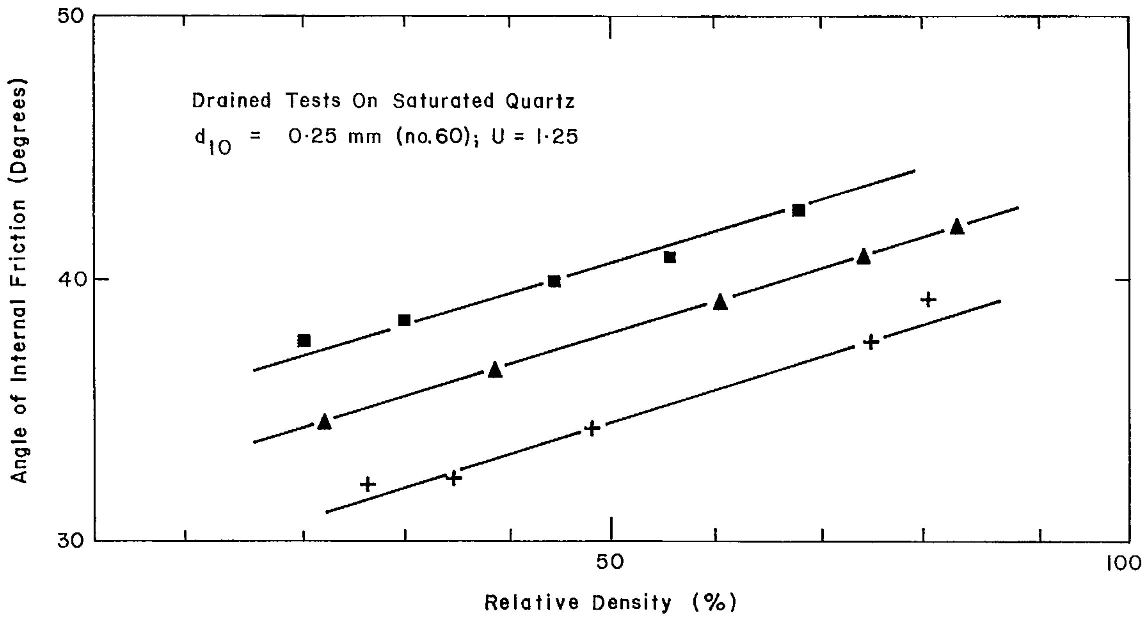
91. The settling pool in a tailings basin is designed for an approximately constant depth of water to provide the desired degree of clarification. The overflow system is designed to maintain this required depth and to handle the maximum volume of effluent flow plus the peak flood flow over 24 hours for a flood with a recurrence interval of 100 years.

92. The tailings basin catchment areas are usually quite small and precipitation and streamflow data can be used to predict peak runoff. A sample calculation is shown in Appendix A. Where rainfall data is not available for a particular catchment area, data can be obtained from adjacent areas or generalized climatic maps can be used.

93. Generally, evaporation is not a critical design parameter for tailings basins in Canada because evaporation is relatively negligible.

94. Topographic maps useful for locating mine waste disposal areas 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 ft and 500 ft (30 m and 150 m) respectively.

95. Aerial photographs of most parts of Canada are also available from the following services: (a) National Airphoto Library, Surveys & Mapping 8ranch, Department of Energy, Mines & Resources Ottawa and Calgary; (b) provincial coverage,



Symbol	Particle Sphericity	Particle Shape	Min & Max Dry Density lb cu ft		Min & Max Dry Density kg/m^3	
■	0·45	Angular	73·6	98·2	1180	1570
▲	0·58	Sub-Angular	86·2	103·8	1380	1660
+	0·67	Sub-Rounded	92·1	108·4	1475	1735

Fig 15 - Effect of relative density on angle of internal friction of quartz materials (Lee)

Fig 16 - Angle of internal friction vs confining pressure for crushed basalt

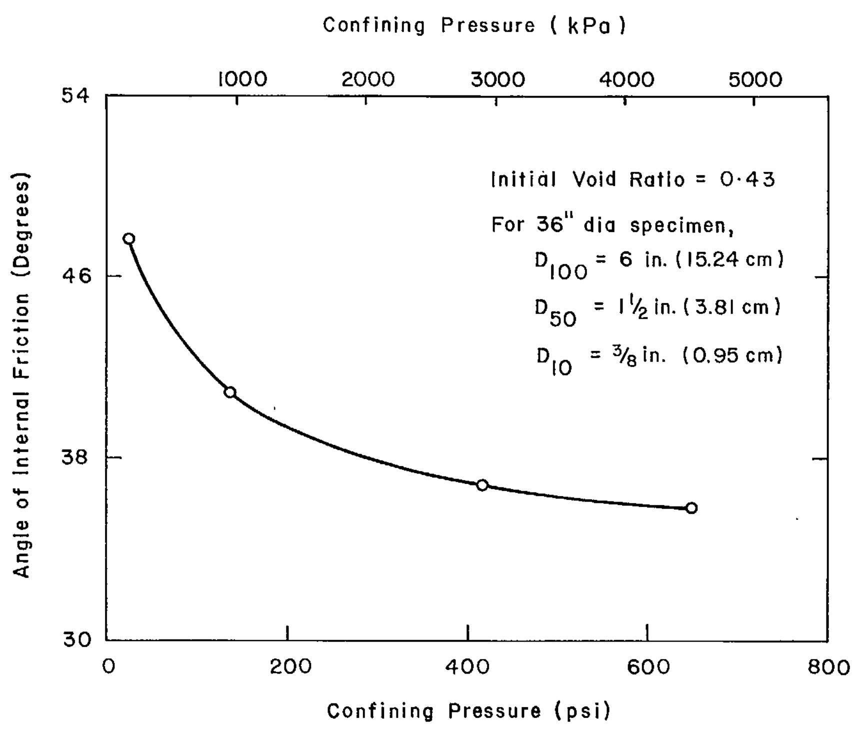


Table 7: Typical mechanical properties of rocks

Rock type	Poissons ratio	Modulus of deformation psi(10^6)	Modulus of deformation kPa(10^6)	Compressive strength psi(10^3)	Compressive strength kPa(10^3)
Diorite	0.26-0.29	10.9-15.6	75.1-107.5	10.0-15.0	68.9-103.4
Feldspathic gneiss	0.15-0.20	12.0-17.2	82.7-118.6	6.0-12.0	41.3- 82.7
Granite	0.23-0.27	10.6-12.5	73.0- 86.1	8.5-25.0	58.6-172.3
Limestone	0.27-0.30	12.6-15.6	86.8-107.5	4.5-15.0	31.0-103.4
Quartzite	0.12-0.15	11.9-14.0	82.0-96.5	15.0-20.0	103.4-137.9
Slate	0.15-0.20	11.5-16.3	79.3-112.4	3.0- 8.0	20.7-55.1
Sandstone	0.15-0.35	4.0-10.0	27.7- 68.9	8.0-20.0	55.1-137.9

available through Airphoto Libraries, Departments of Lands in British Columbia and Alberta, and departmental libraries in various branches of other provincial governments; and (c) private sources such as air-survey companies working in an area often have coverage not available through other sources and will make photographs available with permission of the prior client.

96. Maps having a scale of 1:5,000 and a contour interval of 10 ft (3 m) are generally adequate for preliminary studies. For detailed design, larger scale maps and smaller contour intervals are often required. Specific surveys will be required at the site for the dam.

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

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

99. The map area should cover the entire tailings disposal system. The complete catchment area of the tailings disposal system should be mapped to enable reliable runoff estimates and accurate pond storage-elevation curves to be made.

GEOTECHNICAL INVESTIGATIONS

100. Geotechnical investigations at mine waste embankment sites should determine nature of the soil deposits - geology, recent history of deposition, erosion, and consolidation; depth, thickness, and composition of each significant soil stratum; location of groundwater, pore water pressure, or artesian conditions; depth to bedrock and its general composition; presence of old underground mine workings; geotechnical properties of the soil and rock strata that affect the design of the structure; and availability of suitable construction materials.

101. Geological investigations can include: ob-

taining regional geological reports, maps, and aerial photographs, field reconnaissance and mapping, geophysical surveys, exploratory drilling, borehole photography, measurement of groundwater conditions, groundwater pumping tests, and laboratory testing of samples of rocks and soils including mineralogical analyses. Some details to note are: evidence of buried channels; evidence of instability - collapse, solution cavitation; evidence of recent tectonic disturbance; evidence of weak formations - bentonite layers, mylonite, shear planes; and seismic history.

102. The extent of investigation will vary depending on embankment height and 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, determine whether the foundation is strong enough to withstand the shear stresses, and evaluate methods of controlling seepage.

103. For mine waste piles, detailed investigations such as core drilling of bedrock should only be required in unusual circumstances. Such investigations are required for water storage dams because of the danger of seepage and uplift pressure in the foundation. They are relatively unimportant for mine waste piles but are important for tailings embankments.

104. The first step in foundation investigation should be an appraisal of any previous underground mining and the general geological character of the site. The second step should be to put down a few exploratory drillholes, spotting these after making a detailed examination of soil and rock exposures at the site. The preliminary investigation may indicate that geophysical surveys and echo soundings in lakes would be useful at an early stage. Test pits and trenches will be useful in determining the nature of surface soils.

105. Subsequent steps will depend on the size of the embankment and the character of the soil profile. The importance of the structure and results of the exploratory drillholes will indicate the extent of the detailed drilling program. At sites where the subsoil profile is erratic, it will be necessary to define the pattern of dissimilar soils and characteristics of the various strata. Probing with a cone penetrometer can provide rapid identification of the density of subsoils.

106. The depth of foundation boring will depend on: size of the loaded area, magnitude of loading and the subsoil profile. As a general rule, the borings should be deep enough to determine the subsoil 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. In the case of a tailings embankment, at least one borehole in the foundation soil should extend to a depth equal to 1.5 times the ultimate embankment height.

107. Adverse conditions that require particular attention during sub-surface investigations are: organic stratification, highly plastic clays, expansive clay-shales, stiff fissured clays, large boulders overlying bedrock which may be difficult to differentiate from drill cores, karstic limestone formations, and buried, coarse, talus deposits.

108. A common error is in defining 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 and the approximate depth to rock are known, it may be adequate to limit coring to 5 ft (1.5 m) into rock; however, where bedrock is irregular and where large boulders may be overlying the bedrock, coring should be 15 - 20 ft (4.5 - 6.0 m) into rock.

109. Other foundation problems may be due to: 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.

110. The types of equipment commonly used for foundation investigations are: diamond drill rig with casing and soil sampling equipment; wash boring; churn drill, the common well drill, versatile, with casing; rotary drill, several

types including diamond drills; hammer drill, several types with casing; and power auger, several types with continuous flights and hollow stem.

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

112. Digging test pits with mobile equipment is an expedient method of investigating borrow materials. To provide a competent seal and internal drainage system for a tailings dam, it is necessary to locate a source of both impervious and pervious material. Sampling and testing should be sufficiently extensive to confirm an adequate quantity of each. Normally, testing would include determining in situ moisture content, gradation, optimum moisture content, and optimum density for compaction of the borrow materials, shear strength, and permeability of the materials.

113. Samples are classified according to sampling procedures. A common grouping is: wash samples, disturbed samples, and undisturbed samples.

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

115. Representative disturbed samples are usually obtained from split-spoon samples. They are suitable for general classification tests and identification of the soil but are not suitable for strength tests. They are commonly used to indicate water content, grain size, specific gravity, and Atterberg limits.

116. Undisturbed samples require special sampling equipment and drilling techniques. They should be sealed with wax at the time the sample is taken to prevent loss of water during handling and storage and must be protected from freezing. They must also be protected from disturbance during handling and are usually shipped in tubes held upright in special shock-proof containers. Undisturbed samples are suitable for all laboratory tests.

117. The technique to be used depends on the nature of the ground and the degree of sample disturbance that is acceptable. Table 8 gives techniques for various conditions. The frequency of sampling depends on the variation in the materials being penetrated, but sampling at 5-ft (1.5-m) intervals is usually satisfactory. On occasion, sampling at more frequent intervals or even continuously is necessary to examine weaker zones or zones of relatively low permeability.

118. In cohesive soils, uncased dry boreholes may sometimes be used to shallow depths but stabilization with casing or drilling mud is usually required when samples are to be obtained from deep boreholes. Clay or silt samples are usually obtained with thin-walled Shelby tubes or piston samplers. Care and experience are necessary to obtain good samples in this material.

119. Sampling in sand and gravel usually requires casing or drilling mud to keep the boreholes open. 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.

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

121. Air-driven hammer drills are frequently used in exploring gravels and sands. Samples obtained from the discharge of such machines are valuable for indicating the nature of material at the bottom of the hole but cannot be considered truly representative because of degradation and segregation of particles being transported up the casing.

LABORATORY TESTING

122. The laboratory procedure for determining the grain size distribution of soils is described in ASTM D422 (11). In this procedure, a sample of soil is divided into a coarse 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 material finer than 200 mesh is determined by hydrometer.

123. 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 assessing some of the soil's other characteristics.

124. The specific gravity of soil particles is determined by laboratory test ASTM D854 (12). Knowing the dry density and the specific gravity of the soil particles, the void ratio can be determined from the following relationship:

$$e = \gamma_w G_s/\gamma_d - 1 \qquad \text{eq 4}$$

where:

γ_w = unit weight of water,
G_s = specific gravity of the soil particles,
γ_d = dry density, and
e = void ratio (the ratio of the volume of voids to the volume of solids in a soil).

125. The relative density of cohesionless soils is obtained by determining the void ratio of the soil in its loosest state, in its densest state and in situ. Relative density is given by eq 2:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \; 100\% \qquad \text{eq 2}$$

where:

D_r = relative density,
e_{max} = void ratio in the loosest state,
e = void ratio in situ, and
e_{min} = void ratio in the densest state.

126. The Atterberg limits are used to determine the plasticity of soils (5, 6, 13). 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 over which the soil remains plastic and is called the plasticity index. This parameter is used to classify soils for estimating other physical properties.

127. The compaction characteristics of a soil is obtained by the standard procedure ASTM D1557 (14). 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 desired 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 specified degrees of compaction. Maximum density is obtained by compacting a soil at optimum water content.

128. The shear strength of a soil is measured by triaxial compression tests or by 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.

129. Soil shear strength parameters are affected by many test factors including rate of loading, method of loading, principal stress ratios, rate of specimen strain, total specimen strain, degree of saturation and drainage conditions. Moreover, in selecting parameters 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.

130. 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 presented conventionally in the form of Mohr circles, as shown in Fig 17. The effective

Table 8: 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 (1.5-m) maximum spacing)	(Sampling in borings continuous samples diameter 2 to 3 in. (5-7.6 cm))	(Sampling in borings of controlling strata diameter 4 to 6 in. (10-15 cm))	(Sampling close to surface accessible exploration earth structures)
Common cohesive and plastic soils	Displacement, wash auger continuous sampling (percussion, rotary)	Augers 1- to 2-in. (2.5-5-cm) 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. (5-15 cm) thin-wall, drive or free piston sampler; 4- to 8-in. (10-20-cm) adv. trim, sample 8- to 12-in. (20-30-cm) 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. (2.5-5-cm) piston or open drive sampler (core retainers)	Thin-wall drive sampler with stationary piston	Thin-wall or composite drive samplers with stationary piston vacuum relief required	2- to 6-in. (5-15-cm) 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. (5-cm) diameter	Thin-wall drive sampler free or stationary piston vacuum relief or freezing bottom of sample required	2- to 6-in. (5-15-cm) thin-wall sampler; open or free or sta. piston; 4- to 8-in. (10-20-cm) adv. trim. sample depress ground water level

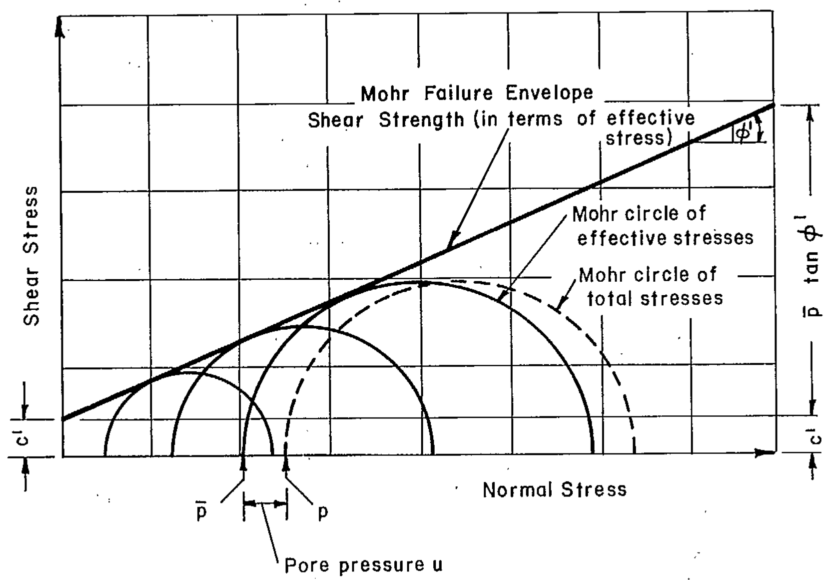
Compact or stiff and brittle soils including dense sand, partially dried soils	Wash, augers percussion, rotary continuous sampling	Augers and 1- to 2-in. (2.5-5-cm) thick-wall piston or open drive sampler	Medium-wall open drive or piston sampler; hammering may be required (partial disturbance)	Core boring may be better than drive sampling but danger of contamination in partially dry soils	4- to 8-in. (10-20-cm) adv. trim sample; 8- to 12-in. sq. (51-77-cm 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 (51-77-cm 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. (51-77-cm 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

cohesion c' and the effective angle of internal friction ϕ' are defined by the position and slope of the Mohr failure envelope.

131. Consolidation tests determine the compressibility of a soil and the rate at which it will consolidate when loaded. Consolidation test procedures are detailed in ASTM D2435 (15). The two soil properties usually obtained from a consolidation test are C_c, the compression index, which indicates the compressibility of a soil, and c_v, the coefficient of consolidation, which indicates the rate of compression under load.

132. 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 is whether or not the soil was preloaded in its past geological history.

133. Permeability test specimens may be either



c' = Effective cohesion

ϕ' = Effective angle of shearing resistance

$\bar{p}$ = Effective normal stress

p = Total normal stress

u = Pore pressure

$$\text{Shear strength} = S = c' + (p - u) \tan \phi'$$

$$\text{or} \quad S = c' + \bar{p} \tan \phi'$$

(Mohr diagram showing envelope of soil strength in terms of effective stresses, and relationship between effective and total stresses.)

Fig 17 - Mohr diagram

undisturbed or representative disturbed samples, depending on the purpose for which they are to be tested. Samples of foundation material 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 testing for permeability depends on the permeability range of the soil to be tested (Table 4). 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 (13). The permeability can also be estimated mathematically from grain size distribution using formulae developed by Hazen (7).

134. Approximate estimates of the relative density of cohesionless soils may be obtained in the field from the standard penetration test. This is conducted by driving a split-spoon sampler having dimensions of 1.375 in. (3.5 m) ID, 2 in. 5 cm) OD, into the soil, using a 140-lb (63-kg) weight dropping a distance of 30 in. (76 cm). The number of blows required to drive the sample spoon 12 in. (30 cm) is called the standard penetration resistance (N value) of the soil. Empirical relationships which allow correlation of penetration resistance to relative density have been developed (7). This relationship is illustrated in Table 9. Soundings with cone penetrometers can also be used for an approximate determination.

135. Approximate correlations have also been made between standard penetration test blow count, N, and the consistency of clay. This relationship (Table 9) should be used only as a guide.

136. Several methods are available for determining the in-place unit weight of soils in pits, excavations, and fills for earth dams or roadways.

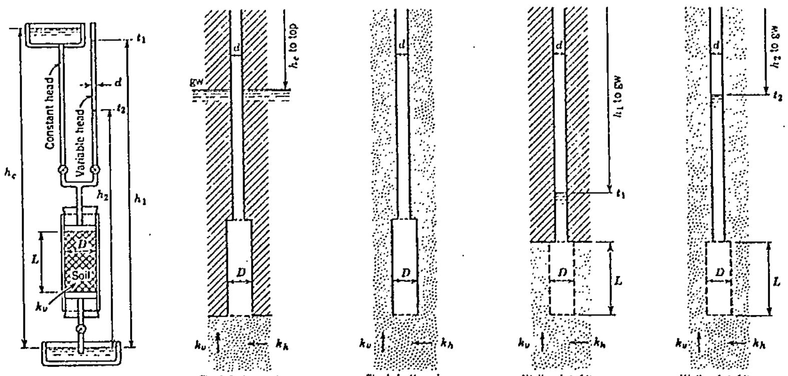
Table 9: Standard penetration vs relative density and consistency

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

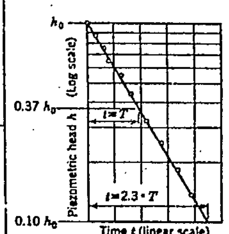
Cohesive soils

Consistency	No. of blows (N)	Unconfined compressive strength	
		tons/ft^2	kPa
Very soft	2	< 0.35	<23.9
Soft	2-4	0.25-0.50	23.9-47.9
Firm	4-8	0.5 -1.0	47.9-95.8
Stiff	8-15	1.0 -2.0	95.8-191.5
Very stiff	15-30	2.0 4.0	191.5-383.0
Hard	30	>4.0	>383.0



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 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 k_h = horz. 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 $\log_{10}$
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 $d = D$	
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 $d = D$	
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 $\frac{2mL}{D} > 4$	 Determination basic time lag T
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 $\frac{mL}{D} > 4$	

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.

Fig 18 - Formulae for determination of permeability (Hvorslev)

Two commonly used procedures are the sand cone method, ASTM D1556, and the rubber balloon method, ASTM D2167 (16). In recent years, nuclear methods for determining field densities and water contents have come into common usage (17, 18).

137. Several methods of conducting vane shear tests and static penetration tests in boreholes are used to evaluate in situ shear strength (19, 20). Occasionally, it may be desirable to conduct a large-scale standard shear box test in the field. A block sample of soil or rock is enclosed within a box and vertical and horizontal loads are applied. The load-displacement curve is then obtained and analyzed, as in the laboratory test.

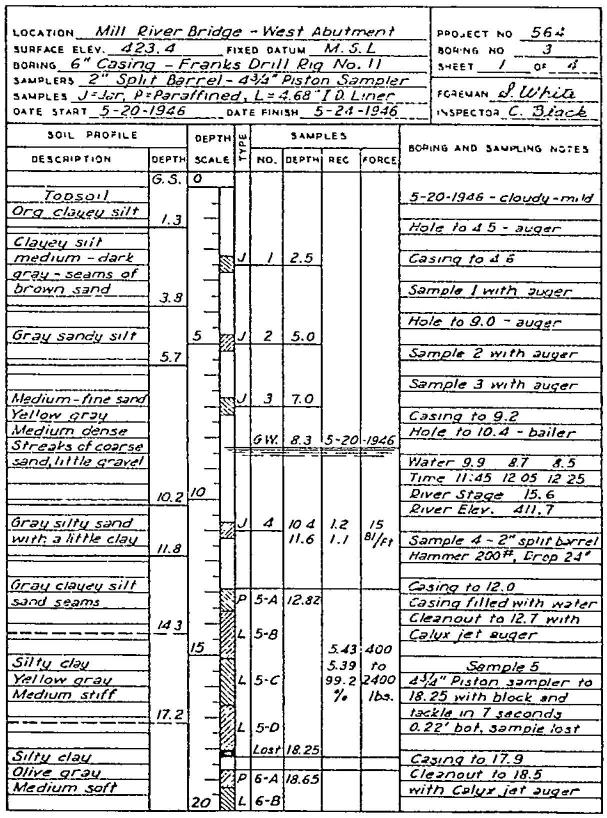
138. Two methods are commonly used for determining permeability of soils in the field: (a) in the infiltration test, water is introduced into a drill hole, well, or test pit and the rate of seepage observed under a fixed or variable head. A variation of this test is to pump or bail the water out of the hole and measure the rate of inflow. (b) In the pumping-out test, water is pumped from a borehole or a test pit at a constant rate and the drawdown of the water table is observed in wells placed for this purpose on radial lines at various distances from the pump.

139. Both methods have the advantage of testing the soil in situ, so that the effects of natural structure, stratification, orientation of the grains, and other natural properties are included in the tests. The pumping-out method is relatively expensive to perform. Computational methods are available to calculate soil permeabilities (21). Figure 18 illustrates a method of computing average permeability from the infiltration test data.

TEST HOLE LOGS

140. The logs of sub-surface investigations and sampling form the permanent record used in analyzing an embankment site. Thoroughness and accuracy of observations are therefore critical. Logs should not only contain data required for determining soil profile and location of the samples but also observations of details contributing to an appreciation of the condition of the samples and of the physical properties of the soil in situ. An experienced observer should be present continuously during drilling to supervise the sampling, prepare the logs and adapt the exploratory program to the conditions encountered.

141. Typical standard forms for logging samples are shown in Fig 19 and 20. A special effort should be made by observers to standardize interpretation of the various materials tested. Logging of samples should be kept up-to-date and logs prepared in final form as soon as possible after the hole is completed. This subject has been treated in considerable detail by Hvorslev (19).



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 - 4 3/4" 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 J. White
INSPECTOR C. Black

SOIL PROFILE DESCRIPTION	SOIL PROFILE DEPTH	DEPTH SCALE	TYPE	SAMPLES NO.	SAMPLES DEPTH	SAMPLES REC	SAMPLES FORCE	BORING AND SAMPLING NOTES
	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	3.8							Sample 1 with auger
								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	11.8				11.6	1.1	Bl/Ft	Sample 4 - 2" split barrel
								Hammer 200#, Drop 24"
Gray clayey silt								Casing to 12.0
sand seams			P	5-A	12.82			Casing filled with water
	14 3							Cleanout to 12.7 with
- - - - - - - -			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	4 3/4" Piston sampler to
Medium stiff						%	lbs.	18.25 with block and
	17.2							tackle in 7 seconds
- - - - - - - -			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				

Fig 19 - Typical field log for drilling (after Hvorslev)

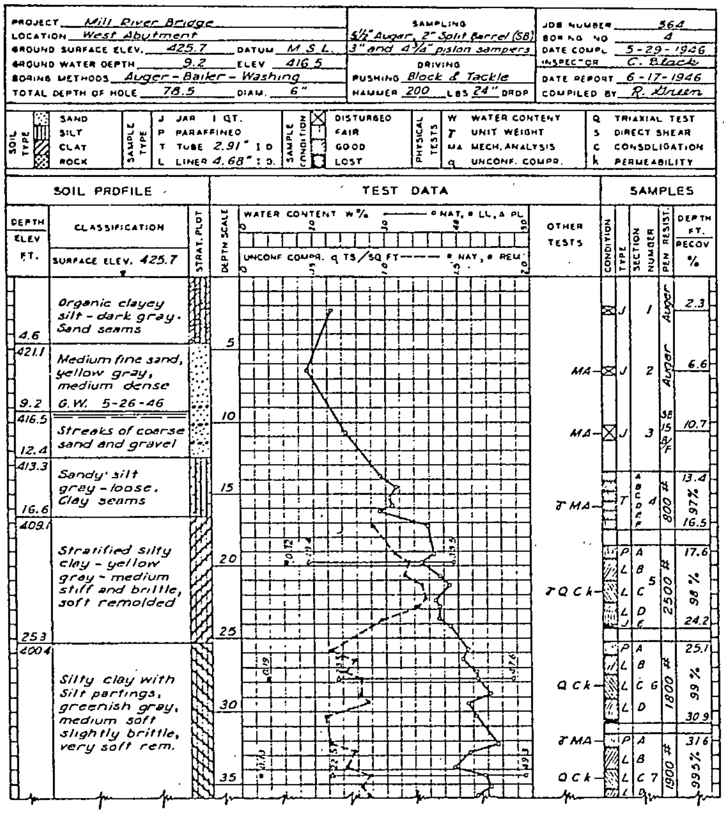


Fig 20 - Typical laboratory test log (Hvorslev)

MINE WASTES

142. The investigation of mine wastes is similar to that of soils. Factors requiring investigation particularly critical to stability of waste embankments are: degradation of the waste materials resulting in loss of shear strength, water levels in the embankments, sealing of embankment exteriors by weathering, effects of drainage water entering the embankments, and incompetent materials at the base of the embankments.

143. Investigations can include: drilling and sampling, as for soils; performance of similar wastes in existing embankments; accelerated degradation tests of waste material; instrumentation of embankments to monitor continuing behaviour, such as piezometers to measure water pressures; deformation measurements; and photographs taken at regular time intervals to record changes in appearance.

144. The strength properties of some waste material change after it has been placed. In some cases, experience with similar materials will indicate that particular wastes are likely to become unstable. In other cases, an apparently sound material may break down under the attack of chemicals or the atmosphere.

145. Materials that commonly degrade are: shales, siltstones, and mudstones. Often, when exposed to cycles of freezing and thawing, wetting

and drying, or to the stress relief by excavation, they will gradually lose their strength. If this loss of strength is substantial, a waste embankment designed on the basis of the strength of the original material may become unstable in future. Therefore, piles of wastes that are susceptible to degradation should be designed to be stable at the lowest strength that the material might achieve.

146. The final strength characteristics of degrading materials can be difficult to determine. Inspection in the field may reveal degraded material 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 material that attacks and influences the waste.

147. 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 waste material is degradable, it may be desirable to provide confining structures of competent material to retain it. In some cases, the strength of the material may be maintained if the waste pile is isolated from such agents as water and air that cause loss of strength.

148. In summary, the best procedure to be followed in investigating material suspected of degrading in storage is to: investigate it in situ and under the conditions to which it might be exposed in storage; perform accelerated degradation tests to duplicate, as closely as possible, the conditions expected in the waste pile, (the result of these tests may indicate the approximate lower-level of strength that might be developed or methods to prevent degradation); perform in situ 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, and a detailed examination of the pile can be conducted which may reveal an exposure where accelerated degradation is already evident.

149. It is sometimes necessary to investigate an existing tailings embankment or waste pile without access to the results of earlier investigations, design or construction procedures. This investigation may be necessary to assess the present state of stability or to evaluate the area as a site for disposing of additional waste material.

150. The investigation of existing embankments is similar to that described for embankment foundations except that additional information may be obtained from persons who may previously have worked on the site. Every effort should be made to obtain information from this source, particularly on the general methods of construction, equipment used, the initial state of the foundation, and distribution of material within the embankments. Investigation equipment and drill sites can then be chosen to provide the necessary information with the least possible outlay of time and expenditures. In general, drilling and sampling should extend well into the foundation beneath the embankment; drill holes should be instrumented for continuing observation of pore water pressures. Where waste embankments are suspected of incipient failure, instrumentation should be provided to monitor movement and provide warning of developing instability and corrective measures should be initiated to improve stability of the embankment.

DESIGN

GENERAL

151. Stability of mine waste embankments must be assured in most cases. Instability of waste embankments located in isolated areas may not involve serious risk to adjacent property or installations at the time they are constructed but may become a serious hazard due to nearby development at some time after being abandoned. Such unstable waste embankments could endanger persons working on or near them.

152. The methods of design commonly used for tailings embankments are described, followed by those for waste piles. Since methods of analyzing are common to both, they are described last.

TYPES OF PROBLEMS

153. Earth slopes commonly manifest instability in the form of a rotational slide, as illustrated by Fig 21. In this type of instability, the surface along which movement occurs closely approximates a segment of a horizontal cylinder. This is characteristic of sliding in slopes composed of materials which possess both cohesive and frictional components of shear strength. Slopes of cohesionless materials may also exhibit this form of instability when the foundation materials under the slope are instrumental in bringing about a slide.

154. 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 material close to the toe, may also occur. The rate at which rotational slides are developed is unpredictable. In some instances, the rate of movement may be slow enough to allow prompt remedial action to arrest the movement. In others, large rotational movements occur so rapidly that insufficient time is available.

155. Changes in the water table in a waste embankment will change the pore water pressures and consequently the resistance of the pile to sliding. Increases in water level on dumps can be

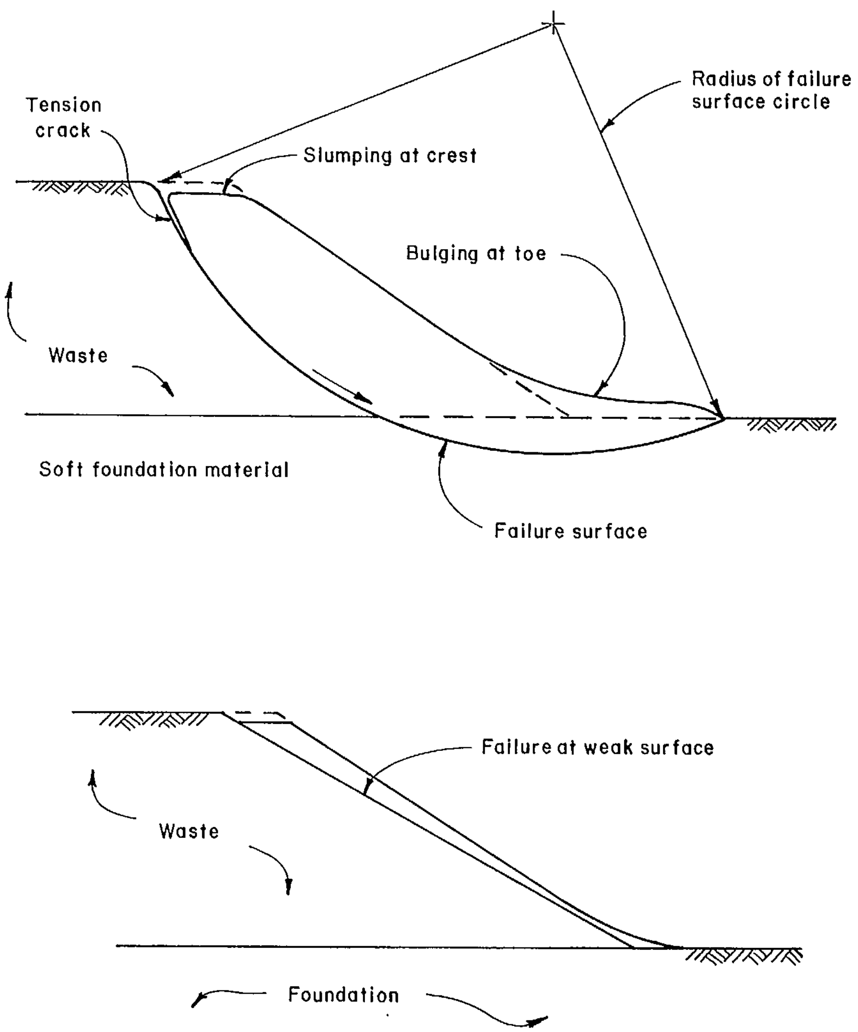


Fig 21 - Typical instability modes. Top: rotational sliding. Bottom: plane shear.

caused by: surface water seeping into a waste pile, springs located under the pile and not effectively drained, seepage water from accumulation of water on the pile, blockage of drainage culverts beneath or around the waste pile, and changes in the characteristics of waste materials placed on the pile. In tailings embankments, changes in 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, changes in methods being used to construct the embankment and changes in elevation of the water pool retained behind the embankment.

156. Changes in permeability of foundation materials below waste embankments are sometimes caused by strains induced by mining subsidence which can also affect the level of the water

table.

157. Whenever an earth dam retains water, seepage will occur through the embankment and there will be some build-up of water pressure in the pores of the soil. The pore water pressure in the soil, adjacent to the interface between the upstream slope and the pond, will be equal to the water pressure in the pond. If the water level drops in the pond, the rate at which the pore water pressure dissipates from the embankment will depend on the permeability of the embankment soil. If the drawdown of the water level in the pond is rapid and the upstream zone of the embankment is relatively impervious, the unbalanced pore water pressure may be sufficient to cause a slide in the upstream slope of the embankment.

158. The drawdown condition does not present a serious hazard with tailing dams when they are operated as a normal tailings dam with only a minimum depth of water in the pool to provide a clear effluent. They can present a hazard in the special case where there is a considerable depth of water impounded behind a tailings dam which has a relatively steep and impervious upstream slope.

159. Disturbances of waste embankments can reduce their resistance to sliding. Such disturbances may be caused by: vibration 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. Particularly where waste material is 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, cracks may develop along which no shearing resistance can be mobilized.

160. 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. It is a special case of rotational sliding where the radius of the sliding surface is large and the failure surface almost planar.

161. Planar surfaces, and other modifications of the cylindrical surfaces occurring with rotational sliding, can be caused by weak material beneath the slope. Such weak 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 material included in a pile of coarser waste, and foundation strata of low shear strength. With these conditions, sliding surfaces may follow the surfaces of weakness producing non-circular or wedge-shaped slides as illustrated in Fig 22.

162. The low shear strength, and in some cases the low permeability, of weak strata can result in sliding along surfaces located partially within the foundations or abutments of mine waste embankments.

163. The principal foundation characteristics affecting stability of waste embankments are: shear strength, compressibility and permeability. Compression or consolidation of the foundation can cause appreciable settling of the overlying material, sometimes causing cracks in tailings embankments which lead to excessive seepage and piping.

164. The contours of the foundation or variation in 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 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 in Fig 23.

165. The permeability of a foundation can have a significant effect on the stability of an embankment. When an embankment is constructed on a foundation of saturated impervious clay, the loading of the embankment will create excess pore water pressure in the foundation material. Because the immediate loading is taken by the

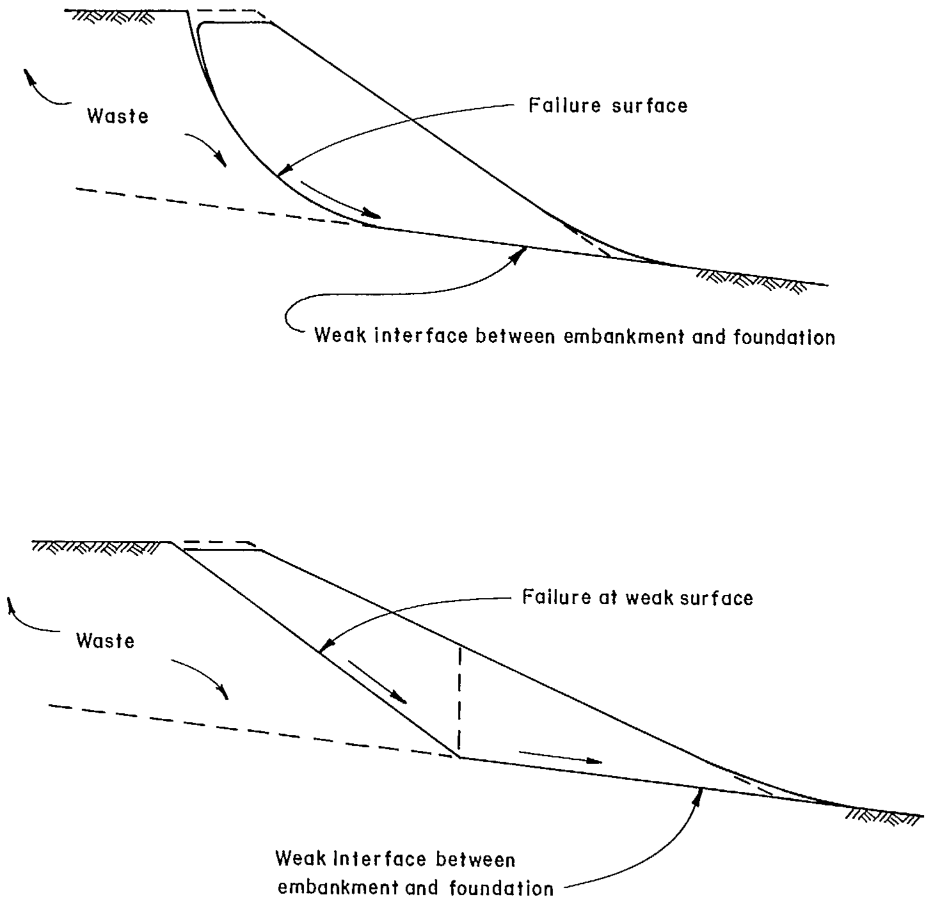
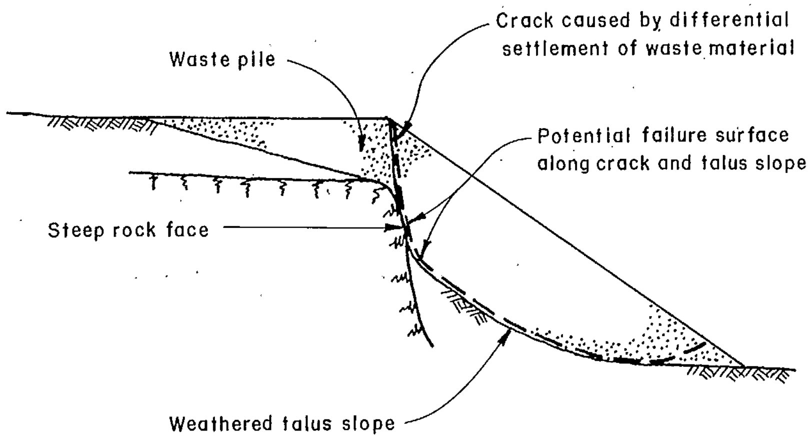


Fig 22 - Typical instability modes. Top: non-circular sliding. Bottom: wedge instability

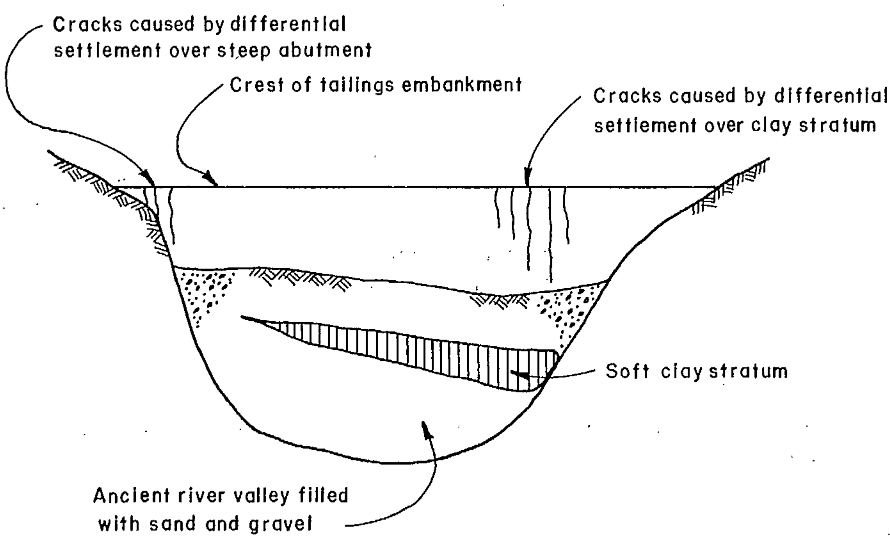
water phase in the foundation material, there is no increase in shear strength and the rapid increase in loading can precipitate embankment failures which extend through the foundation. On the other hand, if the foundation for a tailings dam is pervious, excess seepage under the dam during the early years of operation may lead to serious piping. Examples of foundation conditions which affect the stability of an embankment are illustrated in Fig 24.

166. Continuous movement of a mass of material forming a slope, at a slow and steady rate and parallel to the slope is known as creep, particularly where the material moves as a consequence of deformation rather than along a defined sliding surface. Locating 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 explicit cause of such deformation. The apparent creep of a slope may thus develop into a slide along a defined surface.

167. A mud flow occurs in the form of a rapidly moving stream of waterborne soil having the consistency of mud. It is usually caused by the saturation of soil masses by heavy rainfall or



DIFFERENTIAL SETTLEMENT OVER STEEP FOUNDATION SURFACE



DIFFERENTIAL SETTLEMENT OVER SOFT FOUNDATION STRATUM

Fig 23 - Differential settlement affecting stability

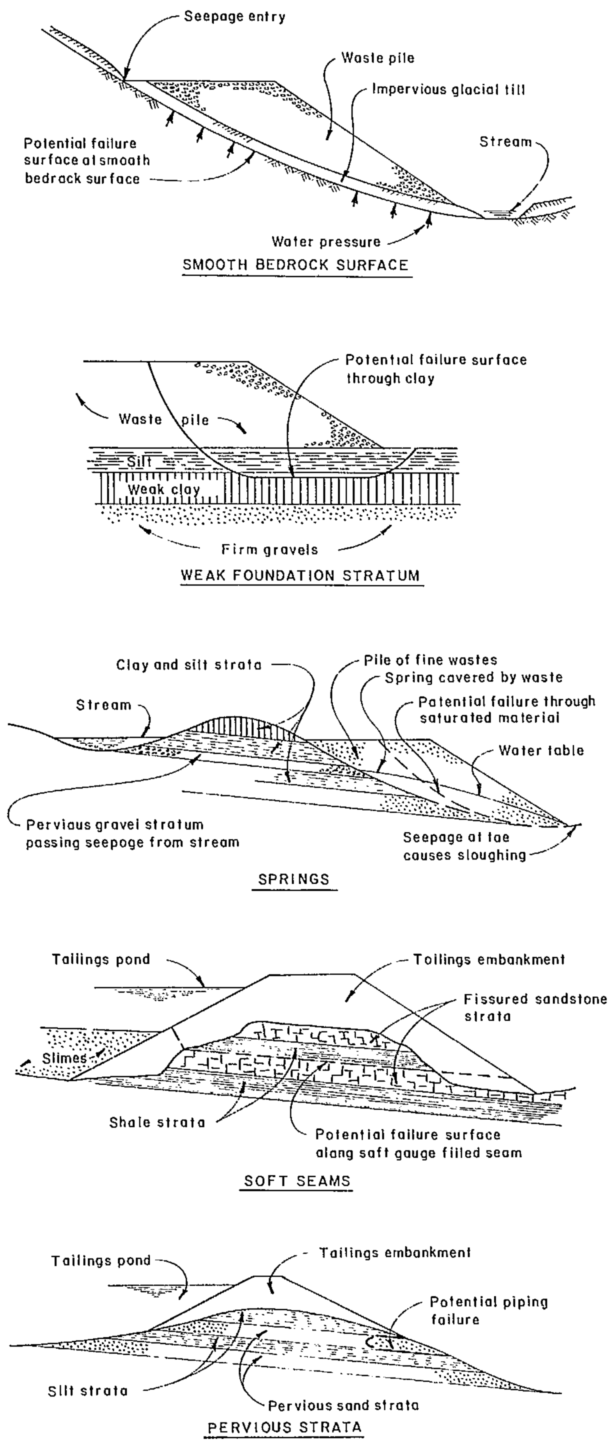


Fig 24 - Conditions giving rise to potential instability

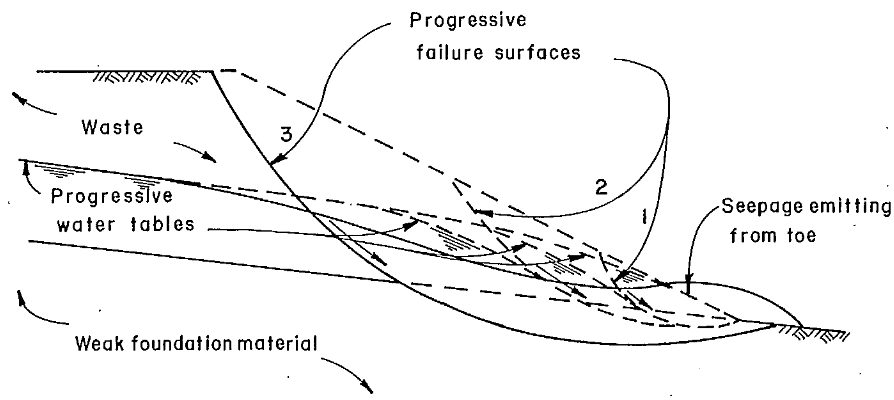
springs and commonly occurs at the downstream toe of tailings embankments and at the toe of waste piles constructed of relatively fine impervious material. Dumping over-the-side on steep slopes can produce critical conditions.

168. Softening or swelling of some soils and waste material in contact with water will often result 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 since 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.

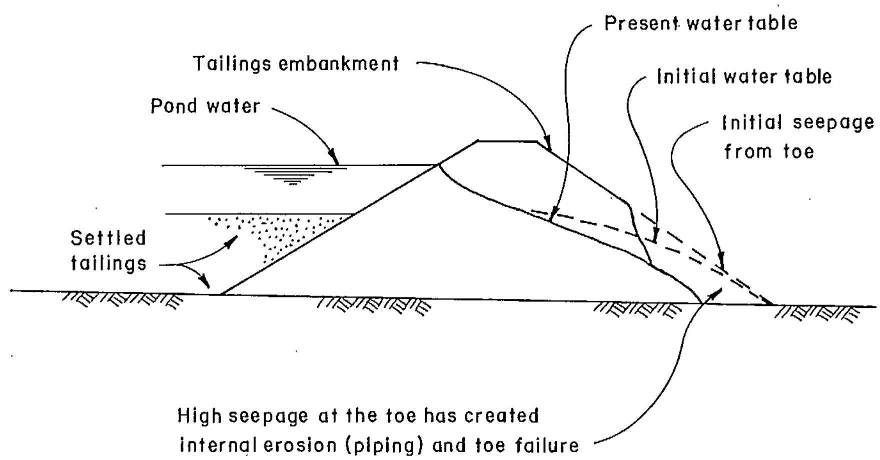
169. Susceptible materials at the surface or toe of a waste pile, particularly where there is ponding, are most likely to exhibit softening or swelling. This may cause local sliding and hence sudden changes in stress within the adjacent intact pile or its foundation. Material which has softened or swollen may constitute surfaces of weakness within any waste pile built over them. Similarly chemical solutions seeping through a pile or its foundations may react with the waste and bring about changes in its shear strength.

170. Where a minor slide occurs, as may happen as a result of seepage, subsequent slides of progressively increasing size may occur repeatedly above the first slide; the material from each slide being removed by seepage water at the toe of the slide area. Similar slides may occur where the material from a minor slide is excavated without restoring the original geometry of the slope. The form taken by progressive sliding is illustrated in Fig 25.

171. Flow slides occur in some soils, such as very fine-grained uniform sand and cohesionless silt or rock flour which adopt a loose unstable structure when deposited through water. 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 by drainage from the voids



PROGRESSIVE FAILURES



PIPING FAILURE

Fig 25 - Progressive slides and piping

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.

172. Large volumes of liquefied soil can flow through relatively narrow openings and can travel considerable distances. Loose soils deposited at void ratios above a 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

frequently satisfy the criteria which identify soils susceptible to liquefaction. A number of breaches of tailings dams have been followed by liquefaction of the saturated tailings slimes behind the embankment.

173. Overtopping of an earth embankment by water usually results in serious damage and may even cause washout of the entire embankment.

174. The significant parameters which determine the extent of damage as a result of overtopping of an earth dam are rate of flow over the crest, duration of flow over the crest, and the erodibility of the material in the embankment and its foundation.

175. The rate of flow and the length of the embankment crest govern the depth and velocity of initial flow over the crest. With a relatively low flow and 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 concentrated with rapid increase in flow velocity. With a sustained high flow through such a breach in an embankment of erodible materials, a major failure can occur very rapidly.

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

177. In areas of high runoff, particularly where the slopes are long, erosion gullies may form on the slopes of the waste embankment. A common location is the contact line between the embankment and the abutment where runoff from adjacent hillsides will concentrate. If a deep gully is permitted to develop, this will promote slides of adjacent waste material on the slope of the embankment. When sufficiently deep, the gully will intersect the water table within the embankment causing an increase in the rate of seepage, possibly leading to a piping failure.

178. A common form of sub-surface erosion is piping which sometimes occurs on the downstream slopes of earth dams and tailings embankments. This is caused by seepage through the embankment or its foundation. The seepage flow usually concentrates in relatively pervious layers or in lenses of material more pervious than the average. If the exit velocity of the seepage water from the soil is above the critical velocity, 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. Cohesive material or damp cohesionless material, may arch over the developing hole and progressive erosion will develop a pipe extending back towards the source of seepage. The development of this pipe decreases the length of the seepage path with consequent acceleration of the seepage and of the 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. The development of piping is illustrated in Fig 25.

179. This type of instability can occur rapidly if initial development is not noticed. The piping does not necessarily occur at exposed ground or embankment surfaces but often occurs beneath the ground surface by erosion of fine material into the voids of coarser material. An example would be the movement of grains of tailings sand into the open voids of a rockfill embankment. Piping is promoted where there is a tendency to concentrate seepage along a confined path, such as along the smooth external surface of a culvert or decant pipe passing through an embankment. Numerous such cases have occurred in tailings embankments by erosion near decant lines under an embankment. Seepage diaphragms should thus always be included in the design of a decant tunnel to prevent

external seepage.

TAILINGS POND

180. 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 theoretically. Although settling velocities of various types and grain sizes of solids can be determined theoretically and experimentally, many factors influence 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 effluent discharge. Factors affecting settling time are the size of grind, the tendency to slime as with clay type minerals, pH of the water, wave action, depth of water and distance between the tailings and effluent discharges (22).

181. The grind required to liberate the valuable mineral is usually under 200 mesh. Particles in the range of 300 mesh or 50 µm with a settling rate of 0.05 in. per second (0.12 cm/sec) can be affected by wind action but will settle in a reasonable time. The major problem is caused by particles of 2 µm or less which produce turbidity. These have settling rates of less than 0.01 in. per second (0.025 cm/sec) in still water and, under conditions prevalent in most tailings ponds, require some days to settle in the turbulence caused by wave action.

182. Various rules for clarification have resulted from observations of existing ponds. Among these are that the pool should be of a size to allow 5-days retention time and that the area of the pool should provide 10 to 25 acres (4.04 to 10.11 ha^2) of pond area for each 1,000 tons of tailings solids delivered per day. An average of 15 acres (6.07 ha^2) per 1,000 tons is usually considered adequate, unless unusual conditions are present.

TAILINGS EMBANKMENT DESIGN

183. The design of tailings embankments depends on the method of construction, particularly when the primary embankment material is sand obtained from the tailings slurry. In this case, 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.

184. Embankment construction methods are described after the Design section. An overriding factor will be the need to keep the embankment crest above the pond surface. This can affect the entire design and basic construction methods. Where tailings sand is the principal embankment material, one of three basic placement procedures can be used. These, and the types of embankment cross-sections resulting from their use, are shown in Fig 26. When other borrow or dry-waste material is incorporated in the embankment, many alternative types of embankment are possible. Some of these are illustrated in Fig 27 to 29.

185. Designing a tailings embankment is a process of successive trials and refinements (23-37). Generally, the steps required to develop the final cross-section are as follows:

a. determine the long-term storage volume and schedule of storage requirements,
b. investigate alternative disposal areas from topographic data and use the potential ratio of storage volume/dam volume for preliminary site selection,
c. determine other possible types and quantities of construction materials available,
d. assess major constraints relating to property acquisition and environmental regulations,
e. determine the proposed method of tailing disposal, ie, select a point of discharge remote from the dam, spigots and hydrocyclones,
f. select a trial embankment section incorporating the most economic and readily available fill material,
g. make a stability analysis for the trial section to determine the factor of safety. The stability analysis should take into account shear strength and density of the material comprising both the foundation and the embankment as well as the expected 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

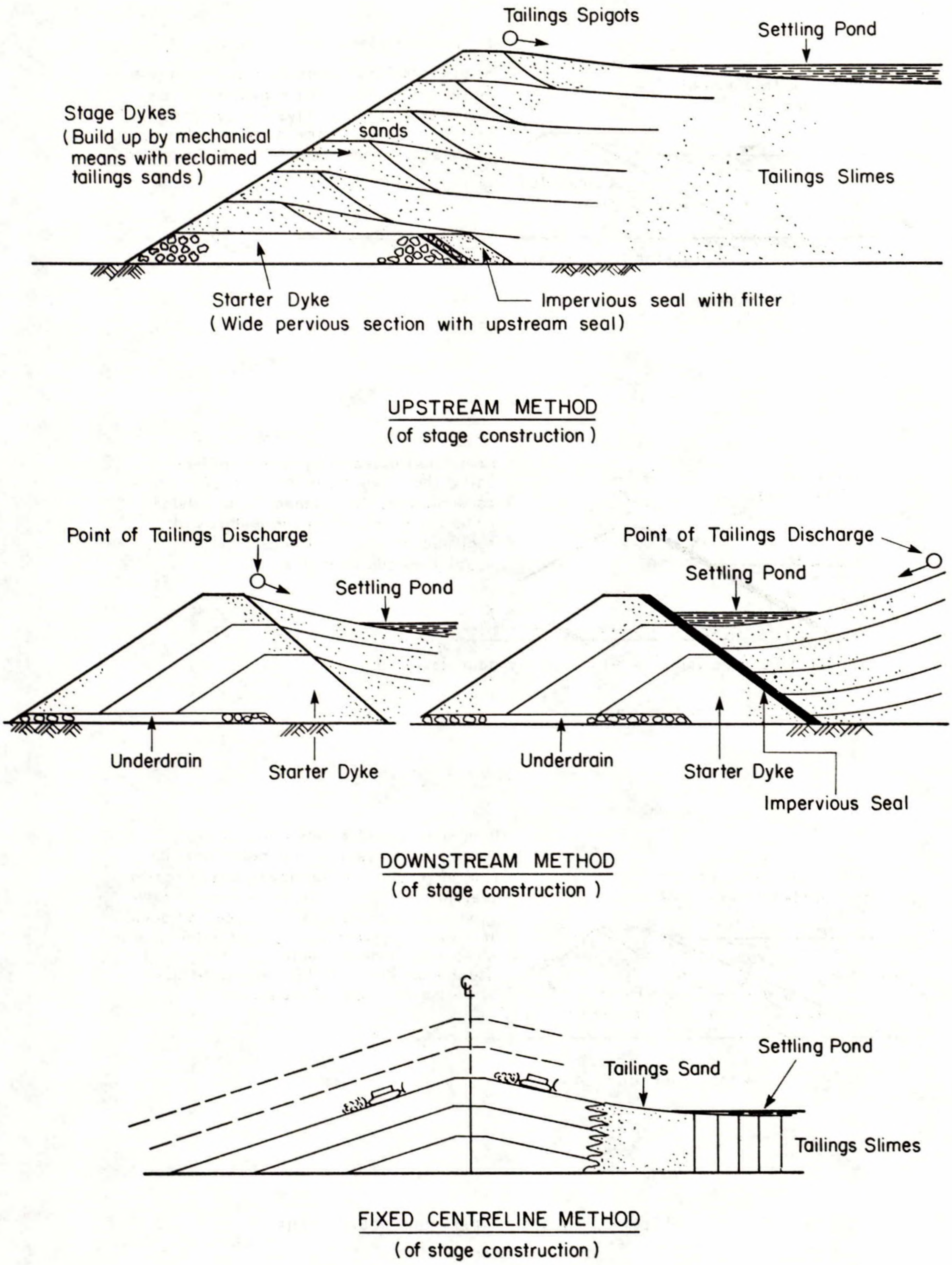
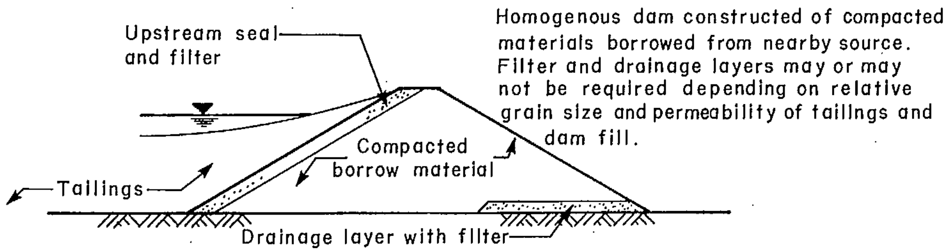
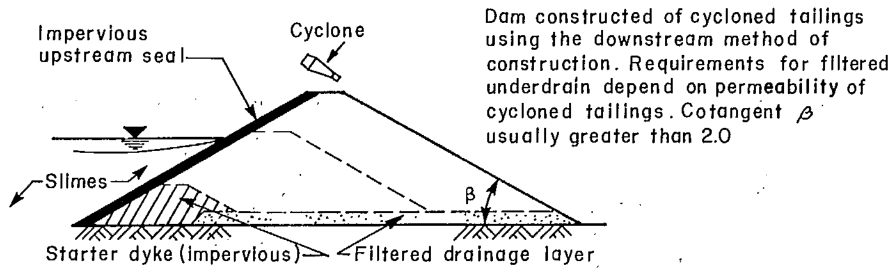


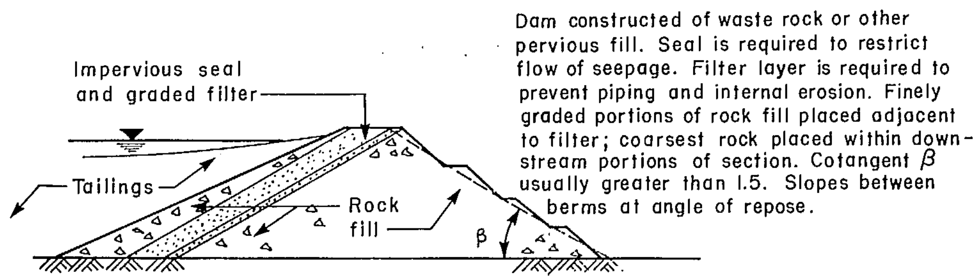
Fig 26 - Tailings embankment construction methods



(a) Homogenous dam



(b) Cycloned tailings dam



(c) Waste rock dam

Fig 27 - Tailings dams on impervious foundations

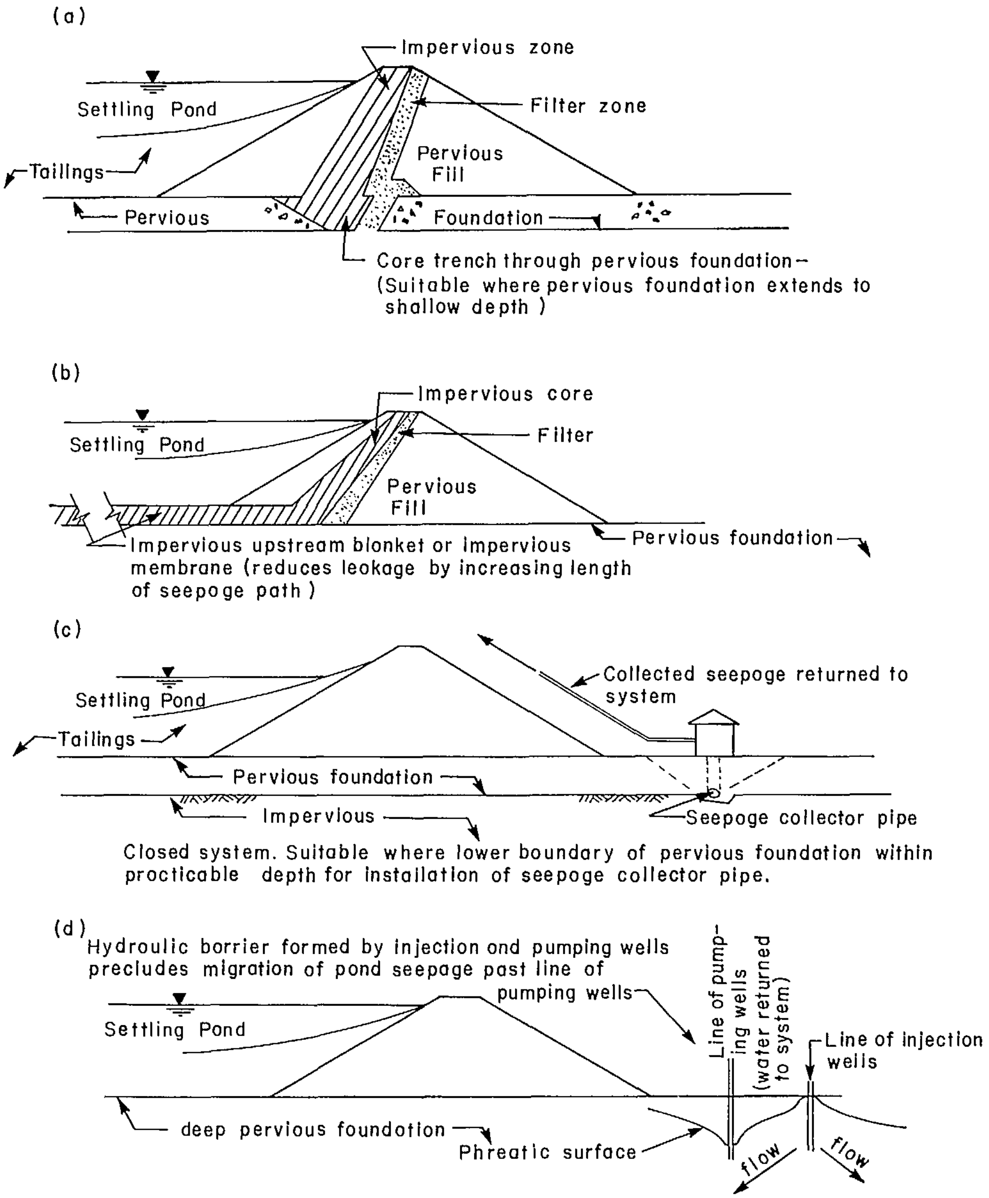
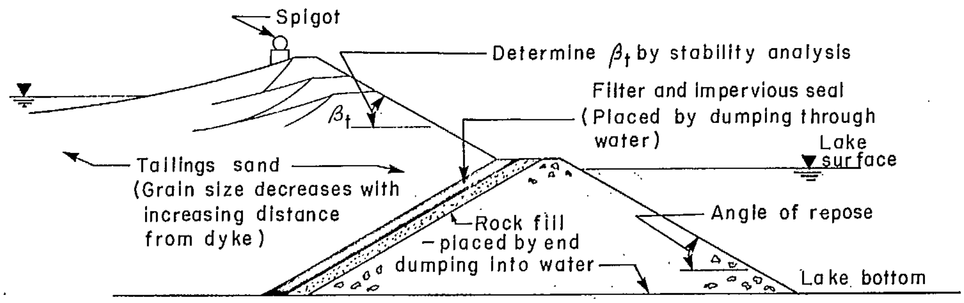
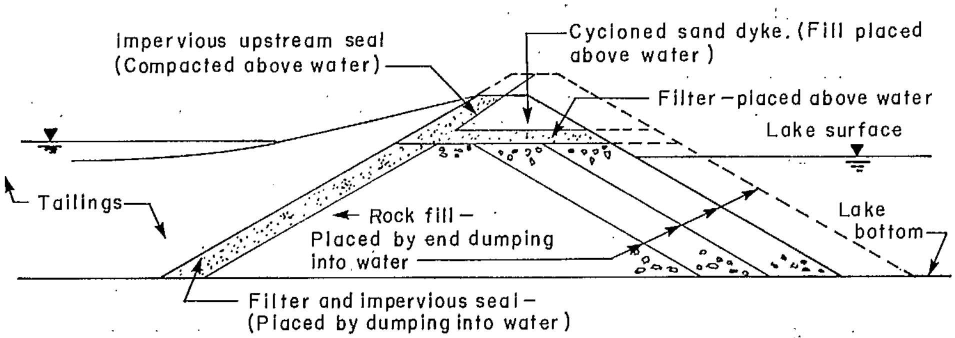


Fig 28 - Tailings dams on pervious foundations showing control of seepage loss



TAILINGS DAM CONSTRUCTED IN WATER
– UPSTREAM METHOD OF CONSTRUCTION



TAILINGS DAM CONSTRUCTED IN WATER
– DOWNSTREAM METHOD OF CONSTRUCTION
(Suitable where large quantities of rock fill readily available)

Fig 29 - Tailings dam construction in water

from flow nets. If compressible foundation strata are located beneath the embankment, foundation pore pressures estimated on the basis of consolidation 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 indicates that the section is unsafe, or that the factor of safety is unduly high, the section should be modified and the stability analysis repeated until a satisfactory section is developed, and

h. prepare detailed construction drawings and specifications for foundation treatment, fill placement and waste disposal.

SEEPAGE CONTROL

186. Seepage will occur whenever there is a differential head of water across an earth dam; however, the quantity can be controlled within reasonable limits. As for tailings dams, it is desirable to minimize seepage. The principal reasons for controlling seepage are to improve and maintain stability of the structure, to minimize pollution effects from seepage water, and to conserve water which can be reclaimed for processing.

187. Steady flow of an incompressible fluid

through a porous medium can be expressed mathematically. The flow net, a graphical solution, is a grid formed by the intersection of two sets of orthogonal lines. One set, 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 equipotential lines, are piezometric contours representing the loci of points having the same pressure head. The flow net is used to estimate the rate of seepage flow and to predict the piezometric pressures at any point within the embankment cross-section.

188. Flow nets may be constructed using graphical procedures (sketching by hand), electric analogs or model studies. Alternatively, the equations may be solved by digital computer using either the finite-element or the finite-difference methods (38, 39).

189. For many problems, flow nets produced by sketching 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 desirable if the material properties and boundaries are well known. Several examples of flow nets are illustrated in Fig 30, 31 and 32. General rules for their construction, using graphical procedures are outlined in Fig 32. A more detailed treatment of flow net construction by sketching has been given by A. Casagrande (40).

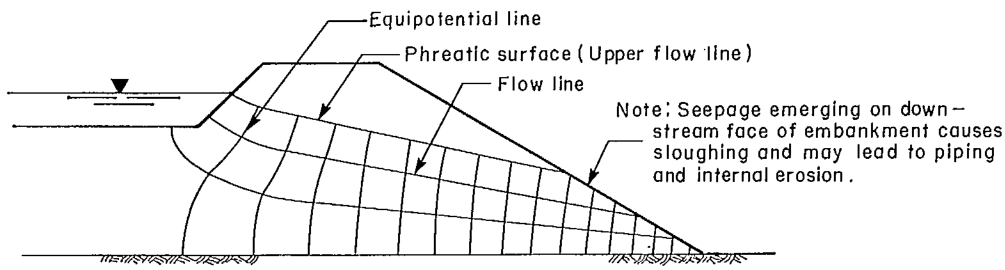
190. Economic considerations frequently dictate that the tailings embankment be constructed using the most readily available fill material commensurate with adequate stability of the structure. The position of the phreatic surface or water table within an embankment has a marked influence on the slope angle required for stability. If the permeability of the embankment fill is of the same order of magnitude or less than the tailings adjacent to the embankment, drains should be provided beneath the downstream zone to lower the phreatic surface. The drainage system may consist of chimney drains, blanket drains, finger drains, toe drains, drainage pipe or a combination of internal drainage methods.

191. Suitable drainage provides the following advantages: (a) the phreatic surface will be lowered in the downstream zone of the dam, thereby avoiding the problem of sloughing along the downstream slope at a point where seepage might otherwise exit; (b) lowering the phreatic surface reduces the pore water pressure and increases stability of the embankment section, thereby permitting steeper downstream slopes and requires less fill to achieve the desired factor of safety, and (c) the internal drainage system can be designed to permit seepage water to drain below the frost line, reducing the possibility of ice lensing (which creates an impervious layer and causes buildup of pore water pressures) and surface sloughing with subsequent thawing. Figures 30 and 31 illustrate the effectiveness of underdrains and pervious foundations in lowering the phreatic surface.

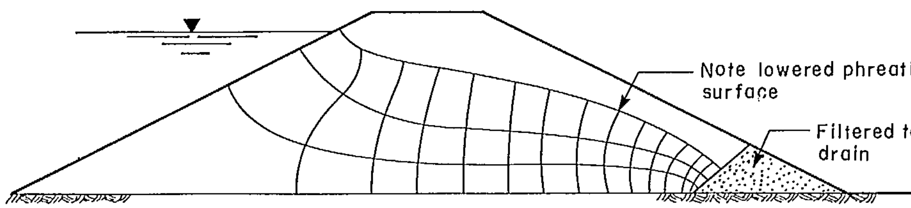
192. The choice of drains depends on the availability of suitable drainage materials, drainage capacity required, cost of construction and foundation conditions. Permeability of the drain material should be at least 100 times greater than the permeability of the adjacent embankment material and its gradation must satisfy filtering requirements.

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

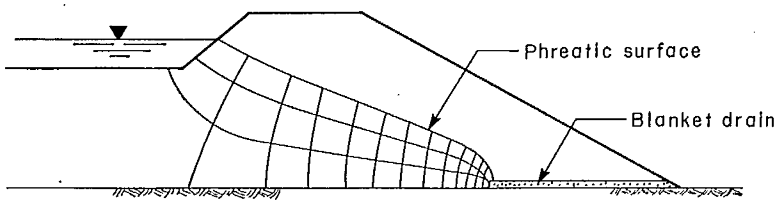
194. Where pipe drainage is used, the pipes should be designed to withstand the maximum anticipated loads, including those imposed by settling of the overlying fill. If perforated pipes are used, the perforations should not be at the top or at the bottom so as to minimize the entry of solids and prevent loss of seepage water once it has entered the pipe. The perforations should not be larger than half of the 85% passing size of the drainage material surrounding the pipe. Larger pipe perforations can be used if the pipe is wrapped with a woven nylon mesh of filter



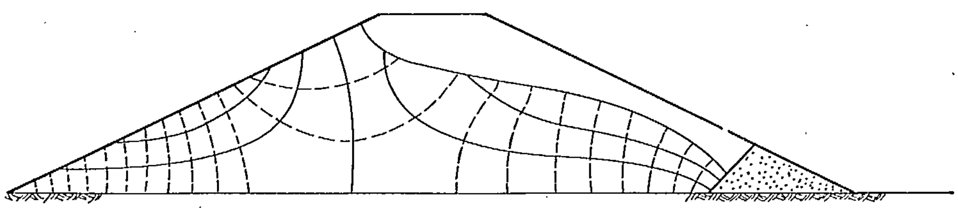
HOMOGENEOUS SECTION



HOMOGENEOUS SECTION WITH TOE DRAIN

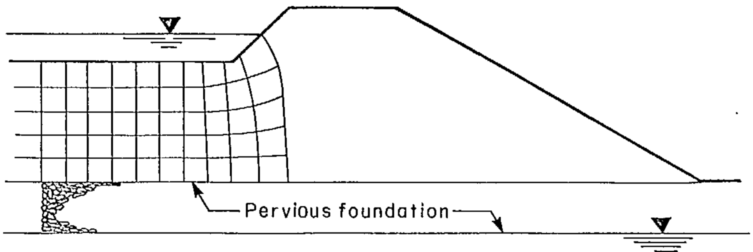


HOMOGENEOUS SECTION WITH BLANKET TOE DRAIN

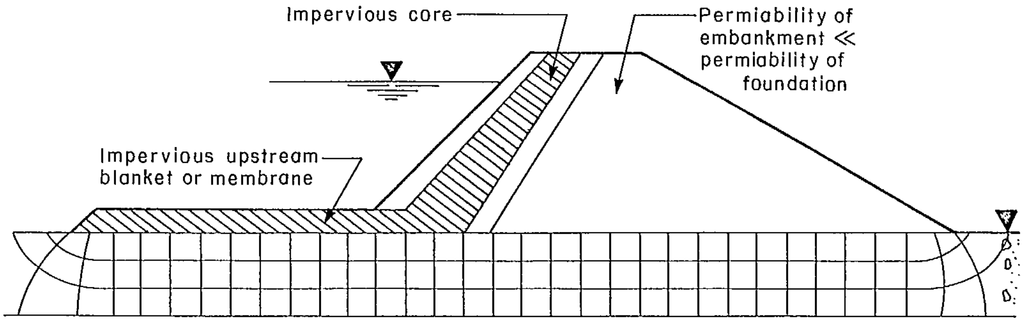


FLOW NET FOR RAPID DRAWDOWN CONDITION

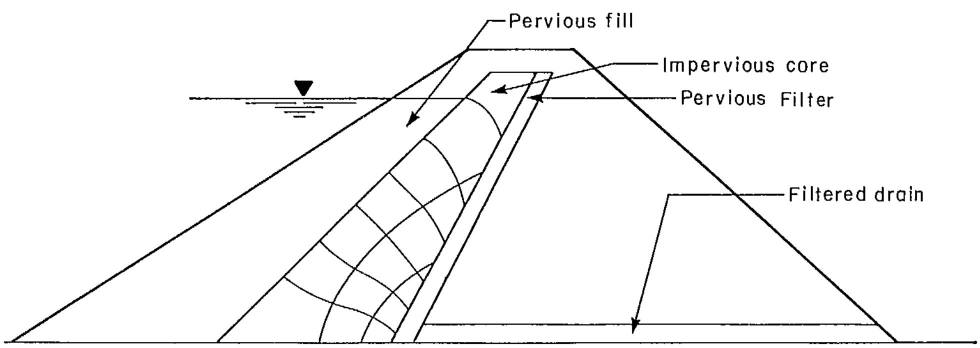
Fig 30 - Flow nets for embankments on impervious foundations



FLOW NET FOR TAILINGS EMBANKMENT AND POND
UNDERLAIN BY PERVIOUS FOUNDATION



FLOW NET FOR SEEPAGE THROUGH PERVIOUS FOUNDATION
IMPERVIOUS–UPSTREAM BLANKET OR MEMBRANE USED
TO INCREASE LENGTH OF SEEPAGE PATH



APPROXIMATE FLOW NET FOR
SEEPAGE THROUGH IMPERVIOUS CORE

Fig 31 - Examples of tailings embankment flow nets

specification. Pipe drains can seldom be repaired. In view of the serious consequences resulting from the collapse of pipe sections, or opening of joints, pipe drainage systems should be avoided; finger drains and blanket drains with suitably graded filters are preferable.

195. Finger drains consist of strips of pervious drainage material placed on the foundation, and in some cases at higher levels also, prior to placing overlying embankment fill. The arrangement and alignment of strip drains will be governed by contours of the foundation surface. The drains should be provided with adequate fill to outlets located beyond the downstream toe of the embankment. All internal drainage systems should be protected with a graded filter to pre-

A flow net is a graphical representation of the flow of seepage water through saturated soil. It is a pattern of lines formed by flow lines intersecting equipotential lines. Typical flow nets are shown in Fig 30 and 31. When material is isotropic with respect to permeability, the flow lines and equipotential lines intersect at right angles.

Rules for Construction

1. First sketch the phreatic surface. The upper boundary or top flow line of a flow net is at atmospheric pressure and is called the phreatic surface. In an earth dam where there is a differential head of water across the embankment, the phreatic surface will approximate a parabola over the major portion of the curve. For a homogeneous section, sketch a parabola with the focus at the toe of the dam (or the drain) and one point on the surface of the reservoir, 30% of the horizontal distance of the upstream wetted slope, as shown in Fig 32(a).
2. An integral number of equipotential lines intersects the phreatic surface at points spaced at equal vertical intervals along the top flow line. Divide the differential water head by a whole number and sketch in the equipotential lines. Then draw in flow lines to intersect the equipotential lines at right angles, forming approximate squares. 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. A discharge face through which seepage passes is an equipotential line if the discharge is submerged.
4. A discharge face through which seepage passes is a flow line (or 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 permeabilities of the layers exceeds 10, the flow in the more permeable layer is the controlling factor. That is, the flow net for the more permeable layer may be drawn 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 interface in accordance with Fig 32(f).
7. If the materials are anisotropic with respect to permeability, the cross section should be transformed by changing the scale as indicated on Fig 32(d). The flow net is then drawn for isotropic materials after which it is redrawn to the true scale. (Fig 32(d) and 32(e)).
8. If only the quantity of seepage is to be determined, an approximate flow net suffices. If pore pressures are to be determined, the flow net must be accurate.

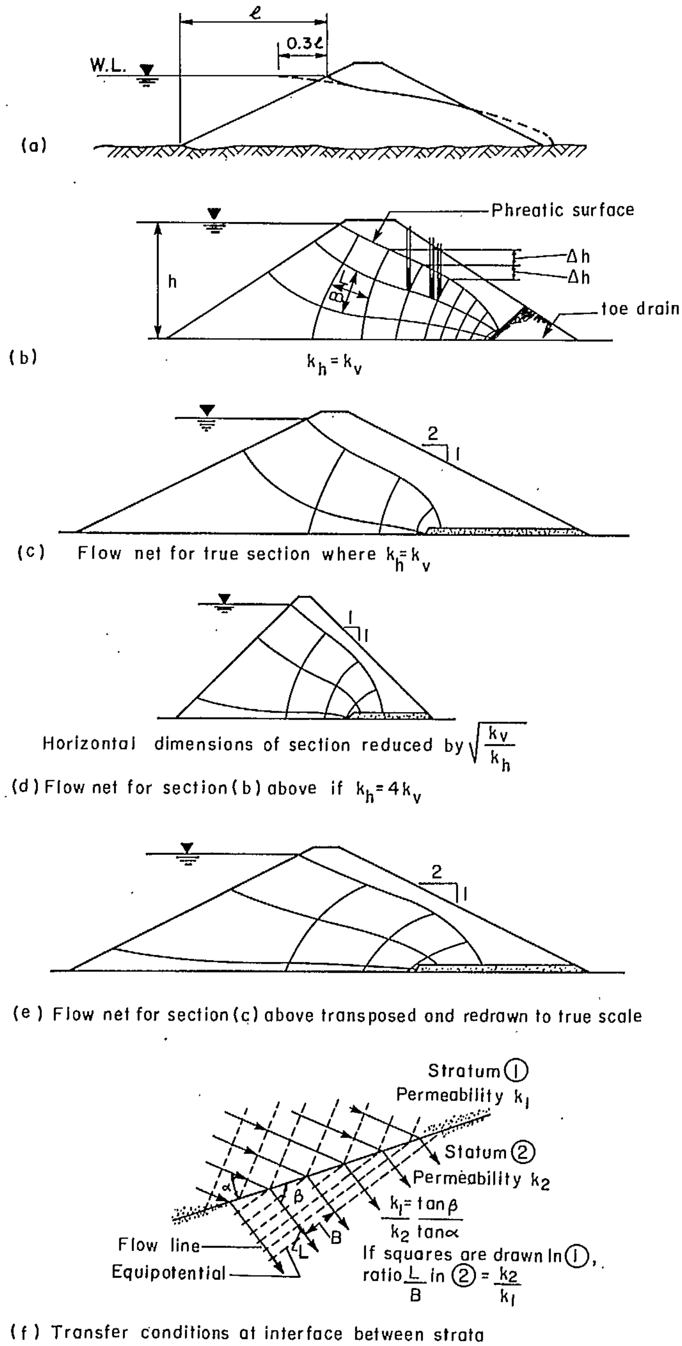


Fig 32 - Flow net construction

vent clogging of the drain.

196. 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 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 making these calculations.

197. Determine thickness of the drainage blanket or the dimensions of finger drains, to ensure that their capacity is greater 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 severly restricted, in which case an impermeable foundation should be assumed. Dimensions of the drains should be as generous as practicable commensurate with the quality and cost of the material available and the need to construct the drains without constrictions, gaps, or segregation of material. The construction of all internal drainage systems for earth dams should be rigidly controlled to assure the quality of this component. The thickness of blanket and finger drains should be at least 12 in. (30 cm), and the width of the drain should not be less than 10% of the difference in elevation between the pond surface and the drain.

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

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

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

201. Filters are required to prevent internal erosion of the fine material where the seepage water passes from finely-graded to coarsely-graded material. 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 fine 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 placing.

202. To ensure that the above criteria are satisfied, the following rules should be applied in selecting suitable filter material. Rules 1, 2 and 3 are illustrated in Fig 33. In the following rules, D_{15} represents the diameter of the 15% passing size, suffix F refers to filter material and suffix B refers to protected material.

Rule 1: $\frac{D_{15\ F}}{D_{B5\ B}} < 5$

Rule 2: $\frac{D_{15\ F}}{D_{15\ B}} > 5 \text{ and } < 20$

Rule 3: $\frac{D_{50\ F}}{D_{50\ B}} < 25$

Rule 4: The filter material should be a filter within itself:

$\frac{D_{85\ F}}{D_{15\ F}} < 5$

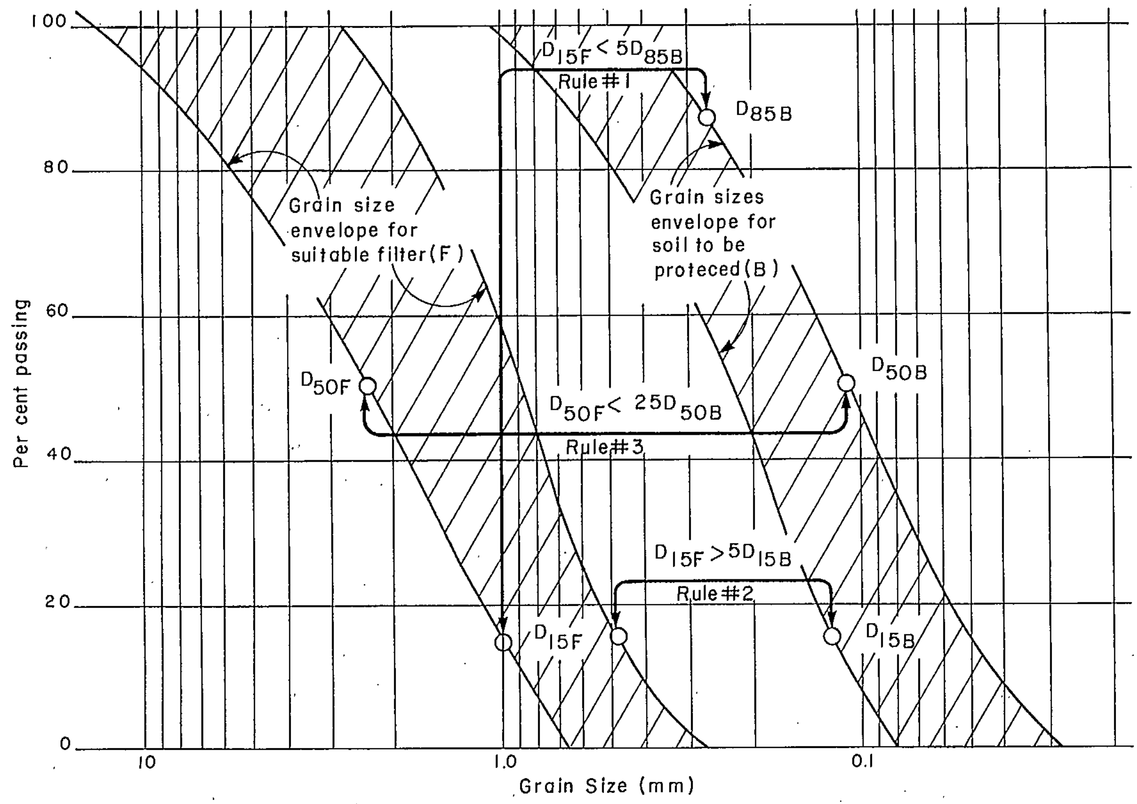


Fig 33 - Gradation requirements for filters

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

Rule 5: Filters should not contain more than 5 wt % passing the 200 mesh sieve.

Rule 6: In the special case where drainage pipe is used:

$$\frac{D_{85\ F}}{\text{Maximum opening of pipe drain}} \geq 2$$

Rule 7: Where the protected material contains a large percentage of gravel, the filter should be designed on the basis of the gradation curve of the portion of the material which is finer than 3/8 in. (10 mm) sieve.

Some designers have found it convenient to combine Rules 1 and 2 as follows:

$$\frac{D_{15\ F}}{D_{85\ B}} < 5 < \frac{D_{15\ F}}{D_{15\ B}}$$

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

204. As illustrated in Fig 33, Rules 1 and 3 should be applied to the fine side of the grain size envelope 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 envelope representing the minus 3/8 - in. (10 mm) fraction of the soil.

205. To ensure that the filter will be appreciably more permeable than the coarsest zones of the soil to be protected, Rule 2 is applied to the coarse limit of the protected soil and to the fine limit of the filter.

206. To provide a suitable filter between fine-grained soil and very coarse fill (for example, a coarse rock fill) it may be necessary to use more than one graded filter layer. 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.

207. The design thickness of filter zones will be dependent on the inclination of the zone and 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 sufficient to permit efficient operation of the construction equipment. Filter zones placed by hand in confined spaces can have a minimum thickness of 6 in. (15 cm).

208. Where embankment foundations are subject to artesian pressures, stability requirements may necessitate relief wells to control pore water pressures beneath the embankment (41). Water from the relief wells should be discharged in suitably constructed ditches or pipe drainage systems located downstream from the toe of the embankment.

209. The required spacing between relief wells will depend on the hydrogeology of the foundation; publications describing their design are listed in the bibliography. However, since unknown variations in the stratigraphy and the permeability of the foundation soils may have an appreciable effect on the drawdown, 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.

210. 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 against 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 the perforated piping or well screens will be governed by flows anticipated but should be no less than 6 in. (15 cm). To assure free flow of water into the well casing, screens should be surrounded with drainage material conforming to the gradation requirements for filters. Monitoring of relief wells by piezometers and flow measurements are essential to detect deterioration and to indicate necessary maintenance.

211. 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, by the hydraulic gradient, and by the coefficients of permeability of the material 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, or low-permeability barrier across the path that the seepage must follow in passing through and beneath the structure.

212. The mathematical expression for the rate of seepage is:

$$q = k \frac{n_f}{n_d} h, \qquad \text{eq 5}$$

where q is the rate of seepage per unit length perpendicular to the plane of the flow net, k is the coefficient of permeability of the soil, n_f is the number of flow paths determined from the flow net, n_d is the number of equipotential drops determined from the flow net, and h is 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.

213. 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 supply of mill make-up water usually necessitate the use of impervious cores, blankets, or membranes to reduce the rate of seepage from the pond.

214. 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 impervious 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 can be maintained within as much of the embankment cross-section as practicable. A core trench is illustrated schematically in Fig 28(b).

215. As an alternative to the cutoff trench, impervious 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 reduce permeability by increasing 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 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 used.

216. As an alternative to methods designed to minimize seepage, 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 or returned to the tailings pond or to the milling operations.

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

218. The hydraulic barrier, illustrated in Fig 28(d) can be produced by a line of pumping wells and a line of injection wells, the latter located downstream from the pumping wells. Fresh water is supplied to the injection wells, while seepage water is extracted from the pumping wells. Provided the piezometric water level along the line of the injection wells is maintained at an elevation higher than the piezometric water level 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 condition should be checked in the field by piezometric measurements.

RUNOFF CONTROL

219. Tailings basins and waste piles should preferably not be located in natural water courses. If it is necessary to locate a disposal area in a stream bed, the stream must be diverted around the disposal area.

220. There will always be some catchment area contributing runoff into a tailings basin or waste embankment. This may vary from a minimum area encompassing the perimeter of the tailings basin to a substantial watershed above the tailings dam.

221. The tailings basin effluent system must be designed to have sufficient capacity to handle the maximum inflow into the basin and maintain a minimum freeboard on the dam during the peak flow. It should therefore be designed to handle the peak 24-hour flood flow, with a recurrence interval of 100 years, plus the maximum production flow from the tailings system. In some instances, the production flow may only represent 5% of the peak flood flow. The common methods of handling tailings effluents are by decant tower and conduit

through the dam, a weir spillway, and reclaim pump-barge on the tailings pond.

222. The minimum desirable freeboard on the dam should be maintained during conditions for peak flood flow. To minimize design capacity of the tailings effluent system, an emergency spillway can be installed in the crest of the dam to handle the flood runoff capacity for the tailings basin watershed. As an alternative to an emergency spillway, the dam can be designed with excess freeboard to accommodate the total flood runoff below the minimum desirable freeboard elevation. In many instances, after water diversion, the watershed for the tailings basin is only slightly larger than the tailings basin itself, making it relatively easy to accommodate the flood runoff with extra freeboard. The most critical period will usually occur during the early years of waste disposal when the storage capacity behind the dam is relatively small. Evaporation is not a critical factor in the maximum design capacity because the peak flood occurs during a relatively short period. It is important to make provision for runoff after abandoning a tailings basin.

223. The effects of runoff can include: overtopping and potential failure of a tailings dam when sufficient freeboard or decant capacity have not been provided, surface erosion or waste piles with resulting down stream pollution, and a decrease in stability of waste piles and tailings embankments resulting from an increase in pore water pressure or erosion from runoff.

224. Methods for the design of diversion channels and spillways are described in readily available hydraulics handbooks. Several of these are listed in the references (42-47). Usually, the most critical point in their design is avoiding erosion affecting the safety of the embankment. For this reason, the gradients of diversion and spillways channels should be kept sufficiently flat that erosive velocities will not occur near the embankments. Alternatively, channels may be protected against erosion with various kinds of lining or with stone paving. The magnitude of permissible flow velocities for various classes of natural soils and the size of paving stones required to prevent erosion are given in Table 10 and Fig 34 and 35. To be effective in preventing erosion of underlying fine soils, paving stones should be founded on a layer of filter gravel graded as described previously.

EMBANKMENT FREEBOARD AND WAVE PROTECTION

225. In addition to the freeboard required for the maximum flood flow and maximum tailings capacity, minimum freeboard should be provided on tailings embankments to prevent overtopping of the embankment by waves. The height of wave depends on wind velocity, duration of wind, the fetch or distance over which the wind can act on the water and depth of water. For most tailings ponds, the maximum wave height is governed by the fetch distance.

226. 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 some 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 wave action. Riprap could be necessary on tailings embankments constructed across the bays of natural lakes or on completed embankments which impound a substantial pond of water. The approximate wave height for various values of wind velocity and fetch, and the necessary freeboard and riprap gradation for 3:1 riprapped slopes, are given in Table 11. For 2:1 slopes, the nominal thickness should be increased by 6 in. (15 cm). With fine-grained embankment material, a layer of filter gravel should be placed beneath the riprap.

227. The minimum freeboard should be measured from the maximum projected flood water level to the crest of the embankment. The maximum flood level will be a function of the type and capacity of the decant system or spillway provided to accommodate the runoff flows.

MINIMUM EMBANKMENT CREST WIDTH

228. The most suitable crest width for a tailings embankment will depend on the allowable percolation distance through the embankment at maximum pond level, on the height of the structure and

Table 10: Maximum permissible velocities in unlined waterways

Original material excavated	A* ft/sec	A* m/sec	B** ft/sec	B** m/sec	C*** ft/sec	C*** m/sec
Fine sand, non-colloidal	1.50	0.46	2.50	0.76	1.50	0.46
Sand loam, non-colloidal	1.75	0.53	2.50	0.76	2.00	0.61
Silt loam, non-colloidal	2.00	0.61	3.00	0.91	2.00	0.61
Alluvial silts, non-colloidal	2.00	0.61	3.50	1.06	2.00	0.61
Ordinary firm loam	2.50	0.76	3.50	1.06	2.25	0.69
Volcanic ash	2.50	0.76	3.50	1.06	2.00	0.61
Fine gravel	2.50	0.76	5.00	1.52	3.75	1.14
Stiff clay, very colloidal	3.75	1.14	5.00	1.52	3.00	0.91
Graded, loam to cobbles, non-colloidal	3.75	1.14	5.00	1.52	5.00	1.52
Alluvial silts, colloidal	3.75	1.14	5.00	1.52	3.00	0.91
Graded, silt to cobbles, colloidal	4.00	1.22	5.50	1.68	5.00	1.52
Coarse gravel, non-colloidal	4.00	1.22	6.00	1.83	6.50	1.98
Cobbles and shingles	5.00	1.52	5.50	1.68	6.50	1.98
Shales and hardpans	6.00	1.83	6.00	1.68	5.00	1.52

* A - clear water, no detritus.

** B - water transporting colloidal silts.

*** C - water transporting colloidal silts, sands, gravel or rock fragments.

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

on the requirements of construction. For equipment operation, the width of the crest should not be less than 12 ft (3.65 m). For embankments under 100 ft (30 m) in height, a suitable minimum crest width is given by the equation:

$$W = \frac{z}{5} + 10 \qquad \text{eq 6}$$

where W = crest width in ft,

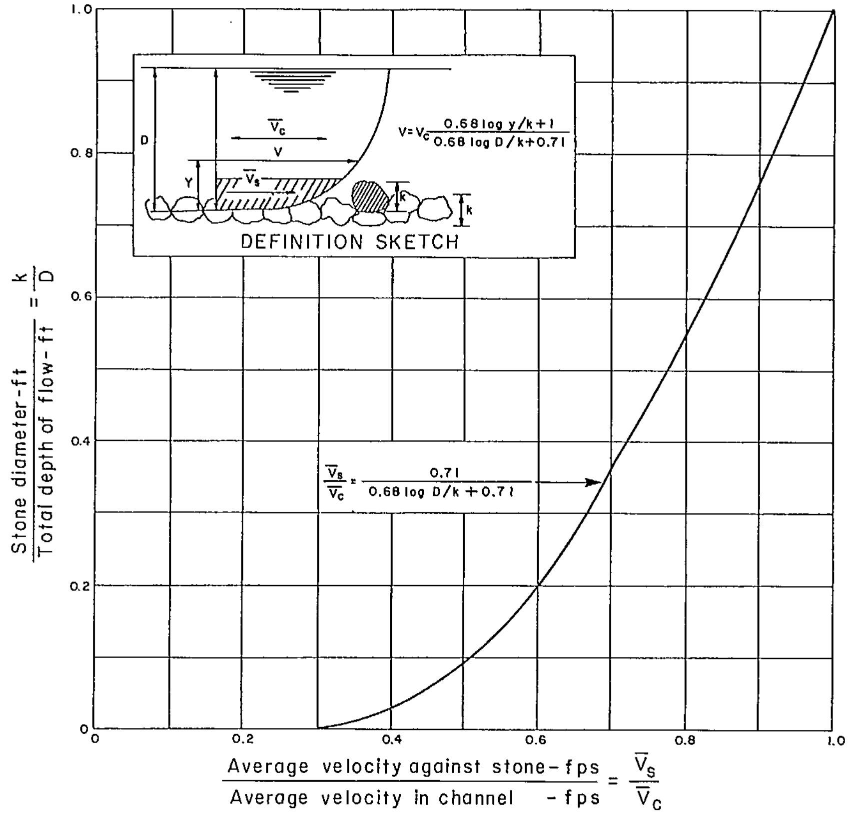
z = height of the crest above the foundation at its lowest point in ft.

Tailings dams over 100 ft (30 m) in height should have crests not less than 30 ft (9 m) in width.

WASTE ROCK

229. To evaluate a proposed waste disposal area, the designer must know the approximate volume of waste material 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 material and the foundation material should be known.

230. Since stability of both the waste and foundation material is influenced by pore water pressures, the construction of waste piles across major drainage courses should be avoided or,



Notes:

Equations apply to two-dimensional flow

$\overline{V}_S$ = Average velocity against stone – fps

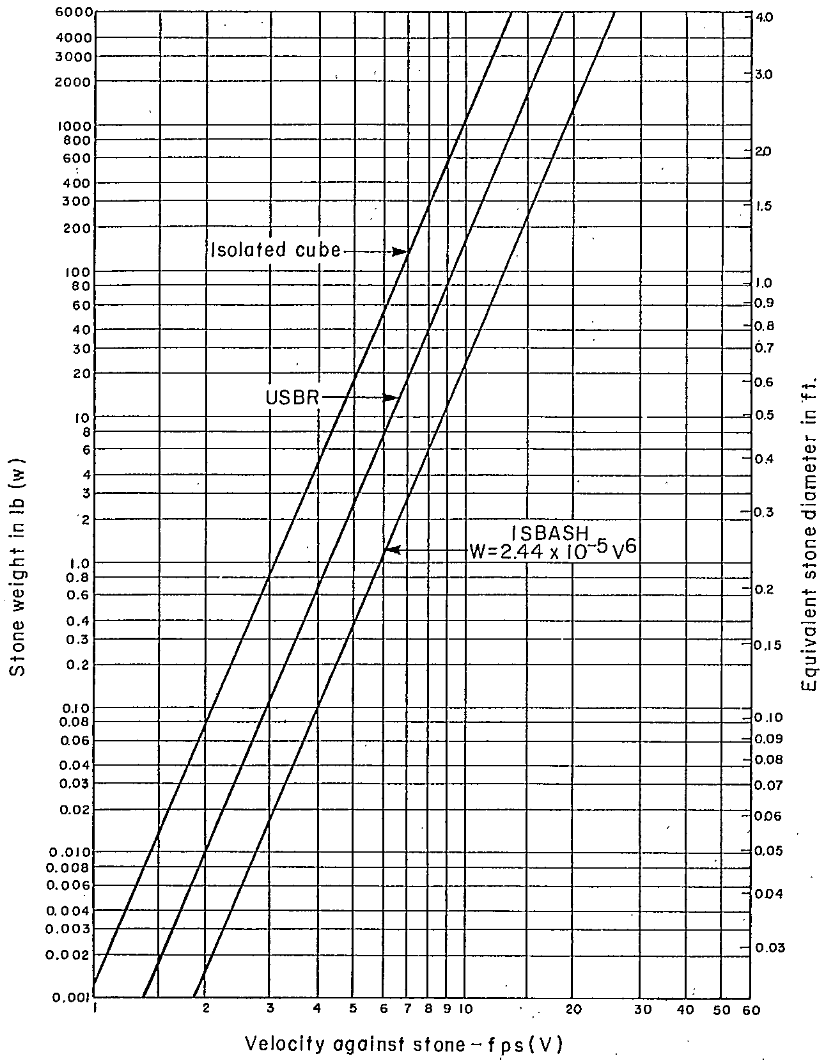
$\overline{V}_C$ = Average velocity in channel – fps

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.

Fig 34 - Average velocity against stone paving on a channel bottom



Note:
Specific weight of rock = 165 lb/cu ft (2642 kg/m^3)

Fig 35 - Stone paving velocity vs stone weight (US Army Corps of Engineers)

Table 11: Embankment freeboard and wave protection

Approximate wave heights

Fetch miles	Fetch km	Wind velocity miles/hr	Wind velocity km/hr	Wave height feet	Wave height m
1	1.6	50	80	2.7	0.8
1	1.6	75	120	3.0	0.9
2.5	4.0	50	80	3.2	1.0
2.5	4.0	75	120	3.6	1.1
2.5	4.0	100	160	3.9	1.2
5	8.0	50	80	3.7	1.1
5	8.0	75	120	4.3	1.3
5	8.0	100	160	4.8	1.5
10	16.1	50	80	4.5	1.4
10	16.1	75	120	5.4	1.6
10	16.1	100	160	6.1	1.9

Freeboard required for wave action

Fetch miles	Fetch km	Normal freeboard feet	Normal freeboard m	Minimum freeboard feet	Minimum freeboard m
<1	<1.6	4	1.2	3	0.9
1	1.6	5	1.5	4	1.2
2.5	4.0	6	1.8	5	1.5
5	8.0	8	2.4	6	1.8
10	16.1	10	3.0	7	2.1

Note: Freeboard should be calculated above maximum design flood-level in the reservoir.

Riprap required on 3:1 slopes for protection against waves

Reservoir fetch miles	Reservoir fetch km	Nominal thickness ft	Nominal thickness m	Gradation, per cent of stone of various weights (lbs) maximum size	25% greater than	45% to 75% from to	25% less than*
≤1	≤1.6	1.5	0.45	1,000	300	10- 300	10
2.5	4.0	2.0	0.61	1,500	600	30- 600	30
5	8.0	2.5	0.76	2,500	1,000	50-1,000	50
10	16.0	3.0	0.91	5,000	2,000	100-2,000	100

Note: *Sand and rock dust less than 5 per cent.

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 piles, ditches should be constructed along the uphill side of the waste disposal area to intercept and divert surface runoff water, and to conduct it beyond the lateral limits of the proposed disposal area.

231. 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 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 material is placed; the height of the waste pile; and slopes at the perimeter of the pile.

232. The height of the pile and its perimeter slopes are both governed by the shear strength of the materials in the pile and foundations. In general, the height of the waste pile can be increased if the slope is flattened and, conversely, the slope can be steepened up to angle of repose if the height of the pile is reduced.

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

234. The strength of the waste materials is governed by the type of material, its 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.

235. 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.

236. Foundations for waste piles may be classified according to the strength characteristics of the sub-surface materials and according to the topography of the ground surface. As used here, the term "competent foundation" refers to foundation material which exhibits shear strength higher than the waste materials; the term "weak foundation" refers to foundation materials that exhibit lower shear strengths than the waste. They are therefore relative to the strength characteristics of waste materials.

237. "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.

238. A waste pile on a level foundation, will generally be safe against mass sliding along its base unless an extremely weak surface layer such as organic material is present. Where a waste pile is to be constructed on a sloping foundation, an analysis should be made to check stability of the pile with respect to mass displacement as a result of shearing along its base. The slope of the ground surface in some areas may be so steep as to be unsuitable for disposing of waste material.

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

$$\tan i \leq \frac{\tan \delta}{FS} \qquad \text{eq 7}$$

where i is the slope of the foundation, δ is the friction angle between the base of the pile and its foundation, and FS is the factor of safety.

240. 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 conditions, the stability against shearing along the base of the pile should be checked by the wedge method of stability analysis as described in Fig 36. If the stability analysis indicates that the factor of safety for the proposed waste pile would be unacceptably low, the steeper portions of the prospective area will be unsuitable for waste disposal. In cases where surface areas with compound slopes are used for waste disposal, a stability analysis may indicate it is necessary to place waste material on the flatter portions of the slope before placing it over the steeper slopes, assuming adequate support throughout the disposal period.

241. 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.

242. The factor of safety of a waste pile will

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 angle δ to the horizontal. Note that the neutral forces (water pressure across the boundaries between adjacent wedges act horizontally.

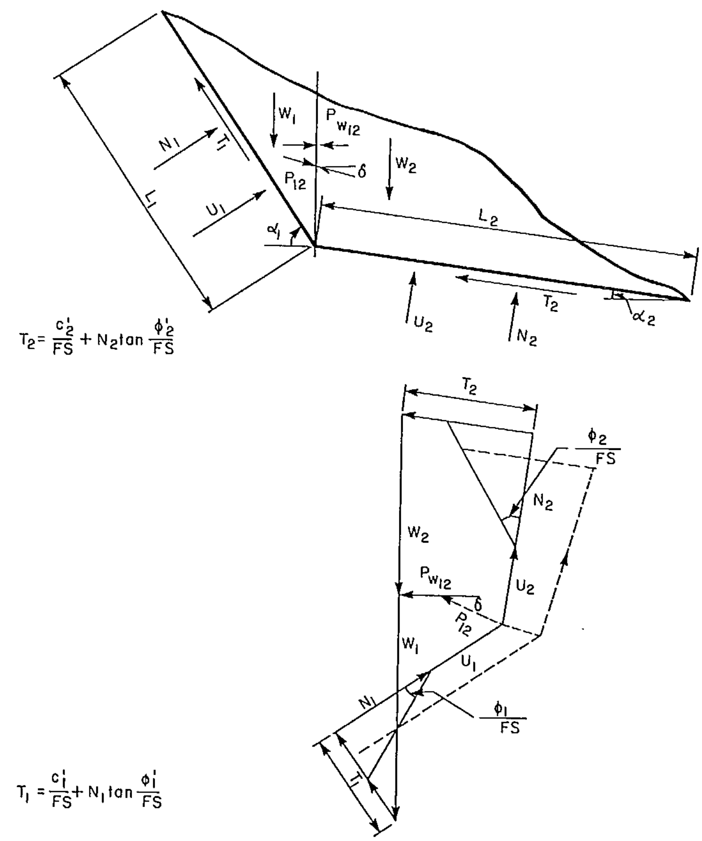
4. Choose a trial factor of safety (FS).

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 FS and repeat steps 5 and 6 above until a value of FS 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. The assumption that $\delta = \phi$ leads to an overestimation of the factor of safety. The assumption that $\delta = \phi/3$ usually leads to reasonable results.



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
α_1, α_2 Inclination of the base to the horizontal
P_{W12} Resultant hydrostatic force at interface
δ Inclination of P to the horizontal
ϕ_1, ϕ_2 Effective friction angle on base of wedge
c_1, c_2 Effective cohesion on base of wedge
P_{12} Effective force at the interface

Fig 36 - Procedure for analysis of wedge instability most likely be at a minimum and instability will most likely develop, following a period of prolonged rain. After prolonged rain, the near-surface soils surrounding the waste pile may be saturated. If sliding 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. If relatively impervious, these soils 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 a considerable distance downslope. An unstable waste pile on a sloping foundation is potentially 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.

243. Frictional waste material such as blasted rock and permeable sand-gravel mixtures, may be placed by casting, end-dumping, or bulldozing over a face. Segregation of coarse particles will often occur as the materials cascade down the face so that a concentration of the coarsest material will be deposited at the base. As the leading edge of the fill advances, the coarse 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 pressure at the base of the pile.

244. If the waste material consists of sand, segregation of particles by size on the advancing face of the fill may not be significant, so that permeability at the base of the waste pile may not be significantly higher than the average of the pile as a whole. Drainage from within the pile may result in minor sloughing at the toe. If the waste material must be confined within stringent boundaries, it may be advisable to provide a toe drain at its perimeter. The gradation of material used to construct the toe drain should conform to gradation requirements for filters.

245. Where pervious, frictional waste material is 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 heights at their angle of repose.

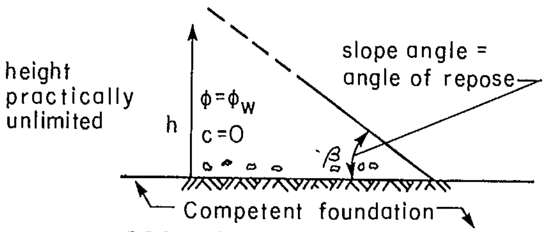
246. Where frictional waste is placed on a "weak foundation", the maximum permissible height of the waste pile and the perimeter slopes will be governed by the shear strength of the foundation. The maximum pile heights and the perimeter slopes must be determined by stability analyses.

247. The various approaches in determining the permissible heights and slopes of piles of frictional wastes are illustrated in Fig 37.

248. The permissible heights and perimeter slopes for waste piles containing cohesive materials should be determined by stability analyses. Methods for determining permissible combinations of height and slope for any required factor of safety are illustrated in Fig 38 and 39.

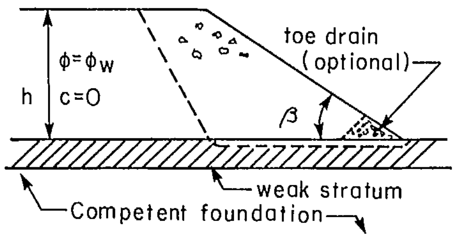
249. Where the cohesive material is placed on a competent foundation, the height of the pile and the perimeter slope will be governed by the strength of the material comprising the pile. Where the cohesive materials are placed on a weak foundation, the strength of the foundation will govern.

250. Both the friction angle and the cohesion of the material in a waste pile can be increased by compacting. The maximum shearing stresses within the pile occur beneath the perimeter slopes. Compaction of the material 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 material placed within the central portions of the pile. Extent of compacted perimeter zones and internal drainage systems are indicated in Fig 40. In some cases, it may pay to



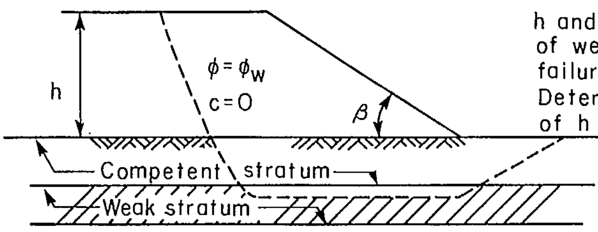
Pile can be constructed to any height provided $\beta \leq$ an angle of repose.

COARSE FRICTIONAL WASTE ON COMPETENT 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.

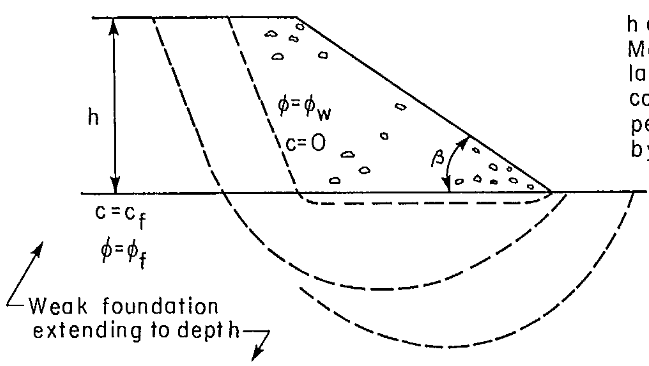
FRICTIONAL WASTE ON SHALLOW WEAK FOUNDATION



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.

Competent stratum

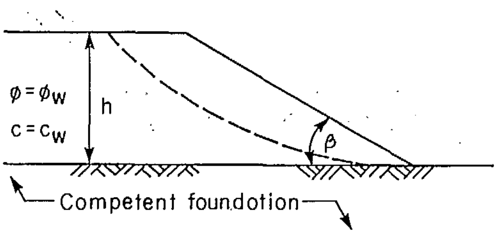
WEAK STRATUM AT SHALLOW DEPTH



h and β governed by stength 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.

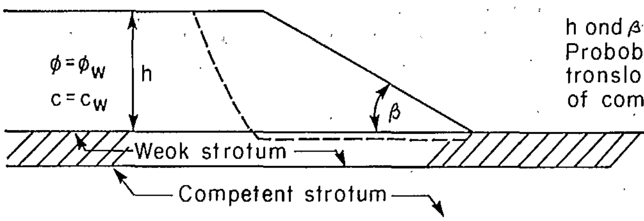
WEAK FOUNDATION EXTENDING TO CONSIDERABLE DEPTH

Fig 37 - Permissible slope height and angle for free draining cohesionless waste



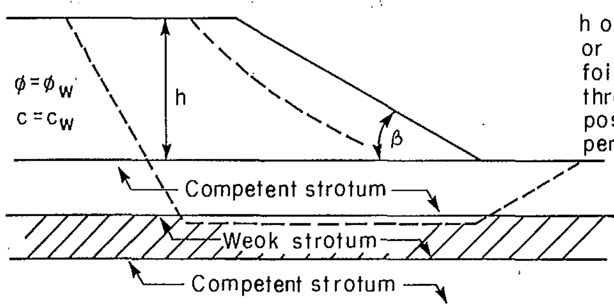
h and β governed by strength of waste material. Mode of failure may be either along circular or noncircular surface within waste material. Determine permissible combinations of h and β by stability analysis.

COMPETENT FOUNDATION



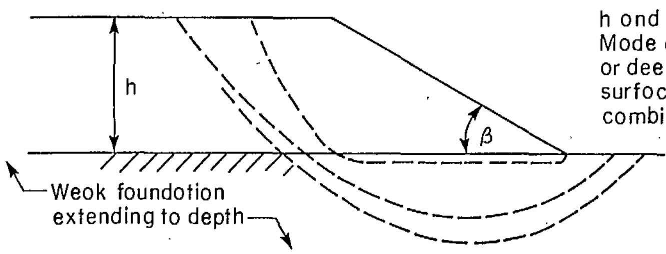
h and β governed by strength of weak stratum. Probable mode of failure is horizontal translation (spreading). Determine permissible of combinations of h and β by stability analysis.

SHALLOW WEAK FOUNDATION STRATUM



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 SRATUM AT SHALLOW 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.

WEAK FOUNDATION EXTENDING TO CONSIDERABLE DEPTH

Fig 38 - Permissible slope height and angle for cohesive waste

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

2. Choose a trial height, h, for the waste pile and a trial slope angle β (Fig 39(a)). Calculate the factor of safety FS, 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 FS vs β. This plot is illustrated on Fig 39(b).

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

7. From the graph of FS vs β, (Fig 39(b)) select values of h and β corresponding to the desired factor of safety, FS, plot these data in the form of a graph of h vs β, (corresponding to the required factor of safety, FS, (Fig 39(c)).

8. If the maximum volume of available storage within the available area is required, proceed with step 9.

9. Calculate the volume of available storage using combinations of h and β selected from the graph of Fig 39(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 (Fig 39(d)).

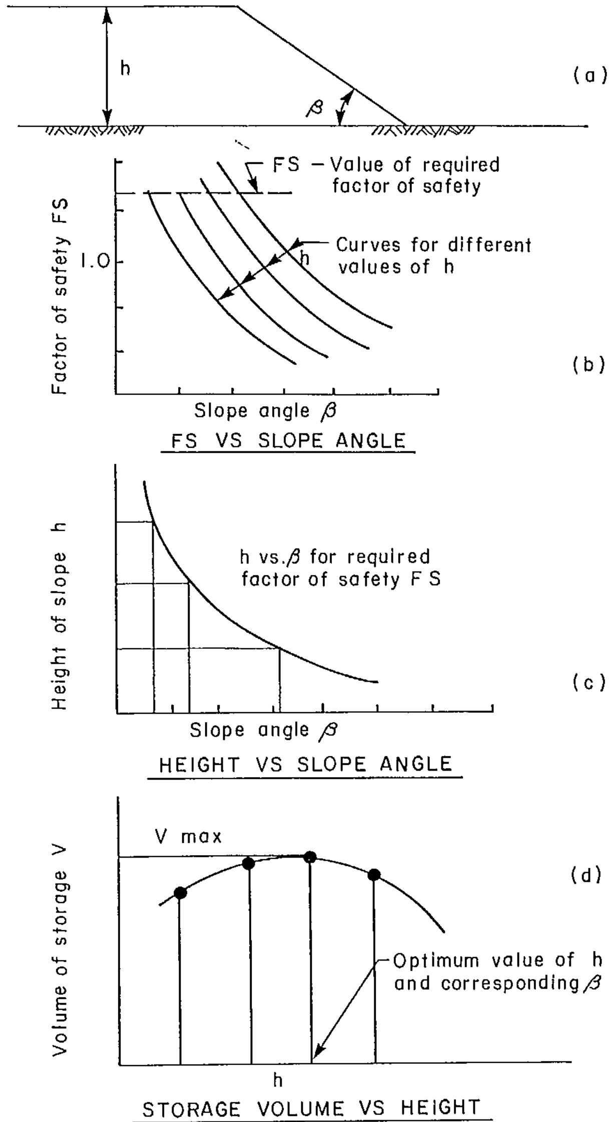


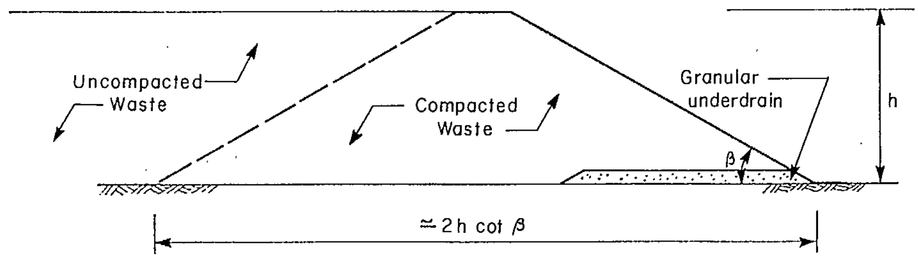
Fig 39 - Design procedure for waste piles

use compaction equipment and methods developed for highway and earth dam embankments. Stability studies for specific cases may indicate that the boundaries of the compacted zones may be shifted outward from those shown on the figure.

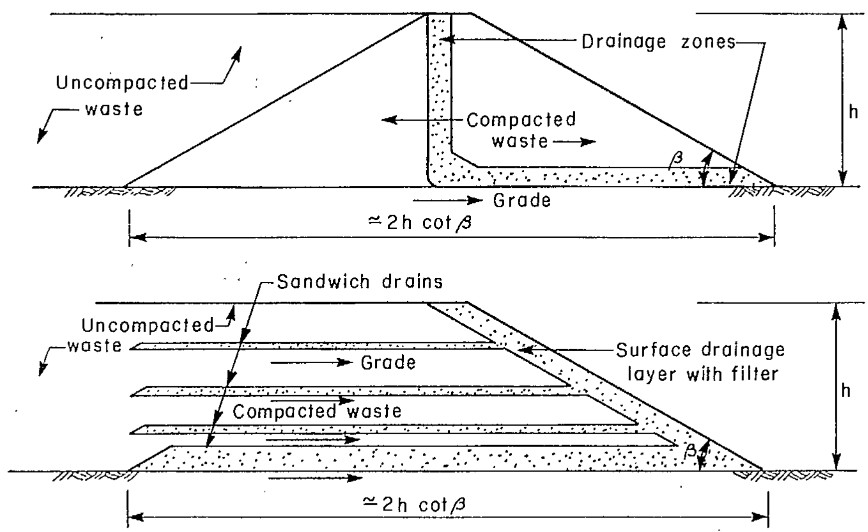
251. As compaction will also reduce permeability, it is desirable to place a drainage layer on the foundation beneath the perimeter of the pile. This will tend to lower the phreatic surface beneath the slope, thereby increasing stability of the pile.

252. When hauling equipment is used to place waste materials within a pile, this traffic may produce zones of lower than average permeability within the pile. These zones will be 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.

253. Where the waste disposal area is limited, it may be necessary to incorporate horizontal drainage layers within the pile during con-



Compacted perimeter zone serves as a dyke to retain uncompacted material in the central portions of the tailings basin.



Granular drainage zones maintain drained conditions within the perimeter of the pile. Perimeter zone acts as a dyke to retain the tailings slimes within the central portion of the tailings basin.

Fig 40 - Perimeter zones for waste piles

struction, or to provide inclined drainage zones at the perimeter of the pile as indicated in Fig 40. The drainage zones will reduce undesirable pore water pressures within the pile.

254. To prevent surface water from entering waste piles during periods of rainfall and snow melt, exposed surfaces should be adequately drained.

255. One of the most difficult problems in the design of piles for storing wastes which degrade by weathering, softening or chemical change, is determining the shear strength parameters to be used in stability analyses. The 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. In other respects, design procedures for piles containing degradable waste are similar to those described for piles of cohesive wastes.

256. Usually, the degree to which degradable waste materials lose strength can be reduced by sealing the surface of the pile, reducing entry of

surface water and air circulation; however, minimum expected strength parameters must be used for design purposes.

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

258. Burning may cause the formation of voids within a waste pile resulting in local collapse and adverse changes in stability conditions. Alternatively, burning may cause an increase in the shear strength of the waste and, where temperatures are sufficiently high, fusing may occur. 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.

259. The chief hazard of burning coal wastes is the generation of noxious gases. These include carbon monoxide, carbon dioxide, sulphur dioxide and occasionally hydrogen sulphide. The rate of production of the gases may be accelerated by disturbing a burning pile, for example by excavating spoil or by re-shaping.

260. 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 are readily detectable by smell, taste and irritation long before the lethal levels are reached.

261. Steam 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% 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 in a waste pile.

262. When working on a burning waste pile every effort should be made to prevent 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.

STABILITY ANALYSES

263. The factor of safety, "FS", for a slope is usually defined as the ratio of available shear strength to shear stress on the critical failure surface. 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 to allow for differences between the pore water pressures, shear strength parameters and strains assumed in design and those that may actually exist within the slope.

264. Stability analysis is a procedure of successive trials (48-55). A potential sliding surface is chosen and the factor of safety against sliding along that surface is determined. Different surfaces are selected and the analysis is repeated until the surface having the lowest factor of safety, known as the critical surface, is found. The calculated factor of safety against sliding along the critical surface is the indicated factor of safety for the slope.

265. The critical surface may be located completely within an embankment; it may lie totally outside the embankment if it passes through weak material retained by the embankment and weak foundation soils, or it may be located at any position between these limits.

266. With the exception of a few special cases, stability calculations to determine the factor of safety should be based on effective stress analysis. The determination of effective stress requires a knowledge of the pore water pressure within the embankment. For a fully consolidated fill subjected to steady seepage conditions, the pore water pressure can be determined from a flow net, thereby permitting the calculation of effective stress. When the fill or the foundation are consolidating under the weight of the overlying material, the pore water pressures must be

measured or estimated using consolidation theory. In cases where the pore pressure is critical in the stability analysis, piezometers should be installed within the embankment to measure the field conditions. If the pore water pressures are significantly higher than those used in the design, it will be necessary to recheck the stability analysis and modify the design section to maintain the desired factor of safety.

267. In the design of a tailings embankment, the stability of the downstream slope is usually the principal concern. However, the stability of the upstream slope should also be checked in special cases, particularly if there is a considerable depth of water against the upstream face of the dam. In this case the dam should be designed as a water dam, and a drawdown stability analysis should be made. (Refer to Fig 30 for drawdown flow net) A drawdown condition may also exist if tailings are recovered from the pond.

268. The most commonly assumed critical surface used in stability analyses for embankments is a cylinder whose axis is oriented parallel to the strike of the slope. On a two-dimensional cross section, the cylindrical surface is represented by a circular arc. Observations of full-scale slides in the field show that some sliding surfaces are nearly circular. However, many documented examples are available which show that the shape of the surface is often non-circular. When sliding occurs, differential shearing takes place along the surface on which the factor of safety is lowest. The most practical procedure is to base the design on the critical circular surface unless definite knowledge exists of critically oriented weak bands.

269. The true surface of sliding will deviate from the commonly assumed circular surface if the potential sliding surface passes through zones having different shear strength or different pore water pressure. Methods of stability analyses applicable to circular and non-circular sliding surfaces are included in Supplements 5-1 and 5-2. Traditional analyses are provided in the descriptions given below.

270. With the exception of a few special cases, the methods used to calculate the factor of safety for any trial sliding surface should account for changes in the shear strength and varying pore water pressure along the potential sliding 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 this method, a trial surface is chosen and the potential sliding mass is divided into a number of vertical slices. Each slice is acted on by its own weight which produces shearing and normal forces on its vertical boundaries, and shearing and normal forces along its base.

271. In the method of infinite slices, a circular trial sliding surface is selected, and stability of the potential sliding mass is considered as a whole rather than stability of each individual slice. As 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 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 sliding 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 in Fig 41.

272. Factors of safety determined by the method of infinite slices will err on the conservative side.

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

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 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 u (pore water pressure) along trial failure surface. Determine values of u/cos i. A graphical method of determining u/cos i is indicated by Fig 41(c).

8. Plot values of u/cos i below the total normal stress curve. The area of the hachured portion of the normal stress distribution curve equals the total effective normal stress on the trial failure surface.

9. The factor of safety for the trial surface selected can be determined graphically as indicated on Fig 41(f).

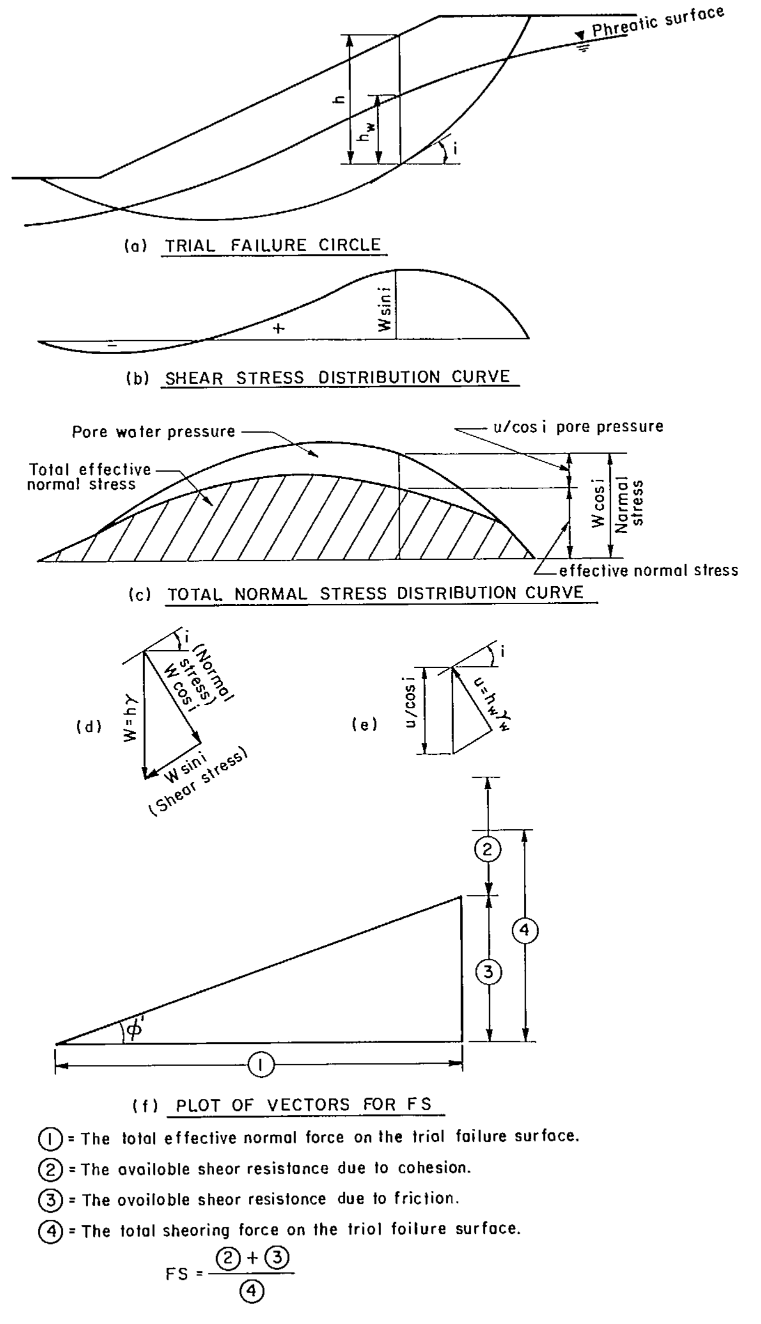


Fig 41 - Stability analysis by the method of slices depend on the stress-deformation characteristics of the material comprising the sliding mass and cannot be evaluated exactly. However, the summation of the forces acting on the vertical boundaries of the slices is zero, and these forces may be neglected without serious reduction in accuracy. Neglecting the forces acting on the sides of the slices, the factor of safety is expressed by the equation:

$$FS = \frac{\sum(c'b \sec\alpha + N'\tan\phi')}{\sum W_o \sin\alpha} \qquad \text{eq 8}$$

where FS is the factor of safety, W_o is the weight of material within the slice, c' is the effective cohesion for the soil, b is the width of the slice, α is the angle of inclination at the centre of the base of the slice, N' is the effective normal force on the base of the slice, and ϕ' is

PROCEDURE

1. Select a trial circular arc failure surface.
2. Divide the mass above the trial failure surface into approximately 10 vertical slices.
3. Determine the weight (W_o) of each slice.
4. Determine α, the angle of inclination of the trial failure surface at the base of each slice.
5. Compute W_o Sin α, the overturning element (1 on the table).
6. 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).
7. Calculate pore pressure and neutral force of the base of each slice. The pore pressure = the weight of water times the average piezometric head above the base of the slice ($u = h_s\gamma_w$), and the neutral force = the pore pressure times the width of the slice (bu).
8. Compute the frictional element (W_o - bu) tan ϕ', where ϕ' is the effective angle of internal friction along the base of the slice.
9. Assume a factor of safety FS, and determine m_a for each slice using the graph. Calculate a new factor of safety (5) = [(2) + (3)]/m_a. The first approximation of the factor of safety then is given by

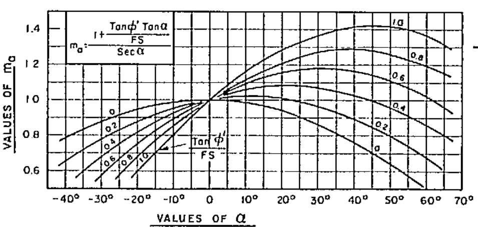
   $\sum(5)/\sum(1)$
10. Using the factor of safety determined by step (9) above, repeat step (9) to obtain the second approximation of the factor of safety.
11. Repeat (10) until a factor of safety FS is obtained which is equal to the value of F used to determine m_a.

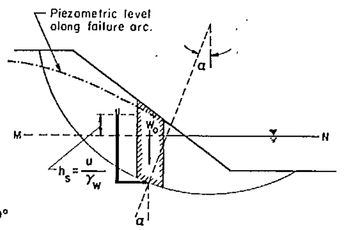
   $FS = \frac{1}{\sum(W_o \sin_\alpha)}\sum[(c'b + (W_o - bu)\tan\phi')\frac{1}{m_a}]$

   $= \sum(5)/\sum(1)$

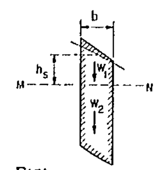
SLICE NO	OVERTURNING ELEMENTS (1)							COHESIVE ELEMENTS (2)			FRICTIONAL ELEMENTS (3)						(4)	1st. Trial FS = Tan ϕ'/FS =		2nd. Trial FS = Tan ϕ'/FS =	
	W OVERTURNING = W_o				α	SIN α	W_oSINα	b	c'	c'b	h_s	$u = \gamma_w h_s$	bu	W_o-bu	Tan ϕ'	(W_o-bu)Tan ϕ'	(2)+(3)	m_a	(5) = $\frac{(4)}{m_a}$	m_a	(5) = $\frac{(4)}{m_a}$
	γ	AREA																			

$\Sigma(1)$ $\Sigma(5)$





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)

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

TRIAL (1)

$FS = \frac{\Sigma(5)}{\Sigma(1)}$

= ______

=

TRIAL (2)

$FS = \frac{\Sigma(5)}{\Sigma(1)}$

= ______

=

FINAL FS =

TABULATED SOIL PROPERTIES			
γ			
γ_b			
ϕ'			
c'			

Fig 42 - Stability analysis by the Simplified Bishop Method the effective angle of internal friction.

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

$$N' = \frac{[W_o - b \sec \alpha \ (u \cos \alpha + \frac{c'}{FS} \sin \alpha)]}{[\cos \alpha + \frac{\tan \phi' \ \sin\alpha}{FS}]} \qquad \text{eq 9}$$

where u = pore pressure. Therefore,

$$FS = \frac{1}{\sum W_o \sin\alpha} \sum \left[\frac{(c'b + (W_o - ub) \tan\phi') \sec\alpha}{1 + \frac{\tan \phi' \ \tan\alpha}{FS}}\right] \qquad \text{eq 10}$$

275. Since FS appears on both sides, the equation must be solved by successive approximations. The procedure for calculating the stability for a single trial surface is indicated in Fig 42.

276. 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 1% to 1.5%.

277. Where the potential sliding 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 (49) have presented a method for calculating the factor of safety for non-circular surfaces of sliding. This takes into account the shearing 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 with the Morgenstern-Price analysis, use of a computer is necessary. 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.

278. 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.

279. If computing facilities are not available, 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 in Fig 42 may be used. Where the configuration of the trial sliding surface conforms approximately to two or more intersecting tangents, the factor of safety may be determined by using the wedge analysis illustrated in Fig 36.

280. 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 sliding surface is chosen that passes through the soft foundation layer and the components of all of the forces acting on the potential sliding 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 in Fig 43.

281. Where an embankment or a slope is subjected to an earthquake, the ground motions produce stress fluctuation so that the dynamic shearing stresses are alternately higher and lower than the static shearing stresses.

282. The general distribution of 100-year earthquake accelerations for Eastern and Western Canada is shown in Fig 44. The following design features may be included to decrease the risk of damage by earthquakes: (a) increase normal freeboard, (b) increase width of crest, (c) reduce the zone of saturation in dam with an improved seal and better internal drainage, (d) compact embankment fill to achieve relative densities of 60% or greater, (e) mass of the embankment should be large enough to provide the required factor of safety against horizontal sliding along its base

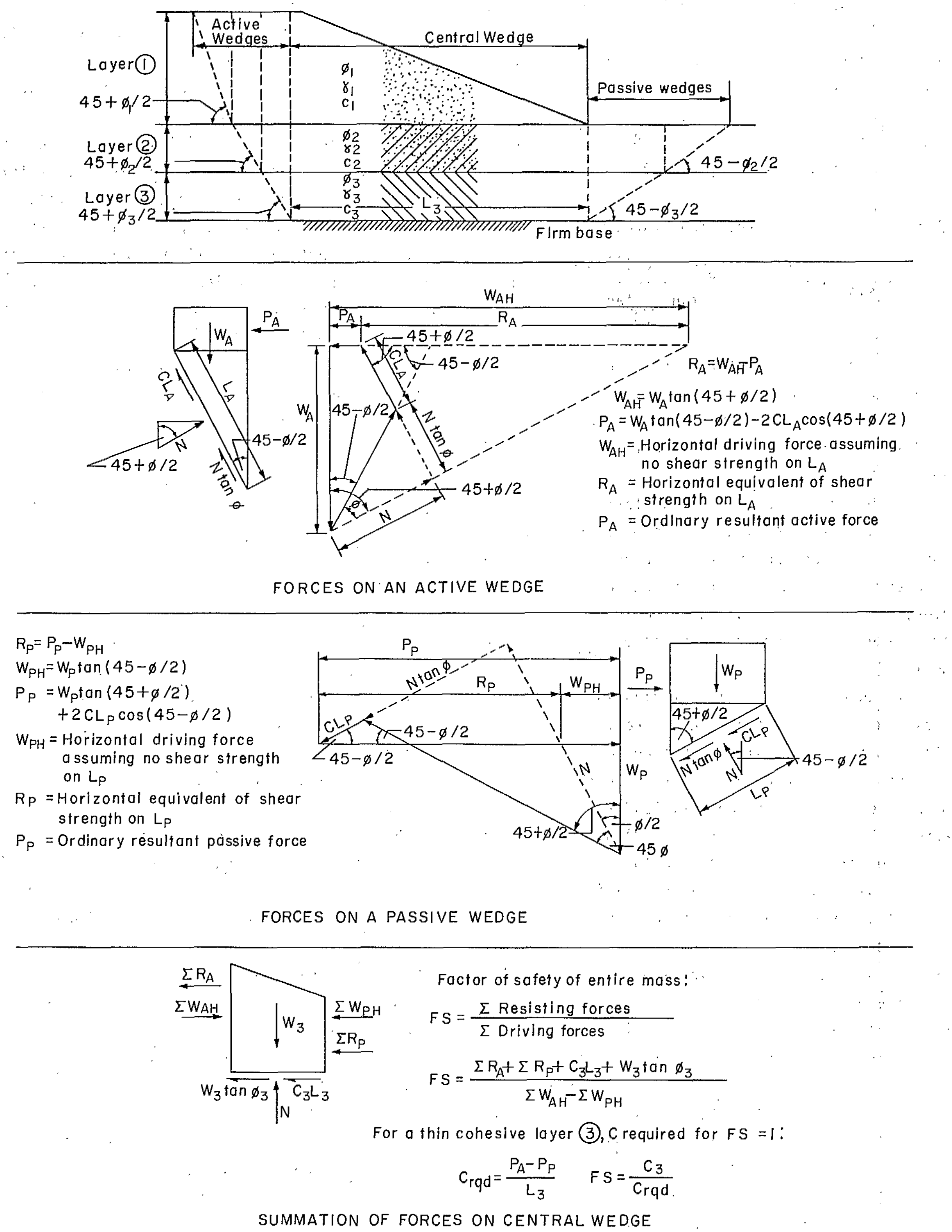


Fig 43 - Stability analysis for horizontal translation

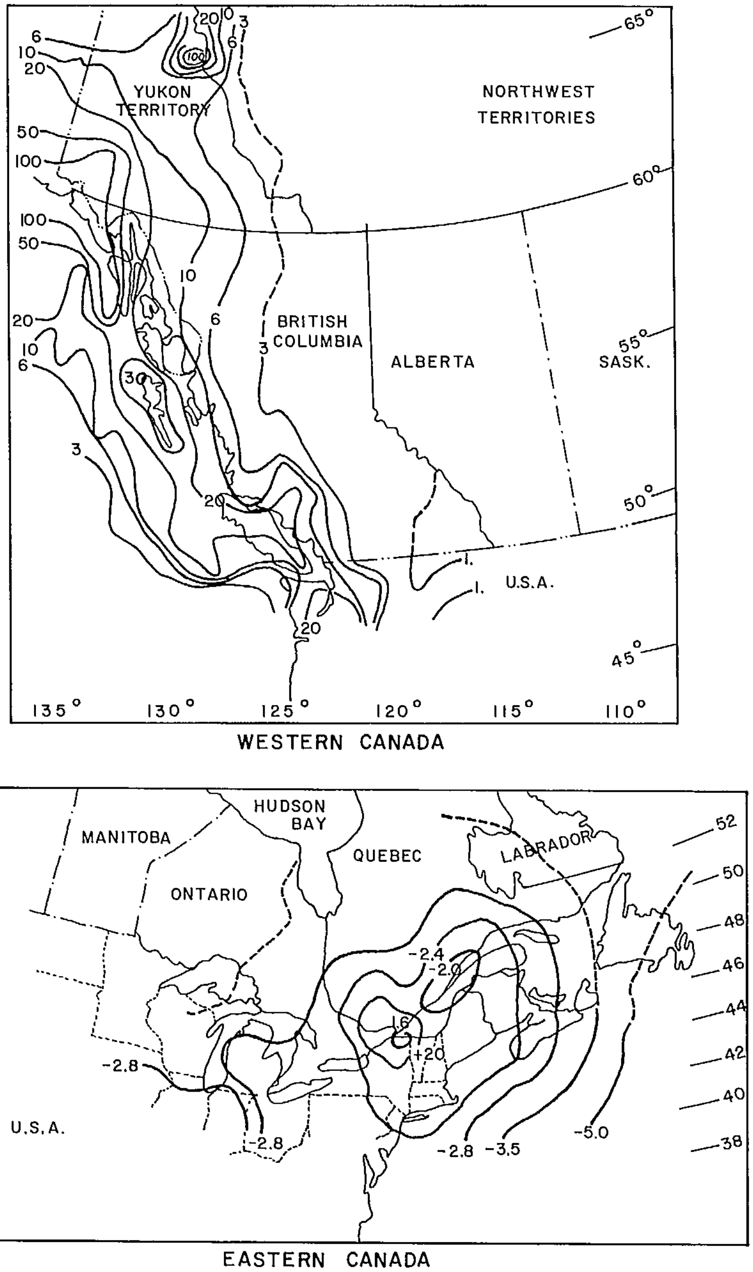


Fig 44 - One hundred-year return period accelerations as a percentage of gravity (after Milne and Davenport, 1969)

in the event that shear strength of the saturated tailings slimes stored behind the embankment is reduced to zero, and (f) decrease slope angles.

283. In instances where the dam is constructed without an engineered seal, the pool should be maintained as far away as practical from the upstream slope to minimize the zone of saturation. In general, spigotting procedures should not be used to construct high tailings embankments of over 25 ft (8 m) in areas of seismic activity.

284. Where stability is to be ensured by decreasing slope angles, 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 earthquake is used.

285. Strains which may occur during an earthquake can result in distortion of the embankment as shown in Fig 2. Procedures for estimating the magnitude of these strains have been proposed by a number of researchers (2, 4). 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 such forces should be selected on the basis of the probability of earthquakes of various magnitudes occurring in the region. This information can be obtained from the Earth Physics Branch, Department of Energy, Mines and Resources, Victoria, B.C., and Ottawa, Ont.

286. 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 when subject to shock loads or seismic activity.

287. Tailings deposited by sluicing or spigotting remain loose and, if saturated, may be susceptible to liquefaction. When the upstream method is used for constructing a tailings embankment, as its height increases, the critical failure surface is located at progressively increasing distances from the downstream slope of the embankment. For a high embankment, a large proportion of the critical failure surface may be located within the sedimented tailings.

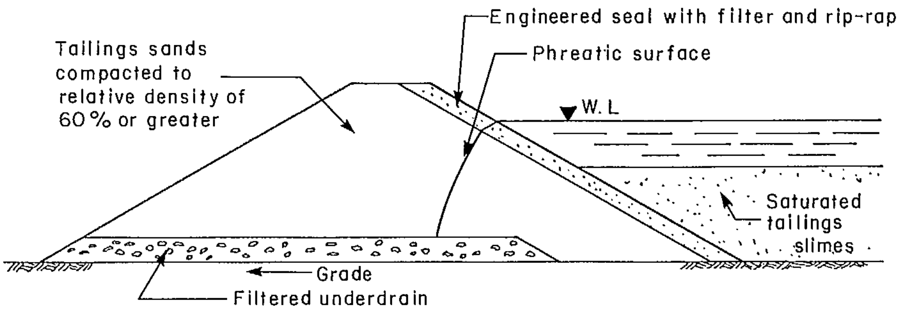
288. For any given saturated sand, the danger of liquefaction as a result of cyclic loading or of progressive unidirectional shearing strains is governed by: (a) density of the sand - the lower the density, the more easily liquefaction will occur, (b) confining pressure acting on the sand - the lower the confining pressure, the more easily liquefaction will develop, (c) magnitude of the cyclic stress or strain - the larger the stress or strain, the lower the number of cycles required to induce liquefaction, (d) 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, and (e) in the case of unidirectional strain - the higher the rate and magnitude of the strain, the greater the likelihood of liquefaction.

289. Clearly, 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 away as practicable from the embankment or by compacting the fill materials during construction, the density, saturation, and confining pressures can be controlled so as to reduce the likelihood of liquefaction.

290. If the tailings embankment is constructed of fine sands, compaction of these sands will increase their density and reduce their susceptibility to liquefaction (56). Compaction to relative densities of 60% or greater provides reasonable protection.

291. Non-saturated sands do not liquefy. Hence, if the phreatic surface can be maintained at a position well below the base 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. Effective underdrainage systems which substantially reduce the level of the phreatic surface increase an embankment's resistance to failure.

292. In summary, provided the materials comprising the embankment have a relative density of 60% or greater, or provided 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



Design feature to minimize probability of liquefaction

1. Increase the normal freeboard
2. Increase the width of the crest.
3. Reduce the zone of saturation in the dam with a positive seal and better internal drainage.
4. Compact embankment fill to achieve in situ relative densities of 60 per cent or greater.
5. The mass of the embankment should be large enough to provide the required factor of safety against horizontal sliding along its base in the event that the shear strength of the saturated tailings slimes stored behind the embankment is reduced to zero.
6. Decrease the slope angles.

Note: In instances where the dam is constructed without an engineered seal (by spigotting hydraulic tailings), the pool should be maintained as far as practical from the upstream slope in order to minimize the zone of saturation. (High dams should not be spigotted in seismic areas)

Fig 45 - Design safeguards against liquefaction

embankment should be sufficient to provide the required factor of safety against horizontal displacement along its base, assuming 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 in Fig 45.

293. 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 assumes that it will maintain strains within tolerable limits.

294. In choosing the factor of safety, the possible consequences of instability, 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 the most adverse combinations of conditions which are realistic for the factors pertinent to stability, so that their significance can be assessed.

295. Where instability could be dangerous, 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 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 water pressures within the embankment and the foundation if these presssures are significant in determining their stability. Pore water pressures significantly higher than those assumed in design may necessitate a modification of the design.

296. The factors of safety listed below are suggested minimum values for design purposes. The values presuppose that the stability analysis has been sufficiently comprehensive 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 in the embankment.

297. Suggested factors of safety for the downstream slope for long-term steady seepage conditions are given in Table 12.

298. 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 as listed.

299. Where the number of field and laboratory tests, 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.

300. Where a waste pile is constructed on a steeply sloping foundation, the suggested minimum factors of safety should be increased by at least 10%.

Table 12: Minimum factors of safety for the downstream slope

Assumptions	I*	II**
Using peak shear strength parameters	1.5	1.3
Using residual shear strength parameters	1.3	1.2
Including the loading for a 100-year earthquake	1.2	1.1
For horizontal sliding on the base of embankments retaining tailings in earthquake areas assuming shear strength of tailings behind the dam reduced to zero	1.3	1.3

*I - where it is anticipated that severe damage would occur as a result of an embankment failure.

**II - where it is anticipated that severe damage would not occur as a result of an embankment failure.

SETTLEMENT ANALYSES

301. If the foundations beneath an embankment consist of dense glacial till, dense sand and gravel or rock, vertical deformations under the weight of the embankment will be largely elastic. Settlement will occur as the loads are applied but their magnitude will be so small as to be insignificant with respect to performance of the embankment.

302. If, however, the foundations beneath the embankment contain layers or strata of normally consolidated fine-grained sediments, such as clays and silts, significant vertical deflection of the foundation may occur under the weight of the embankment fill as the fine-grained sediments consolidate. The magnitude of 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 foundation settling occurs will depend on the magnitude of the change in vertical stress, ie, the rate of fill construction, on the permeability of the compressible material, and on the drainage characteristics of the foundation.

303. 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 settling records at other sites underlain by compressible strata having similar water contents and index properties are useful in assessing the probable range.

304. 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 settling occurs is controlled by minute geological details which may not be detected even by carefully conducted foundation investigation programs. Estimates of the magnitude and rate of settling should be made to the extent possible with the best available data. However, these predictions should be checked by instrumentation, measuring the rate and magnitude of the settling that actually occurs. The initial 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.

305. Differential foundation settlements could produce cracking of an impervious core or cracking of the embankment which could lead to internal erosion. Differential settlements may also cause damage to decant lines installed in or below the structure. In assessing the effect foundation settling may have, it should be assumed that localized areas will be subjected to differential settling at least two times greater than indicated by the contours of predicted settlements.

306. Installing pipes within or beneath an embankment should be avoided at locations where significant foundation settling is expected and decant pipes through a tailings embankment should be located as far away as practical from the areas of maximum anticipated settling.

CONSTRUCTION AND OPERATION

TAILINGS EMBANKMENT CONSTRUCTION

307. The vast majority of mine concentrators use a wet process to separate the valuable minerals and the tailings material is in the form of a slurry for convenience of disposal. Therefore, when tailings are used to construct a tailings dam, it is frequently constructed by hydraulic means using one of two common methods - by hydrocycloning and by spigotting - to separate the relatively coarse-grained sand which is useful for building purposes from the fine-grained slurry.

308. The three common construction methods illustrated in Fig 26 are the downstream method, the upstream method, and the fixed centreline method.

309. To regulate expenditures for tailings disposal, embankments are frequently raised in stages as required to maintain the desirable freeboard of the dam above the pool surface. The most practical method of raising the embankment in stages will be governed by such considerations as: the design of the dam, materials available for construction, methods and distribution of tailings sand production, and volume of sand required in the embankment cross section.

310. In the downstream method the crest of the embankment is raised in successive stages by placing tailings sand on the downstream side of the starter dam. The starter dam forms the upstream toe of the ultimate dam and should be impervious to restrict seepage. This method provides major structural advantages by facilitating installation of internal drainage at the base of the dam beneath successive stages of construction, enabling the total structural section to be built with competent material, and permitting an engineered upstream seal to be included in the embankment (Fig 26).

311. In the downstream method, the total embankment section lies outside the boundaries of the sedimented tailings slimes. Material incorporated in subsequent stages of the embankment

may consist of the coarse fraction of the tailings separated by cycloning, waste rock from the mining operation, or natural soils from nearby borrow pits. When cyclones are used, the overflow, or slimes product, is discharged beyond the upstream toe of the embankment. The downstream method of construction permits controlled placement of the embankment materials and compaction can be included when it is desirable to increase the shear strength of the construction materials. The inclusion of internal drainage and an upstream seal will result in a low phreatic surface within the embankment. The downstream method is an inherently safer procedure than the upstream method of construction.

312. Hydrocyclones, or cyclones, can be used to separate sands from the slimes. A series of cyclones can be placed along the crest of the embankment as shown in Fig 46, or a group of cyclones can be mounted in parallel as a mobile unit which travels parallel to the longitudinal axis of the dam. It is possible to construct an embankment or a stage of the embankment to any desirable height in a single lift, using a mobile cyclone unit without the assistance of other machinery. Also, the tailings header can be laid along successive berms with a series of cyclones mounted on raised movable platforms or the tailings header can be mounted on trestles or towers with lateral takeoffs for each cyclone to construct the next stage.

313. It is necessary to elevate the cyclones to provide temporary storage for the cyclone underflow sand prior to spreading and compacting. The overflow from the cyclones is discharged upstream into the slimes basin (Fig 46 and 47).

314. By changing the design or operating variables, the cyclones can provide almost any desirable point of cut-off in the underflow sand product. As long as the conditions affecting the supply of tailings slurry to the cyclones remain relatively constant, the characteristics of the underflow product will remain constant for any particular setting. With a rope type product, the per cent weight recovery of free draining sands in the underflow will approximate 25-35% when the tailings supply contains 50-60% minus 200 mesh. Cyclones are usually used for the downstream method or the fixed centreline method. The limitation on the application of cyclones for building tailings dams is often dictated by freezing weather or the limited amount of sands in the tailings slurry. Generally, it is not practical to construct a tailings dam by the downstream or centreline methods if the tailings contain more than 75% slimes. When the tailings sands are used for backfill underground and the remaining tailings directed to the tailings basin, it is not practical to use cyclones for dam building. Cycloned tailings sands are pervious and therefore it is essential to provide an upstream impervious seal to restrict the flow of seepage water from the tailings basin.

315. In the upstream method of construction, the crest of the embankment is raised in stages by placing tailings sand in successive dykes above the upstream side of the starter dam, or the upstream side of a preceding dyke. The successive stages form a relatively thin structural shell on the downstream slope, and it is generally necessary to improve stability of the dam by including berms at the stages to flatten the overall effective slope. The initial starter dam forms the downstream toe of the ultimate dam. It must be pervious to prevent the buildup of pore water pressures which may permit more seepage than desired. This problem can be reduced by providing a low starter dam with a wide base of pervious material, sealing the upstream slope with a limited amount of impervious material. The second stage is built above the crest of the wide starter dam (Fig 26).

316. Spigots are frequently employed in the upstream method of construction. As the tailings slurry is discharged from a series of spigots along the crest of the dam, the slurry meanders in a random manner depositing sands and slimes in a series of loose, discontinuous, horizontal stratifications. To provide the required freeboard on the crest of the dam, it is necessary to reclaim the tailings adjacent to the crest with mechanical equipment such as draglines or dozers. It is difficult to provide a competent seal above the base of the settling pool and the line of sat-

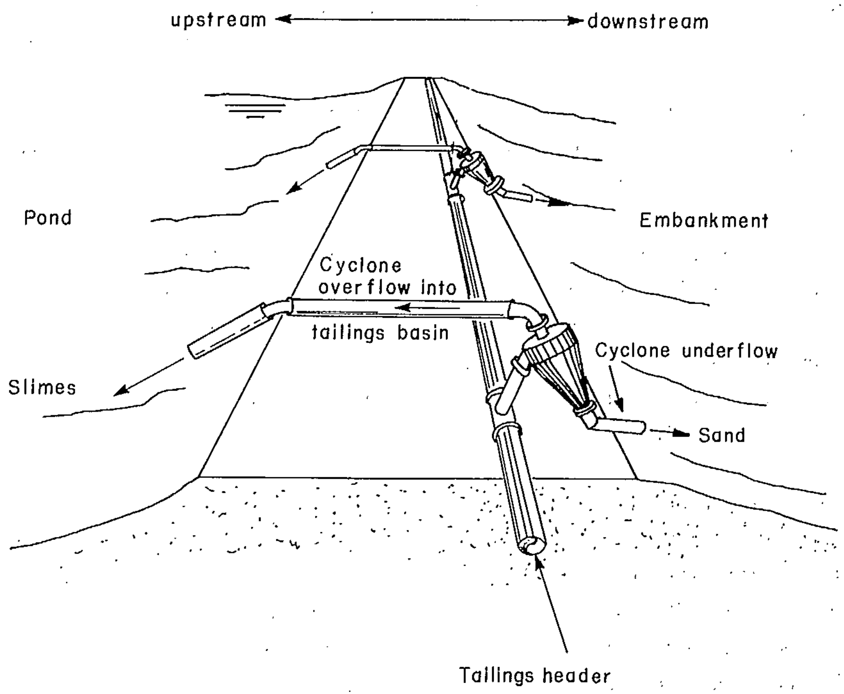


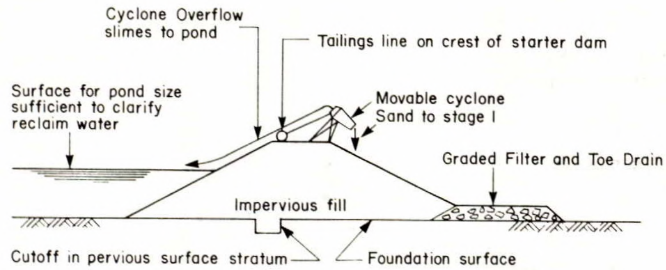
Fig 46 - Typical cycloning arrangement

uration within the embankment varies as the elevation of the pool surface is increased. A major portion of the structural section of the embankment is composed of loose material with a relatively high phreatic surface and low shear strength. Therefore, to provide an adequate factor of safety for the embankment, the downstream slope must have a relatively flat angle or berms must be included to provide a desirable overall effective slope. Owing to the wide variation in permeability and the possibility of high porewater pressures, low relative density and low shear strength, the upstream method of construction may be unsuitable for areas subject to intense seismic activity.

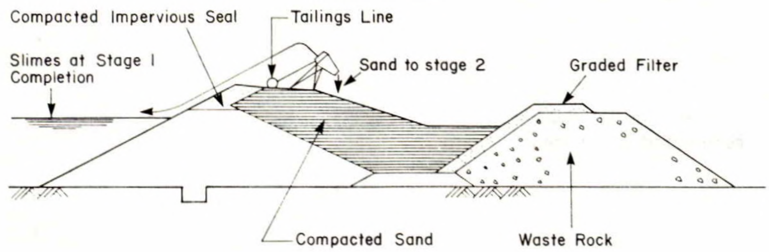
317. The operating procedures are relatively simple. The limitations of spigotting for dam construction are that it must usually be stopped during severe winter weather, it is not particularly favourable for earthquake zones, and the method is usually restricted to upstream stage construction. Where it is desirable to minimize the volume of contaminated seepage, other methods of construction should be considered which facilitate the design and installation of an impervious seal.

318. In the fixed centreline method, the centreline of the embankment is maintained in the same position with each successive stage of construction up to the ultimate height of the embankment.

319. Sluices can be formed by diking the edges of the embankment crest, as shown in Fig 48, to form a channel sloping gently downwards. 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 bull-



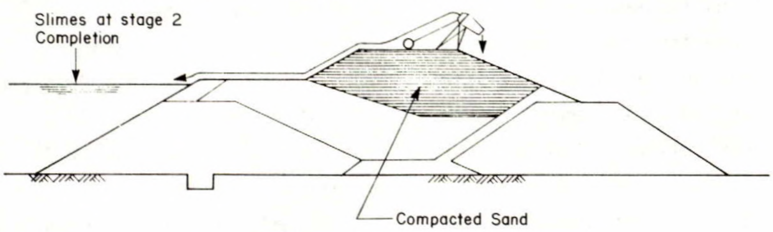
STARTER DAM



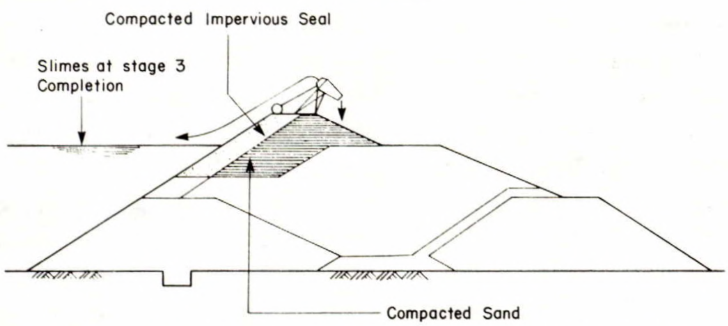
STAGE I

Notes on Basis of Design

1. Raw tailings fines content (-200 mesh by weight) = 70%
2. Sand fines content after two stage cycloning (under flow) = 11%
3. Sand yield as percentage of raw tailings solids = 34%
4. Sand dry density in embankment = 100 lbs./cu.ft. (1600 kg/m^3)
5. Slimes dry density in pond = 85 lbs./cu.ft. average (1360 kg/m^3)
6. Impervious fill and filter material obtained from borrow pits; rock waste obtained from ore processing plant.
7. Volume of waste rock 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 51.
8. Cyclones operate 60% of time following plant startup.



STAGE 2



Bulk tailings to be discharged upstream after completing construction of embankment

STAGE 3

Note: Decant system or reclaim water system are not shown on the sketches above.

Fig 47 - Tailings embankment construction stages

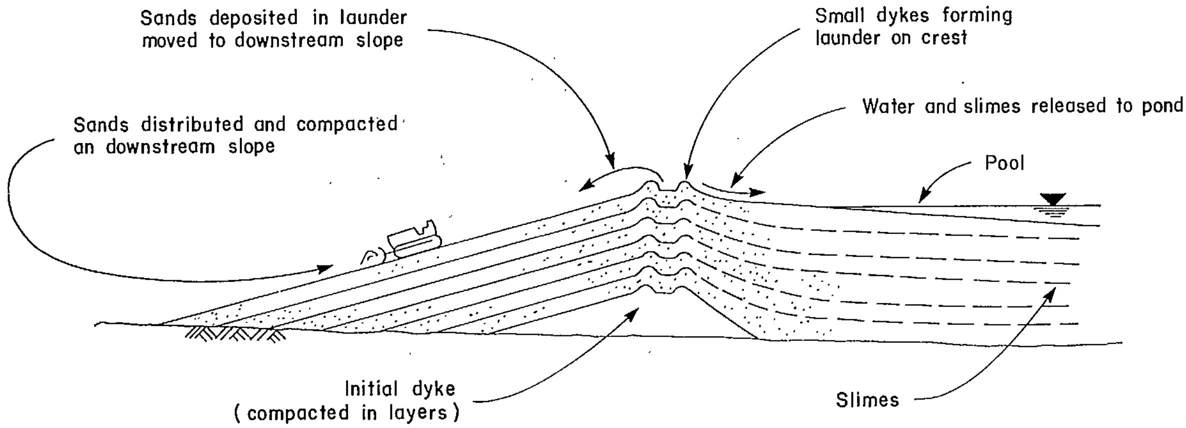


Fig 48 - Typical sluicing arrangement (35)

dozers or draglines and are spread and compacted on the shoulders of the embankment. A variation of the channel sluicing method is 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 into the pond.

320. The sluicing method of construction requires flat fill slopes for operating 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 section, this method is only suitable when the tailings contain high percentages of medium to coarse sand particles.

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

322. Whatever the method of sand separation, when the sands deposited on the embankment have to be spread and compacted, they must be relatively free-draining. As deposited, they will 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% by weight. The equivalent soil water content, expressed as a percentage of water to solids by weight, is 33-43%. By comparison, the optimum water content for compaction is usually in the range of 10-20%.

323. If the sands deposited on the embankment have a relatively high content of slimes, they will not only be slow in draining to optimum water content, but they will be difficult to handle. With a high water content and a high slimes content, as is frequently the case with spigots, they will flow on very flat slopes, even spreading beyond the designed 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 being deposited.

324. The most suitable fines content for compacted sands will to some extent depend on the mineralogical nature of the slimes. The reduction in fines content of the tailings that can be

accomplished on the embankment will depend on the method of sand separation and on the gradation of the tailings. For example, with tailings containing 70-80% by weight of fines (-200 mesh), even two-stage cycloning may not reduce the fines content to less than 10-15%. A lower fines content would improve the drainage and handling characteristics of the sands on the embankment. However, it would also increase permeability which would increase the seepage rates through the embankment unless an upstream seal is provided.

325. The sand characteristics from the cyclone underflow are relatively constant for a particular set of operating conditions, whereas the characteristics of the spigot product vary widely from one location to another depending on velocity of the meandering discharge and location of the sedimented particles within the stream. An embankment which has been constructed by spigotting usually consists of a series of horizontal discontinuous stratifications of sands and slimes.

326. In general, the most suitable gradation for embankment sand should be studied during the design phase by making laboratory tests on samples of tailings. Trial batches of tailings should be produced for this purpose after which practicable methods of producing a suitable sand at the site can be determined. Discussions with equipment manufacturers will be necessary to decide on the type, design, size, and number of cyclones required to provide a suitable underflow sand product. A knowledge of the range of permeability of the sand from laboratory tests will enable suitable seepage control provisions to be included in the design.

327. Because of variations which occur in the sand characteristics, the properties should be checked by sampling and testing during the early stages of construction. The sand separation and placement procedures should also be monitored frequently during construction.

328. 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 tailings. Variations in sand yield have a dual effect - a decrease in yield will retard the rise of the embankment crest and increase rise of the tailings in the pond. Particularly with finely ground tailings, the yield may be too low to maintain adequate freeboard on the crest of the dam. In such cases it may be necessary to supplement the sand recovered from the tailings with borrow material to provide sufficient embankment fill, as shown in Fig 26.

STARTER DAMS

329. The first step in constructing any dam is to prepare the foundation for the proposed embankment. In some instances it may be desirable to build a cofferdam. In the case of a tailings dam, a starter dam will usually serve the dual purpose of cofferdam and provide sufficient storage capacity to retain the tailings until the first stage of the embankment is complete. The required storage capacity will dictate the minimum height of the starter dam to schedule closure of the first stage construction.

330. It is important in downstream stage construction to provide a relatively impervious starter dam when it is located at the upstream toe of the ultimate embankment section and to provide a pervious downstream zone for the starter dam when upstream stage construction is used.

331. The planning, design and construction of the effluent or reclaim water system from the tailings basin must be consistent with the schedule for the starter dam and the first stage construction.

332. It should be noted that height of the starter dam will depend on: (a) the area and volume of the tailings basin, (b) the volume and schedule of tailings disposal, (c) the necessary depth of water in the pool to maintain a clear effluent, (d) the height, details and schedule of stage one construction, and (e) details and scheduling of the effluent or reclaim system.

333. The embankment construction schedule, illustrated in Fig 49, is related to the rate of rise of the pond. The elevation of the crest of the starter dam, the volume of rock fill or borrow material in the embankment, and the stage boundaries are established so that successive berms are ready for the tailings pipelines before the tailings in the basin rise to the level of the

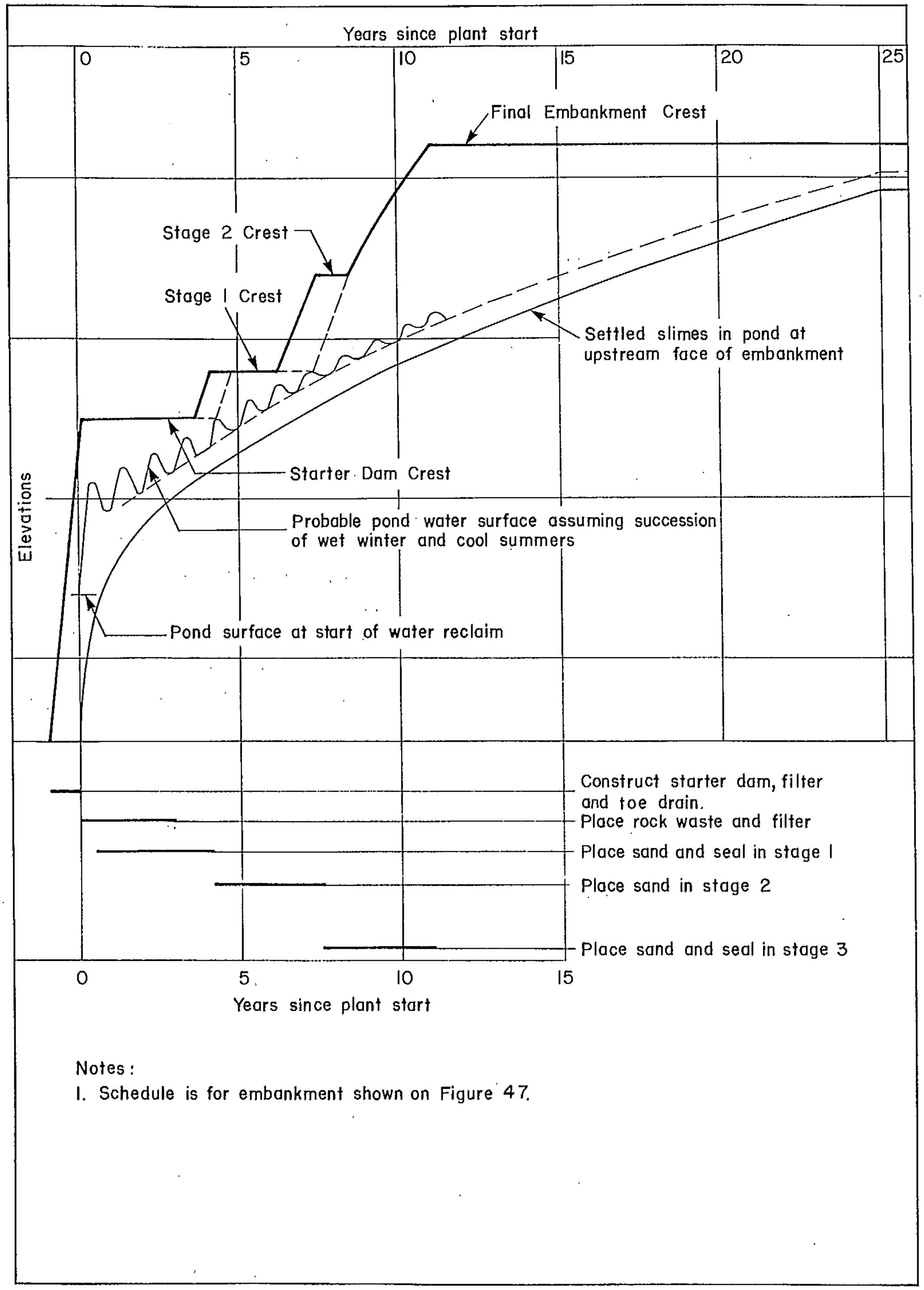


Fig 49 - Tailings embankment construction schedule construction berm.

DAM FOUNDATIONS

334. A foundation investigation should be carried out for all dams over 25 ft (8 m) in height or where it is suspected that the foundation may contain weak or incompetent materials. In addition, a separate foundation investigation should be made whenever a decant tower and tunnel is used for the tailings basin effluent disposal system.

335. Foundation preparation measures are made to reduce the possibility of bearing failure, sliding on the base, excessive settlement, differential settlement which is particularly important when weak foundation materials overlie a bedrock profile which changes rapidly, and to restrict the flow of seepage under a dam when the foundation is pervious.

336. If the foundation surface contains a layer of weak, incompetent organic material, this should be removed during preparation of the dam foundation.

337. When the foundation stratigraphy indicates weak layers of compressible materials such as clays or low density silts, it may be desirable to remove or displace them. The decision will depend on thickness and elevation of the weak material, and it should be made during the design stage as a result of the findings of the foundation investigation.

338. Seals or cutoffs are used to restrict the flow of seepage. When the upper horizons of the foundation stratum contain pervious materials, an impervious cutoff trench may be used between the impervious zone of the embankment and the impervious stratum below the pervious foundation material. It may also be necessary to seal exposed fissures or faults in the bedrock by grouting or other measures.

339. To provide an effective dam the upstream zone of the embankment, the foundation, the abutments, and the tailings basin should all be sealed. In the case of a tailings dam, if the material is relatively fine - 60% minus 200 mesh - it is usually a simple process to seal the tailings basin and abutments with tailings slimes. On the other hand, if the dam retains significant depths of water for extended periods, the treatment of the foundation can be critical to its stability. If it is necessary for the starter dam or the first stage of the tailings dam to retain a considerable depth of water - more than 8-10 ft (2.5 - 3 m) - unassisted by blanketing of the upstream slope with tailings, it should be designed as a water dam.

340. Special precautions must be exercised to provide seepage diaphragms along decant tunnels to restrict seepage along the external surface.

FILL COMPACTION

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

342. Variables affecting compaction are: (a) type of compactor (non-vibratory pneumatic, steel-wheel, sheep's foot, and grid rollers, vibratory steelwheel rollers, plate compactors and track-type tractors), (b) weight and energy of the compactor, (c) thickness of layers, and (d) placement water content.

343. The types of compactors and layer thicknesses suitable for the compacting of various classes of soils to 95-100% of Standard Proctor Maximum Density are indicated in Table 13. For cohesionless tailings sands, compacting by track-type tractor is normally adequate and efficient. Controlling placement water is usually unnecessary as it drains quickly when the water content is high and slime content is low. The tractor weight is limited to the support capacity of the wet sand.

344. For cohesionless borrow and waste material such as rockfill, gravel and clean sand, 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

Table 13: Compaction equipment and methods

Equipment type	Applicability	Requirements for compaction of 95 to 100 per cent Standard Proctor Maximum Density: Compacted lift thickness in. (cm)	Passes or coverages	Dimensions and weight of equipment	Possible variations in equipment
Sheepsfoot rollers	For fine-grained soils or dirty coarse-grained soils with more than 20% passing No. 200 mesh; 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 (15)	4 to 6 passes for fine-grained soil. 6 to 8 passes for coarse-grained soil	Soil type / Foot contact area in.^{2} (cm^2) / Foot contact pressures psi (MPa): Fine-grained soil Pl > 30: 5 to 12 (32 to 77); 250 to 500 (17-34) Fine-grained soil Pl < 30: 7 to 14 (45-90); 200 to 400 (1.4-2.8) Coarse-grained soil: 10 to 14 (64-90); 150 to 250 (1.0-1.7) Efficient compaction of wet soils requires less contact pressures than the same soils at lower moisture contents	For earth dam, highway, and airfield work, drum of 60-in. dia. (152 cm), loaded to 1.5 to 3 tons per lineal ft (43.7-87.5 kN per lineal m) of drum generally is utilized; for smaller projects, 40-in. dia (101-cm) drum, loaded to 0.75 to 1.75 tons per lineal ft (21.9-43.7 kN per lineal m) 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% passing No. 200 mesh	10 (25)	3 to 5 coverages	Tire inflation pressures of 60 to 80 psi (0.41-0.55 MPa) for clean granular material or base course and subgrade compaction; wheel load 18,000 to 25,000 lb (80-111 kN); tire inflation pressures in excess of 65 psi (0.45 MPa) for fine-grained soils of high plasticity; for uniform clean sands or silty fine sands, use large size tires with pressure of 40 to 50 psi (0.28-0.34 MPa)	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
	For fine-grained soils or well-graded, dirty coarse-grained soils with more than 8% passing No. 200 mesh	6-8 (15-20)	4-6 coverages		

Smooth wheel rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures;	8-12 (20-30)	4 coverages	Tandem type rollers for base course or subgrade compaction, 10 to 15 ton weight (89-133 kN), 300 to 500 lb per lineal in. (3.4-5.6 kN lineal cm) of width of real roller.	3-wheel rollers obtainable in wide range of sizes; 2-wheel tandem rollers are available in the range of 1 to 20 ton (8.9-178 kN) weight; 3-axle tandem rollers are generally used in the range of 10 to 20 ton (89-178 kN) weight; very heavy rollers are used for proof rolling of sub-grade or base course
	may be used for fine-grained soils other than in earth dams; not suitable for clean well-graded sands or silty uniform sands	6-8 (15-20)	6 coverages	3-wheel roller for compaction of fine-grained soil; weights from 5 to 6 tons (40-53 kN) for materials of low plasticity to 10 tons (89 kN) for materials of high plasticity	
Vibrating baseplate compactors	For coarse-grained soils with less than about 12% passing No. 200 mesh; best suited for materials with 4 to 8% passing No. 200, placed thoroughly wet	8-10 (20-25)	3 coverages	Single pads or plates should weigh no less than 200 lb (0.89 kN); 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 1.5 to 15 ft (0.45-4.57 m); 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% passing No. 200 mesh, placed thoroughly wet	10-12 (25-30)	3 to 4 coverages	No smaller than D8 tractor with blade, 34,500 lb (153 kN) weight, for high compaction	Tractor weight up to 60,000 lb
Power tamper or rammer	For difficult access, trench backfill; suitable for all inorganic soils	4-6 in. (10-15 cm) for silt or clay, 6 in. (15 cm) for coarse-grained soils	2 coverages	30 lb (0.13 kN) minimum weight; considerable range is tolerable, depending on materials and conditions	Weights up to 250 lb (1.11 kN); foot diameter 4 to 10 in. (1.57-3.93 cm)

additional compaction is required. Control of water content will increase the efficiency of compaction but is not essential for such cohesionless materials. For stony materials, the maximum stone size permitted should be two thirds of the specified compaction layer thickness. Layer thicknesses up to 24 in. can be used for granular material under favourable conditions with heavy vibratory steel wheel compactors.

345. Cohesive soils are efficiently compacted by heavy pneumatic or sheepsfoot rollers. Control of placement water content is important for 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 placing of the overlying layer.

EFFLUENT SYSTEMS

346. One of four basic systems is generally used for discharging clarified water from a tailings pond.

a. Simple spillways or overflow weirs are used for relatively low embankments. The weir elevations are increased with time to maintain a clear effluent.
b. The most common system is a decant tower and conduit through the embankment. The surface water of the pond flows into a vertical drop inlet in the tower and then through the horizontal conduit which passes through the embankment and discharges the clear effluent downstream. The elevation of the inlet is adjustable to assure sufficient depth of water in the pool to provide adequate retention time for clarification and a clear effluent.
c. Floating pump barges are used on some tailings ponds to reclaim effluent water for plant processing.
d. Syphon pipes installed over the crest of the embankment to maintain an approximately constant depth by discharging the effluent overflow downstream (Fig 50).

347. Spillways or overflow weirs have the advantages of being economical and easy to construct, particularly for low embankments. A power supply is not required unless the effluent is reclaimed for process water; they are simple to operate and make it easy to regulate pool elevation. The disadvantage for high dams is that they are expensive to construct where regulation of pool elevation is required over the total height.

348. Decant towers and conduits will likely remain the predominant method of tailings effluent disposal system for the foreseeable future. Their advantages are: they can be constructed to cover the whole range of pond surface elevations; they are simple to operate, the primary requirement being a periodic increase in the discharge elevation; there are no requirements for a power distribution system or operating power and, therefore, electrical interruptions do not affect the system; and the system can function as a permanent drain to handle runoff after the tailings operation has been abandoned. When a reclaim system is used, a permanent pumphouse can be established in the pond. The disadvantages in the past have been numerous structural failures with decant towers and conduits as a result of differential settlement, seepage, and collapse. Small operations with low flow velocities are inclined to freeze during extremely cold weather.

349. A floating pump barge can serve the dual purpose of an effluent disposal system and reclaim water system. Reclaiming the water from the pond above the dam reduces the pumping head for the reclaim pumps. Their disadvantages are that a power system is required; provision must be made using high pressure bubble pipes, and it is essential to have standby pumping equipment; an emergency spillway should be provided; the barge will have to be moved periodically as tailings accumulate in the basin; a pontoon bridge to the barge or some alternative method of access must be provided; power interruptions can cause severe operating difficulties during freezing weather; and there is no provision for tailings basin drainage after abandonment.

350. Syphons have a relatively low initial cost if the pool is close to the embankment. However, they lose their prime under varying pool conditions; they can be difficult to start and to

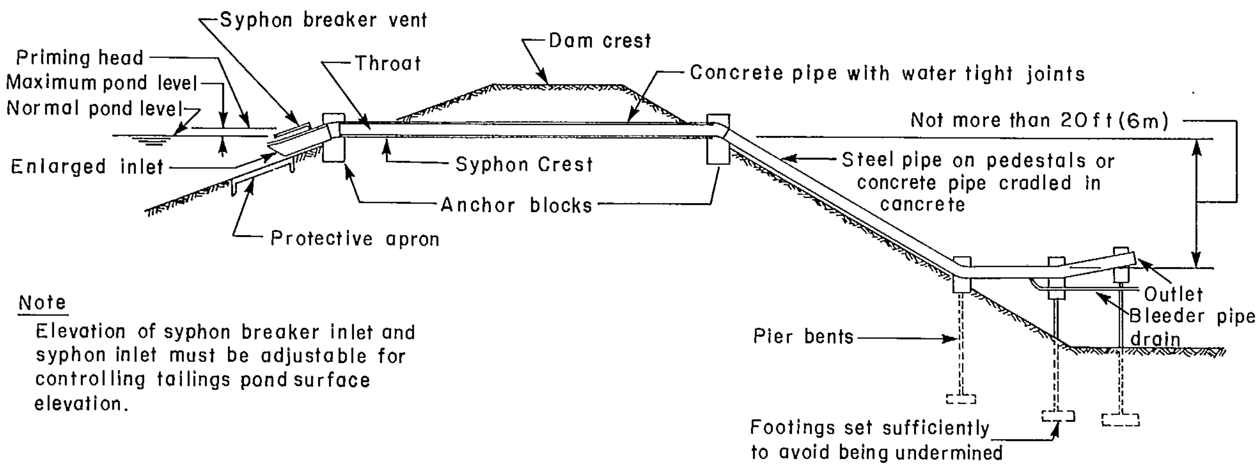


Fig 50 - Small syphon spillway

regulate; the water pool must be relatively close to the embankment; and they are unreliable during freezing weather.

MAINTENANCE

351. Maintenance should be programmed. The formation of tailings embankments and waste piles occurs over a period of years. Conditions develop during these long periods which affect stability of the embankment. In this respect, tailings dams differ substantially from normal water retaining dams, the latter usually being constructed over a relatively short period with rigid control over the quality of the material and construction methods.

352. The characteristics of the material and construction methods may change substantially over the years. Such changes can alter the conditions governing the stability of an embankment. Changes may take place in crest levels, water levels, embankment slopes, cross sections, seepage conditions, and material characteristics. A continuous program of inspection and maintenance of the embankment is therefore required, beginning with the start of waste disposal and possibly continuing after the embankment is completed.

353. The main objectives of such a program are to ascertain: (a) whether the embankment and its foundation are behaving in the manner anticipated in the design - are they moving, settling, cracking, eroding, sloughing, or seeping more than anticipated? (b) whether the waste or 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 embankment stability? (c) whether the distribution of material in the embankment and tailings pond is similar to that assumed in the design - are the waste disposal procedures affecting material distribution and material characteristics? (d) 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 maintain adequate freeboard above the rising pond surface? are actual runoff and evaporation rates of the order forecasted? (e) whether embankment drainage is adequate - is the capacity of culverts adequate to pass anticipated runoff and seepage? - have culverts collapsed or become blocked? is the embankment material becoming saturated by seepage or runoff? - is piping or subsurface erosion occurring in tailings embankments or into decant culverts passing through them? and (f) whether the water table within the embankment varies with pond

level and precipitation - has there been an increase in water table 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 a developing instability of the embankment.

REMEDIAL MEASURES

354. The extent and nature of remedial measures to maintain or improve stability of mine waste embankments will vary with each individual embankment. Some embankments may require extensive work in addition to that anticipated in design. For others, minor remedial measures may be adequate.

355. Slope flattening is illustrated in Fig 51. 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 relocated at the toe to increase stability, measures should be taken to ensure it does not impede internal drainage from the embankment. A drain should be provided if the material has a low permeability compared with that of the material in the base of the embankment.

356. Daily examination is usually required of such areas as tailings lines, decant systems, emergency spillways, and slurry discharge points. Special inspections are usually appropriate after heavy rains. Detailed inspections of high embankments - 25 ft (8 m) or greater - and of those where instability would be serious, should be made at least twice each year during waste disposal operations. Such inspections should include not only on-site inspection of the embankment itself

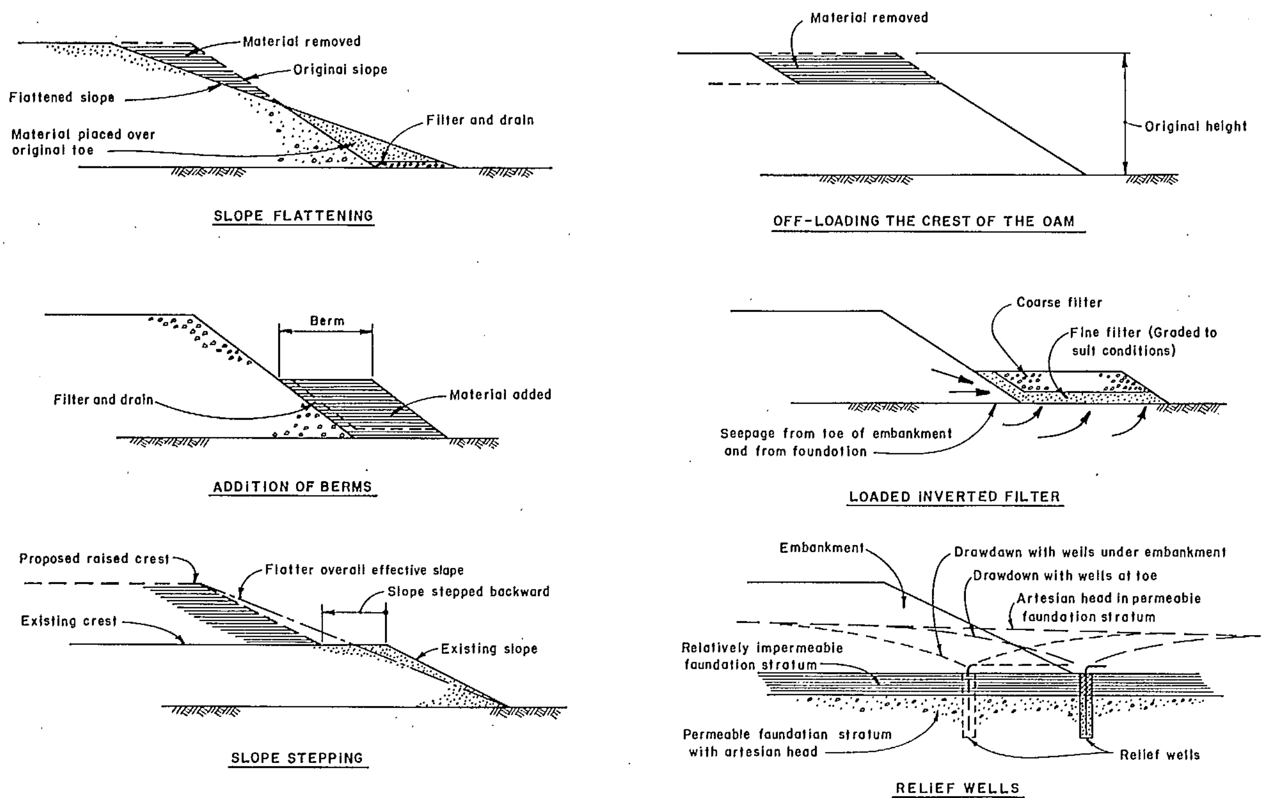


Fig 51 - Remedial measures for improving embankment stability but also a review of the records of instrumentation installed in or near the embankment.

357. Particular points to be checked during inspections are: (a) the presence of longitudinal and transverse cracks on the crest or slopes of the embankment; (b) the extent and rate of horizontal and vertical movement indicated by surface reference markers, settlement rods and slope indicators - are the movements accelerating? (c) any sign of foundation heaving near toes of the slopes; (d) variations in piezometric water levels and seepage flows, the development of any springs or wet areas on the embankment and foundation surfaces; (e) conditions at seepage exit points and at culvert or decant conduit 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 signs of sinkholes appearing on the upstream face of tailings embankments? the latter two points could indicate the occurrence of sub-surface erosion; (f) nature of the material placed in waste retaining embankments, particularly spigotted or cycloned sands in tailings embankments - the important characteristics will be their gradation and moisture content; (g) geometry of the embankment - do the heights, widths and slopes comply with those specified in the design? has any unanticipated excavation been made in the embankment foundations, abutments or slopes that could threaten its stability? (h) spillway and diversion structures - are they in good repair and capable of performing as required by the design? and (i) the extent of any erosion on the embankment and abutment slopes and in the channels of stream diversions or spillways. Careful attention to these points can provide forewarning of developing instability.

358. A special case of slope flattening is the addition of fill in the toe as illustrated in Fig 51. Generally, this action can improve foundation stability but may not be very effective in increasing the factor of safety against sliding unless it is at least one third the height of the embankment. Compaction is usually desirable.

359. If an embankment is to be raised, it may be necessary to flatten the overall slope by stepping the toe of the new waste backward from the original crest.

360. Impervious cores, impervious cut-offs, and upstream seals are used to restrict seepage through tailings embankments, foundations, and abutments. If excessive seepage occurs after completing construction of a tailings dam, it is often possible to reduce or minimize seepage by applying an upstream seal of tailings slimes.

361. Graded filters or transition zones are used to prevent internal erosion or piping caused by seepage through the structure. The filter material is always placed downstream with respect to the flow of seepage from the fine material to prevent migration of the fine particles into the coarse material. To prevent piping near the downstream toe, pervious material can be added at the toe with a suitable filter between the original and the new.

362. Relief wells can sometimes be used to lower the water table under embankment slopes, as shown in Fig 51. Generally, they are not very effective when the holes are located outside the embankment fill. They have the disadvantages of requiring a power source at the tailings dam and regular maintenance for the pumping system.

363. In extreme cases (for tailings dams), it may be desirable to grout highly fractured bedrock abutments or bedrock foundations.

364. When the quality of the seepage water constitutes an undesirable source of pollution and all practical methods have been employed to minimized it, the seepage may be collected and returned to the tailings basin.

SURFACE DRAINAGE

365. All streams and natural water courses should be diverted around tailings basins and waste dumps to prevent possible sources of contamination of the fresh water.

366. Any benches or berms on the downstream slopes of tailings dams should have a gently uniform grade away from the embankment to permit gradual runoff from rainfall and snowmelt. When

berms slope into the embankment, catch basins are formed. There have been numerous cases of severe gullying and surface erosion where flash rainstorms cause these catch basins to overtop. When drainage ditches are constructed on the berms of tailings embankments and disposal operations are abandoned, the drainage ditches often plug causing severe erosion problems when overtopping occurs.

367. Tailings embankments should be protected by seeding or applying riprap to prevent surface erosion by runoff.

368. Re-entrant corners or valleys should be avoided wherever possible on downstream embankment slopes to prevent a concentration of runoff. Where re-entrant corners are necessary because of property configuration, special erosion protection will be necessary to avoid gullying.

369. Breaching a tailings embankment is an unacceptable and dangerous procedure to prevent an accumulation of water in an abandoned tailings basin. An emergency spillway or decant system are acceptable alternatives. The capacity of the system should be suitable to accommodate the peak flood with a 100-year return interval. It is important to carry the discharge from a spillway well beyond the toe of the embankment to eliminate the possibility of undercutting the toe.

WASTE PILES

370. The method employed for placing dry waste will depend on: (a) location of the waste pile in relation to the mine or processing plant, (b) topography of the waste disposal area, (c) foundation conditions for the waste pile, (d) nature of the waste materials, (e) maximum height of the waste dump, and (f) the type of material-handling equipment available.

371. Simple dumping from mobile haulage units or conveyors is usually the preferred method. Several important factors must be considered when constructing waste piles. Site and foundation investigation should be made for all high embankments. If the foundation is covered with a thin veneer of incompetent organic material, it should be removed before waste disposal operations proceed. Streams and water courses should be diverted around the disposal area, and adequate surface drainage should be provided to minimize water content within the waste pile. If the foundation is saturated and contains weak silts or clays, it will be necessary to evaluate stability with respect to the rate of construction of the waste pile. Rapid construction with high pressures exerted by the surcharge of waste material can create high pore water pressures within the foundation. To reduce the pore pressures and improve the rate of consolidation, a blanket of free-draining granular material can be placed on the surface of the foundation. A suitable filter or transition zone may have to be placed both above and below the blanket to prevent it from becoming clogged with fine material. In some instances it may be necessary to monitor pore water pressure with piezometers and to limit the rate of build up of the waste pile to prevent instability.

372. Safety precautions to be exercised in dumping waste material will depend on whether the pile is constructed from the bottom up or from the top down. There are operations where waste material is dumped over the side of an embankment exceeding 2,000 ft (610 m) in height. When trucks are used to dump waste from the top, it is imperative to dump short of the crest and then doze the material over the crest. When material is dumped continuously over the crest of an embankment, a series of slides will occur as a result of the crest being extended in a horizontal direction, creating a temporary angle steeper than the angle of repose and then suddenly breaking down from time to time. When a conveyor is used to dump the waste material over an embankment, the main conveyor can be extended by a short movable discharge conveyor incorporating a flinger. The trajectory of the waste material from the flinger will be such that the crest failures will have a minimum effect on the supporting structure for the conveyor.

373. In the design of mine waste piles, the shear strength and other characteristics of the wastes and foundations are often not adequately known in advance. In some circumstances, therefore, trial sections of the pile might be constructed in the field. The embankments can

then be sampled and tested in the laboratory. Improvements in shear strength produced by compaction can also be investigated on trial embankments. In some cases, such embankments can be used as a trial for the foundation.

374. 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. Oxidation, spontaneous combustion, and burning, will not occur in the absence of air. For burning to occur with a 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.

375. 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 material 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, ie, within the range of about 260-300°C. However, at normal temperatures and in the presence of moisture, these materials are likely to heat spontaneously by the action of certain micro-organisms.

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

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

378. Pyrite, in sufficient quantities 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.

379. At relatively low temperatures, an increase in free moisture increases the rate of 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.

380. 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, the air remains stagnant and the heat generated is retained in the mass. When the available oxygen is consumed, heating stops. With intermediate gradings, the conditions for spontaneous heating are favourable, and the heated parts may form hot spots and eventually break into flame.

381. 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.

382. There is no inexpensive method available to prevent spontaneous combustion of existing coal waste piles which are inclined to catch fire. Removal of all carbonaceous material, extinguishing burning ground, quenching and rebanking in compacted layers are the principal ways of guaranteeing that fires are eliminated.

383. Fire risk can be reduced through close attention to the following points: (a) removing all vegetation in front of an extending pile, (b) ensuring that boiler ashes are properly quenched before being deposited on the pile, (c) compacting the waste material and streamlining the outside surface of the pile, which will restrict the entrance of air, (d) prohibiting the placing of flammable materials (coal, wood, sawdust, sacking,

cardboard, paper, waste oil, oily rags, and containers for oil, grease, paint, etc), on the pile, (e) backfilling and sealing all excavations and boreholes, and (f) inspecting the pile regularly to detect fumes and hot spots.

384. The method of controlling fire in a waste pile will depend on the size and nature of the pile, local conditions, location, extent and progress of the fire and the availability of suitable extinguishing materials. Methods proven by experience include excavating, trenching, blanketing with inert material, and injecting a slurry of incombustible matter and water. Water sprays should not be used except in special circumstances because of the danger of explosion.

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

386. A trench may be dug into the waste pile to isolate and retard 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 ft (2 m) wide and 6 ft (2 m) deep. Other inert material 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.

387. Injection of water alone into a burning pile is rarely effective in extinguishing the fire permanently and may lead to an 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 because it produces carbon dioxide on heating which does not support combustion. This effect, together with water in the slurry, may reduce the temperature so 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 in such cases there may be some risk of explosion.

388. Blanketing 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, if it completely covers the affected area, particularly the lower parts of the surrounding slopes, and if it is maintained airtight. Constant inspection and maintenance are required to ensure a continuous seal.

389. 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 exploding.

390. Some of the disadvantages of using water sprays are: (a) the steam formed is inert and is only useful in displacing 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, (b) any explosion of water-gas and air may aggravate the problem by opening the waste pile to air and causing the fire to spread, (c) 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, (d) the products formed by oxidation of pyrite may be leached out by water, exposing new surfaces of pyrite, and (d) the use of large quantities of water may adversely affect stability of the pile; it may also produce a large volume of acidic drainage.

INSTRUMENTATION

391. Many factors affecting the stability of mine waste embankments can change their active life. Where changes may be critical for stability, instrumentation should be installed in the embankment or its foundation to monitor those changes and to facilitate the prediction of unstable conditions.

392. Instruments can be installed to measure pore water pressures, seepage flows, embankment movements, and total pressures. More information is provided in Chapter 8 on monitoring.

393. Piezometers are used to measure the pore water pressure in the soil. A simple and effective piezometer is the Casagrande type illustrated in Fig 52. 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. The Casagrande type piezometer has a porous ceramic stone element and is designed to reflect pressure changes with a minimum of time lag. Similar reacting types can be installed using porous plastic, porous bronze, perforated steel casing, or steel casing and well points. Alternatively, hydraulic or electrical piezometers are available which can be installed at various levels in an embankment. These are described in the references (7). Generally their operation is more complicated and their reliability over long periods requires great care in fabrication and installation.

394. A number of pressure cells are available for measuring total pressure, ie, 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.

395. Records of seepage flows will indicate when significant changes occur and permit an evaluation of potential problems from piping. The seepage water emerging downstream from the embankment can be collected and directed to a simple weir for flow measurements.

396. Movement of the embankment is usually a good measure of its degree of stability. Markers can be installed on the surface for periodic surveys; aligning these in a straight line-of-sight permits rapid detection of horizontal movement. Levelling of temporary bench marks provides measurement of surface settling. Successive measurements between two pegs spaced on each side of cracks will show if these cracks are widening and if the opening rate is accelerating.

397. Another device for measuring horizontal movement is the slope-indicator. For this device, telescoping cylindrical casing is installed in the embankment during construction. The sensing element is lowered down grooves inside the casing and measures the slope of the casing in two directions at right angles. From the measured slopes, the horizontal movements occurring over the height of the casing can be calculated. These instruments are supplied by various companies.

398. Permanent records should be made of all instrument readings relating to mine waste embankments. The frequency of readings will depend on

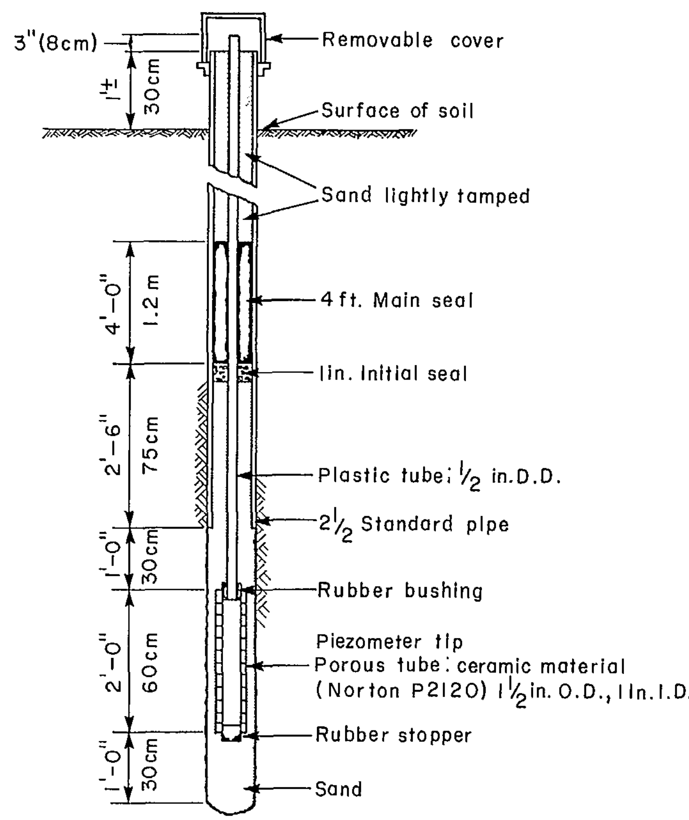


Fig 52 - The Casagrande piezometer

the importance of the structure, the critical nature of the parameters and previous observations. In most instances, it is desirable to make frequent observations during construction and immediately after completing the construction phase. When the records indicate that conditions are relatively stable, frequency of observations can be extended from daily, weekly, or monthly, to as little as twice yearly and even only after unusual conditions such as heavy runoffs, peak floods or seismic activity. Regardless of quantity and quality of observations they are of little value unless reviewed at regular intervals.

ABANDONED EMBANKMENTS

399. Waste embankments may require attention some time after being abandoned. This may be because of progressive deterioration under the action of water and weather or because of further urban or industrial development close to the embankments.

400. Generally, embankments should be put into such a condition before being abandoned that only occasional maintenance will be required to prevent deterioration to the point where instability can develop. This will usually require care to: (a) provide reliable drainage systems capable of functioning adequately for many years with little maintenance, (b) protect embankment slopes against surface erosion and weathering, and (c) prevent spontaneous combustion of coal waste piles.

401. There will be no reason why instability should decrease below that existing at the time of completion if adequate provisions are made (a) to prevent saturation, softening, weathering, and chemical change of the material in the waste embankment, (b) to limit surface erosion, and (c) to prevent the rise of the water table in the embankment. Such provisions will normally be included in the original design.

402. The most appropriate remedial measures to be taken with an existing embankment showing signs of instability or which has become a threat because of adjacent new developments, will depend on evaluating its existing stability and on the cause of the developing instability. Such evaluations require determining the existing geometry, assessing the characteristics of the various materials, and determining pore water pressures within the embankment.

403. Surveys can readily determine the surface geometry of the embankment and its foundation. If adequate records are kept during the waste disposal period, it should be possible to determine 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 are not kept, it may be necessary to drill test holes. With sufficient information, the probable cause of instability can be deduced and analyses made to evaluate the condition of the embankment; appropriate remedial measures can then be determined.

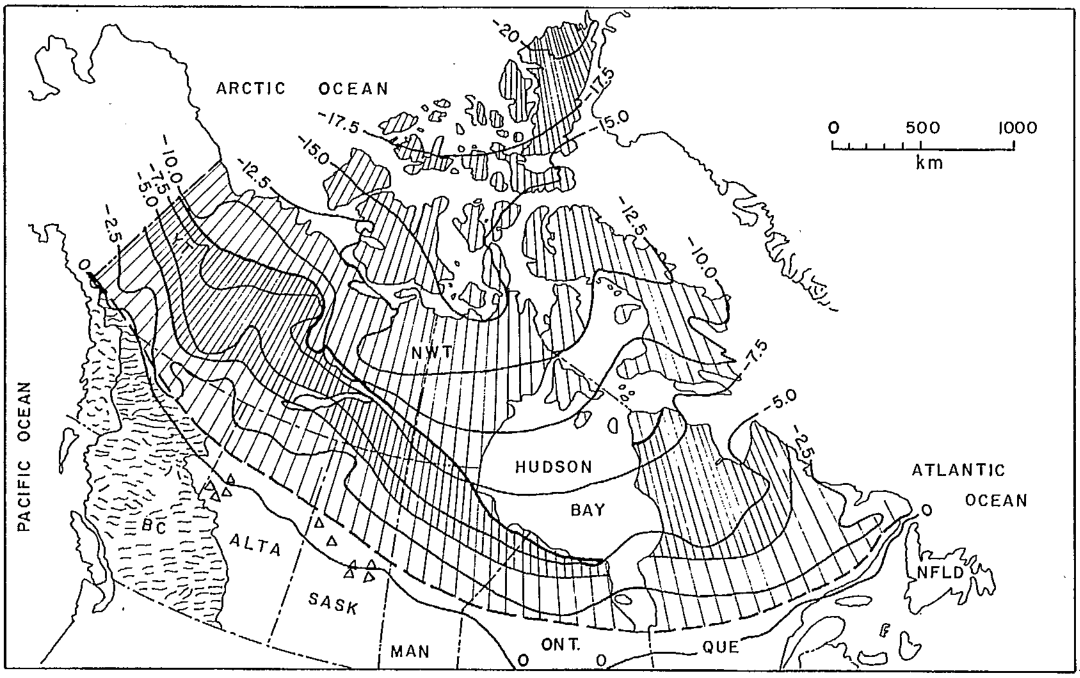
PERMAFROST EFFECTS

BASIC MECHANISMS

404. Permafrost is prevalent over wide areas of Canada. The map shown in Fig 53 indicates those areas in which it is a factor to be considered in the design of mine embankments. Placement of structures of this type has the effect of upsetting the natural thermal equilibrium in the permafrost. The nature of the problem lies in assessing effects caused by the embankment itself. These considerations may dictate redesign or relocation of a proposed structure in permafrost regions.

405. Permafrost may be defined as any soil type which is perennially frozen. The soil may be solid rock, gravel, sands, silt, organic material or clay. Those soils which have high moisture content are particularly important. Permafrost exists because of the energy balance within the soil. A sub-surface soil layer receives heat from geothermal sources below. It also receives heat from the atmosphere during the summer period, and in general it loses heat to the atmosphere during the winter. Permafrost reflects an equilibrium or balanced state of a stable, permanent nature in which the sum of heat losses and gains must equal zero.

406. The modes of energy transport include convection, solar and atmospheric radiation, moisture accumulation and evaporation. These all occur at the surface, with thermal conduction and possibly moisture flow (convection) occurring below the surface. Annual freeze-thaw cycling takes place in the upper regions of the soil, referred to as the active layer. Below the soil surface, the seasonal variations in temperature are attenuated and delayed in time. The depth of active layer penetration, which varies, leaves the physical properties of the permanently frozen portions largely unaffected - even though they are functions of temperature. The depth of



LEGEND

Continuous permafrost zone

Southern limit of continuous permafrost zone

Widespread permafrost

Southern fringe of permafrost region

Southern limit of permafrost

Patches of permafrost observed in peat bogs south of permafrost limit

Permafrost areas at high altitude in Cordillera south of permafrost limit

Mean annual air temperature, °C

Fig 53 - Distribution of permafrost in Canada

penetration is dependent on the extent of seasonal variations and on soil thermal properties.

407. Permafrost can be classified into two groups as either continuous, occurring over widespread areas, or discontinuous, occurring in limited areas or a region usually as a result of terrain features and micro-climatic conditions. It may be further classified as either cold or warm.

408. Warm permafrost is that which has a temperature close to 32°F (0°C). It exists commonly in areas of discontinuous permafrost. Warm permafrost is typified by large active layers. An upset in the equilibrium, unless compensated for by some artificial means, results in irreversible degradation of the frozen state of the soil. When this occurs in soils of high moisture content, the resulting subsidence creates a hazardous condition.

409. Cold permafrost is found in regions of continuous permafrost. It commonly has a shallow active layer and is considered more stable; however, the same care must be taken in assessing the effects of disturbances.

410. The most widely used variation of temperature conditions for analytical purposes is that of a sinusoidal surface temperature oscillating about a mean annual surface temperature. Such a variation may be represented as follows (57):

$$T_s = T_m + A \sin (\eta\text{-}b) \qquad \text{eq 11}$$

where T_s is the soil surface temperature, T_m is the mean annual soil surface temperature, A is the amplitude of the temperature swing, η is the time of year under consideration, and b is the time phase term so that the temperatures may be related to the first day of the year.

411. When equation 11 is combined with a one-dimensional form of the standard heat transfer equation, a solution of the form in equation 12 is obtained describing the time-temperature depth relationship.

$$T(x,t) = T_m + Ae^{-x\sqrt{\frac{\pi f}{\alpha}}} \sin(2\pi ft-x \sqrt{\frac{\pi f}{\alpha}}) \qquad \text{eq 12}$$

where T(x,t) is the temperature at any depth and time, t is the day of the year expressed as a fraction, x is the depth below surface, f is frequency which in this case is one cycle per year, and α is the thermal diffusivity.

412. Should the amplitude of the surface temperature oscillation increase, then the depth of the active layer increases. It is interesting to note that most man-made disturbances at the surface have the effect of increasing this amplitude and thus increasing the thickness of the active layer.

413. As most soils vary in moisture content and as moisture in the soils undergoes a thermodynamic phase change on freezing, the latent heat of this moisture should be considered. This may be accomplished with the finite difference technique by determining the total heat flow to or from the volume element from its surrounding elements and then adding the net flow over the successive time intervals until the required latent heat for the volume has been absorbed or rejected. As melting or freezing takes place at a constant temperature, the element temperature in this case is not allowed to change until the required latent heat has been acquired or lost.

414. Figure 54 shows the effect of latent heat considerations on the envelope of time-temperature fluctuation (case 3). Also plotted on this figure are the envelopes of the fluctuations for cases of the thawed (case 1) and frozen (case 2) media, with appropriate properties for each but neglecting latent heat.

415. Though the atmospheric boundary conditions and the soil properties used in Fig 54 are an extreme case for marginal permafrost conditions, they do show the difference between the predicted depth of thaw for the case of homogeneous soil properties and that when latent heat and the properties of both the thawed and frozen states are considered. Precise predictions of active layer depth depends on accurate knowledge of the soil moisture content.

416. Vapour pressure gradients are present between warm and cold regions of the soil. These gradients act to drive moisture from warm regions of the soil to cooler regions where freezing of the moisture takes place. This mechanism permits large layers of pure ice to accumulate within the soil. Melt water accumulates and freezes in the ground cracks caused by the thermal contraction of the permafrost. These conditions are common in permafrost areas. Their presence affects the thermal behaviour of the soil, and the possibility of their thawing is a major factor in design, as seen in the following sections.

SITE INVESTIGATIONS

417. To accomplish a site investigation in permafrost, information is required to establish the effects of the mine embankment and impoundment on the sub-surface strata and to evaluate the performance of the embankment itself. A preliminary assessment of the site as to type and extent of permafrost may be obtained from existing literature such as permafrost maps, climatological data and reports on surficial geology. This preliminary investigation should be conducted before any on-site programs are planned in detail. It will provide information as to whether the site is located in a region of continuous or discontinuous permafrost, as well as depth of the active layer,

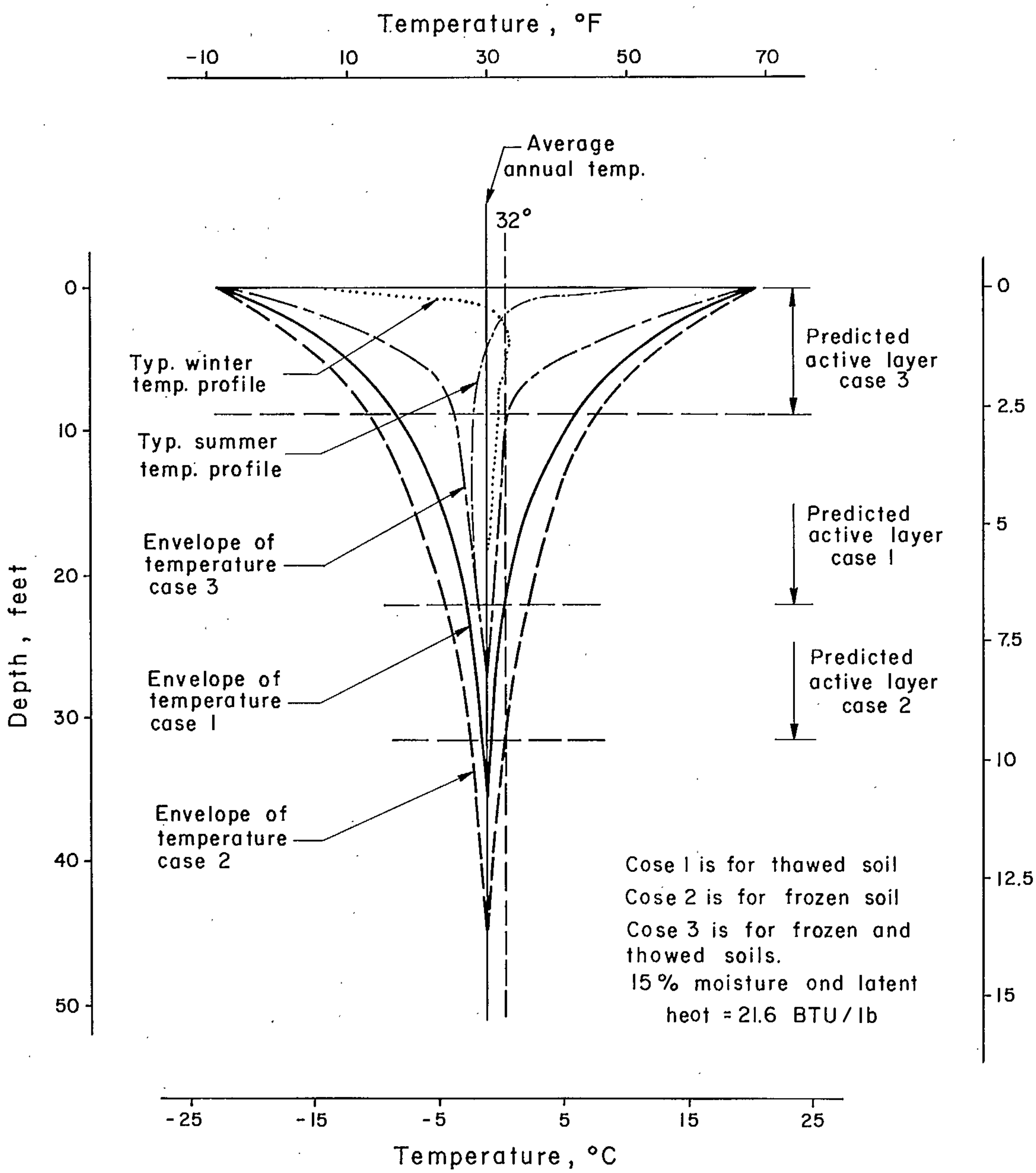


Fig 54 - Time-temperature fluctuation in permafrost probable ice content, and distribution of ice. Should the site be located in an area where permafrost is discontinuous or sporadic and of relatively high temperature - near 32°F - then a more comprehensive on-site investigation should be planned. In addition, if existing information indicates the presence of massive formations of ice in the soil, or that soil conditions are conducive to their existence, then an extensive on-site investigation is indicated with a very comprehensive borehole program.

418. Under conditions where continuous permafrost is known to exist as a result of preliminary information, it would be wise to conduct a comprehensive investigation similar to those where the permafrost is discontinuous or sporadic. This is particularly important for those cases where the continuous permafrost table is shallow and its temperature is near equilibrium for the frozen ground.

419. The planning of site investigations, and particularly of extensive on-site investigations, should include some or all of the factors in the following two major categories: (a) the location and identification of the extent of permafrost soils and potential permafrost-associated hazards, and (b) obtaining specific data to aid in design and location of the embankment.

420. Location and identification of permafrost should be accomplished through a combined program of climatological analysis, air photo interpretation and sub-surface investigation where sufficient on-site information cannot be gathered or estimated from existing data. Extent, both areal and vertical, of the permafrost soils should be ascertained and related hazards should be identified. In permafrost areas, climatological data plays a highly significant role in engineering decisions. Where complete climatological data does not exist, measurements should be made to allow correlation of local conditions with nearby meteorological stations. Complete climatological data would consist of the following: (a) ambient air temperature, (b) relative or specific humidity, (c) solar radiation, both spectoral and all-wave, (d) precipitation, rain and snow cover, and snow density, (e) wind speed, duration and direction, and (f) cloud cover.

421. Where extensive structures are contemplated, justification may exist for measuring ground temperatures at several locations to depths of 50 ft (15 m) or more. A thermocouple or thermistor installation will provide specific design information as well as monitor future thermal performance of the mine embankment structure. Typical installations are shown in Fig 55. Where groundwater and surface runoff are recognized as critical factors, measurement of hydrostatic pressures using piezometric devices may be indicated. Economy would be realized if installation of these devices were made in conjunction with any planned soil boring programs.

422. The designer will require specific information which is both fundamental and in sufficient detail to design and locate the embankment. This information is derived from undisturbed soils samples, which provide values of density and moisture content. Particular care should be taken in collecting frozen samples to ensure they do not thaw before testing. The frozen sample may be subjected to mechanical analysis, consolidation, permeability, and other soil tests if this information is considered necessary. Sampling methods and drilling techniques are fairly well developed for frozen soils; however, difficulty may be encountered in sampling frozen gravels. Methods for obtaining core samples in frozen soils and their descriptions are discussed by Scott (58).

WASTE EMBANKMENTS

423. The interrelationship of the waste embankment and the permafrost foundation soil is complex. The embankment has an effect on the permafrost below, and the presence of the permafrost affects the thermal regime within the embankment. Detailed analysis is required before the transient effects and long term results may be evaluated. Computer simulation is useful in performing this analysis.

424. Preliminary analysis requires a knowledge of the existing temperature profile in the undisturbed soil. Soil temperatures may be obtained from actual measured values or computed from closed form solutions. Variations in the soil

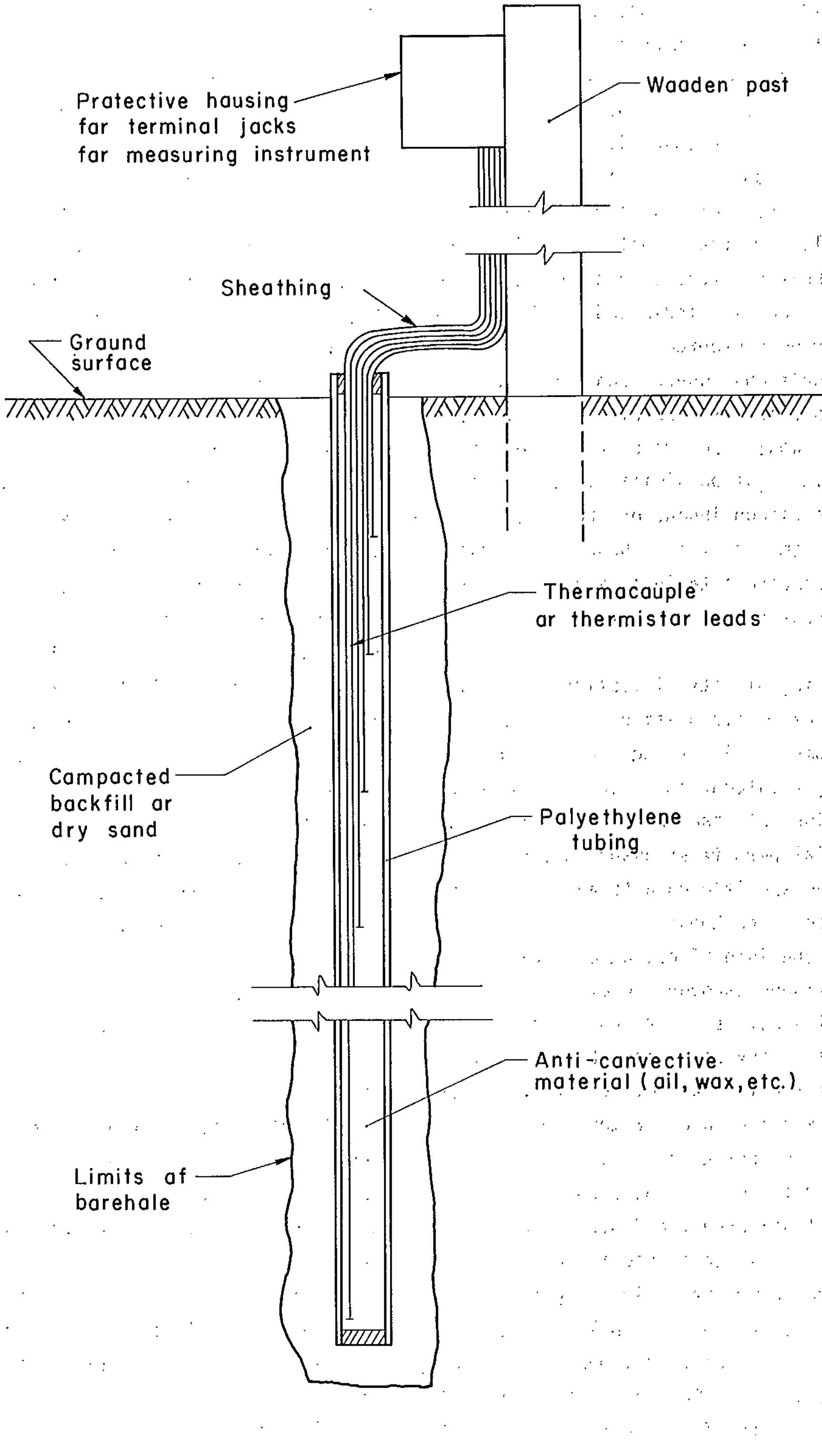


Fig 55 - Layout of permafrost monitoring instruments strata are determined and such physical properties as moisture content, dry density, specific gravity and thermal conductivity are obtained from measured or estimated values. Also considered are possible groundwater flow and probable location and distribution of ice masses.

425. Moisture transport in the soil, whether it be groundwater flow caused by capillary action or by vapour transport, may affect the temperature distribution in the soil; it may also contribute to frost action, such as heave, depending on specific soil and temperature conditions. In general, the moisture transport may be of critical significance in soils containing more than 2% or 3% by weight of fine particles because of capillary action.

426. Once the thermal and soil regime has been established for the undisturbed soil, the proposed embankment is then considered. The physical properties of the material from which the embankment is to be constructed are estimated. The cross-sectional shape of the embankment, its orientation, and time of placement are fixed. The interrelationship of the impoundment embankment and permafrost foundation is then analyzed to determine the thermal changes which will take place in the permafrost sub-strata, in the active layer and within the embankment itself. This analysis may be done by either closed-form mathematical solutions or by computer simulation utilizing finite element or finite difference techniques (59, 60). The results of the analysis will be determination of thermal profiles in the combined embankment and foundation system and of the changes in these profiles as the system proceeds to thermal equilibrium. Potential settling, instability, diversion of groundwater flow, seepage of liquid wastes, and possible ice accumulation with resulting heaving in the soil sub-strata or in the embankment may be estimated from the changing thermal profiles coupled with the data obtained from the undisturbed soil. Figure 56 shows conceptually the effects of placing an embankment on a permafrost foundation. It will be noted that the permafrost table may progress into the embankment or may recede into the permafrost with a resulting thawed region beneath the embankment. Whether the permafrost table advances or recedes is dependent on the energy balance which takes place at the ground surface.

427. Where the waste embankment is constructed so that it remains permanently frozen, and in the absence of secondary problems such as erosion or seismic hazards, it is possible that steeper embankment slopes may be permitted than for a thawed embankment of the same material. Such a design, however, must be justified on the basis of a detailed thermal analysis plus stipulations for the time and methods of placing the embankment material.

428. Deep tailings ponds behind frozen embankments may include a location where thawing can occur into the embankment or into the permafrost beneath. The extent of this thawing and whether or not equilibrium will be reached is dependent on the heat source effect of the pond and the energy exchanges taking place to the atmosphere above and to the permafrost beneath. Such a condition and the resulting effects are shown conceptually in Fig 57.

429. The situation presented in Fig 56, case b, may be extended to indicate possible instability of the embankment slopes. The edges of the embankment have a lower insulating effect than the main structure. This permits depth of the active layer to increase under the side slopes. In cases where the underlying soil contains large ice masses, thawing will cause slumping to occur. Slumping may also occur where the soil is weak and in the thawed, unconsolidated state. Figure 58 diagrammatically describes this condition.

430. In cases where the active layer increases, it is necessary to evaluate the effect of thawing of the sub-strata. In freely draining coarse-grained soils which have no masses of embedded ice, there will be no subsidence related to permafrost. Poorly draining coarse-grained soils may not present a hazard provided that provision is made for drainage. Consolidation of this soil type may be necessary prior to placing of the embankment. Fine-grained soils with greater than 2% or 3% of fine particles by weight usually contain masses of ice. Massive non-uniform subsidence may

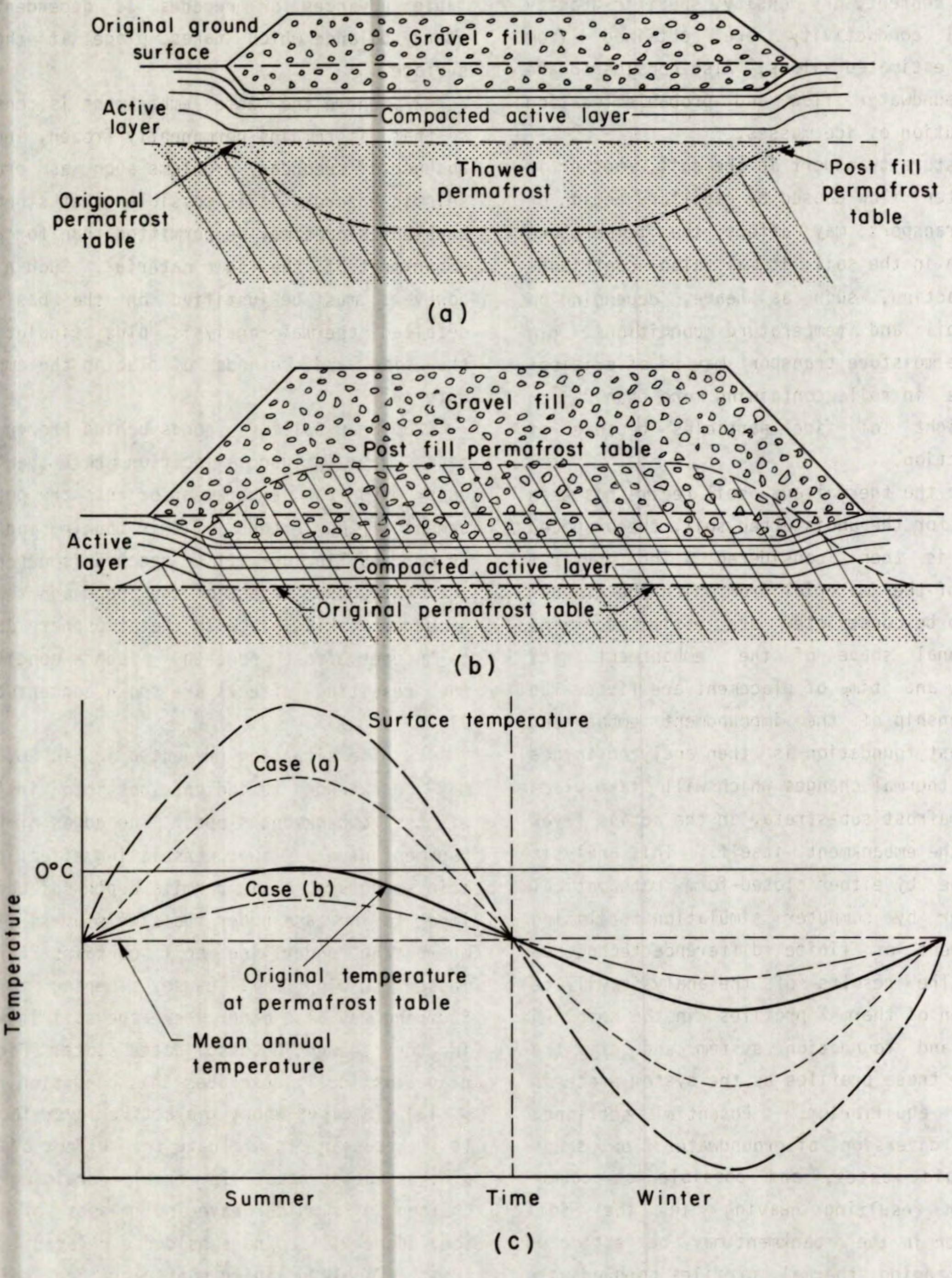


Fig 56 - Influence of waste embankment on a permafrost foundation

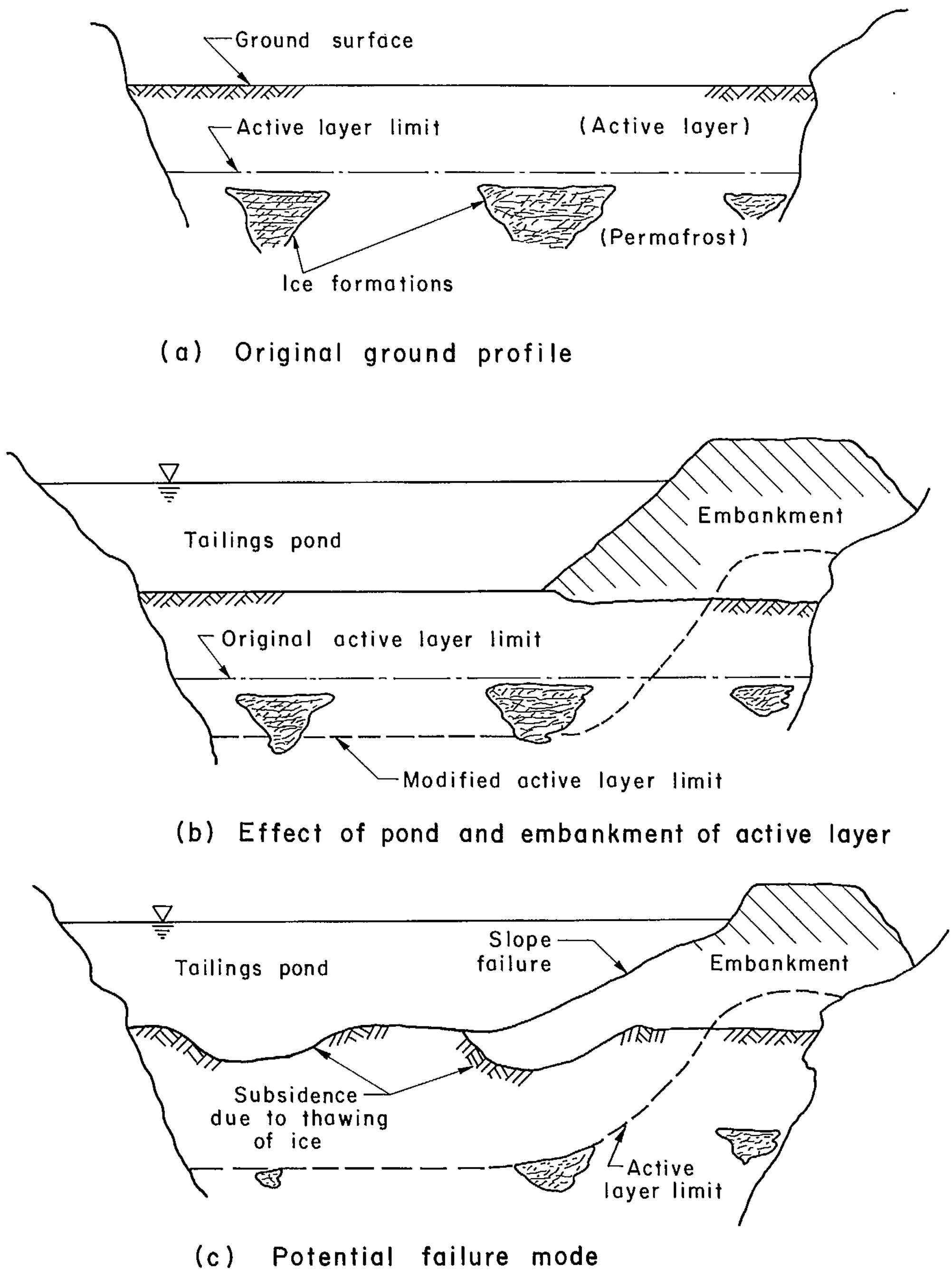


Fig 57 - Possible influence of tailings pond on permafrost and embankment stability

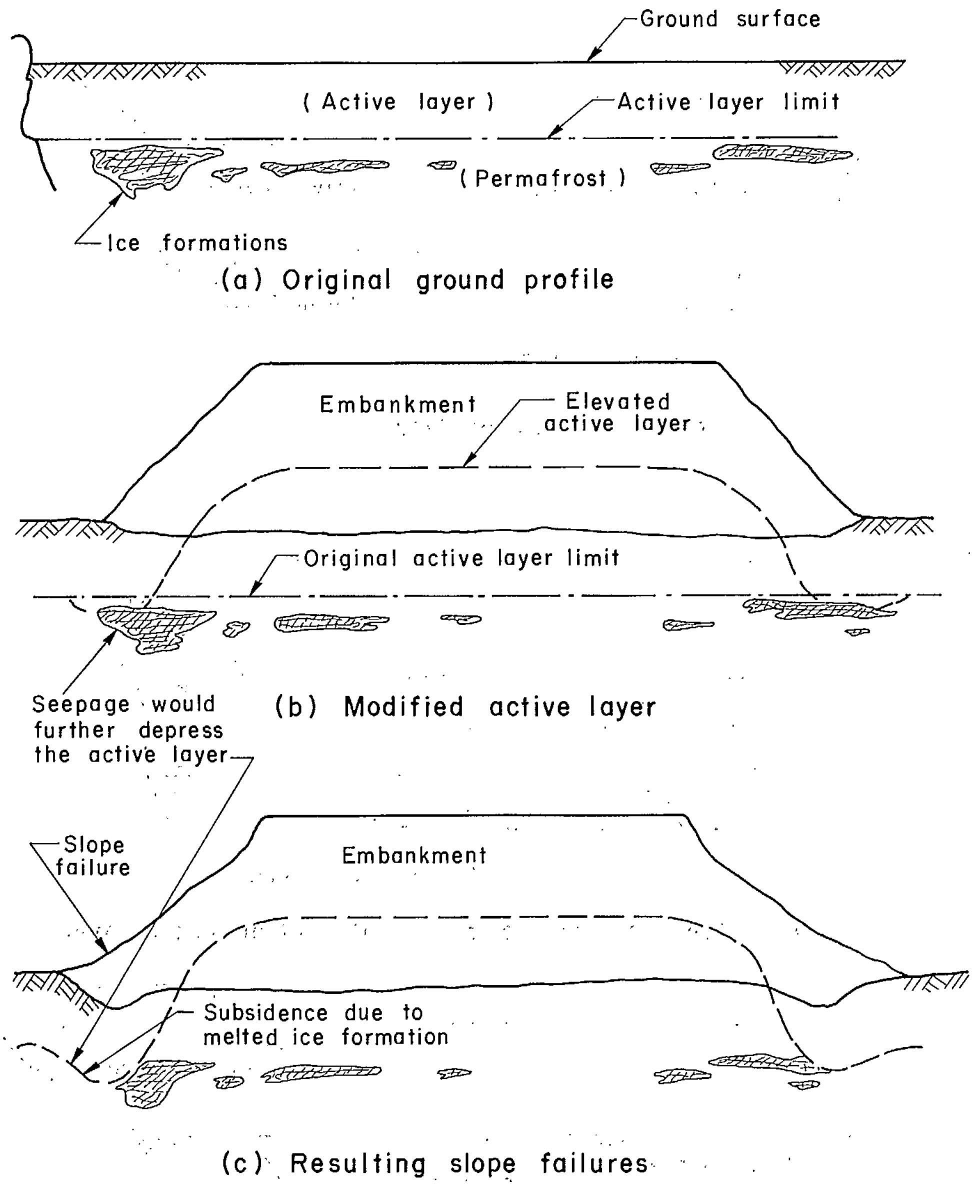


Fig 58 - Possible instability due to sub-surface thawing

take place in such soils if they are thawed. Should thawing occur, instability could result if the design were not based on thawed soils. An additional failure mode in these soils would be the result of frost heaving caused by the accumulation of ice beneath the structure. Soil liquefaction may also be significant in loose, saturated fine-grained soils in areas subject to seismic activity or near large mine blasts.

431. Additional problems may be encountered if mine waste embankments are to be placed on frozen hillsides. Slopes of poorly consolidated frozen soil may experience frozen solifluction (a slow movement of saturated non-frozen earth material behaving as a viscous mass over a surface of frozen material). Thawing of this soil will

result in acceleration of the solifluction process.

432. Where the embankment is placed on a hillside and seepage of groundwater takes place above the permafrost, or in valleys where artesian water lies in a thawed strata below a frozen surface, the potential for ice glaciering and "aufeis" formation exists. In ice glaciering the presence of an embankment structure provides a path of higher thermal conductivity than existed in the natural state. This allows the soil under the embankment to lose heat more rapidly and to freeze down to

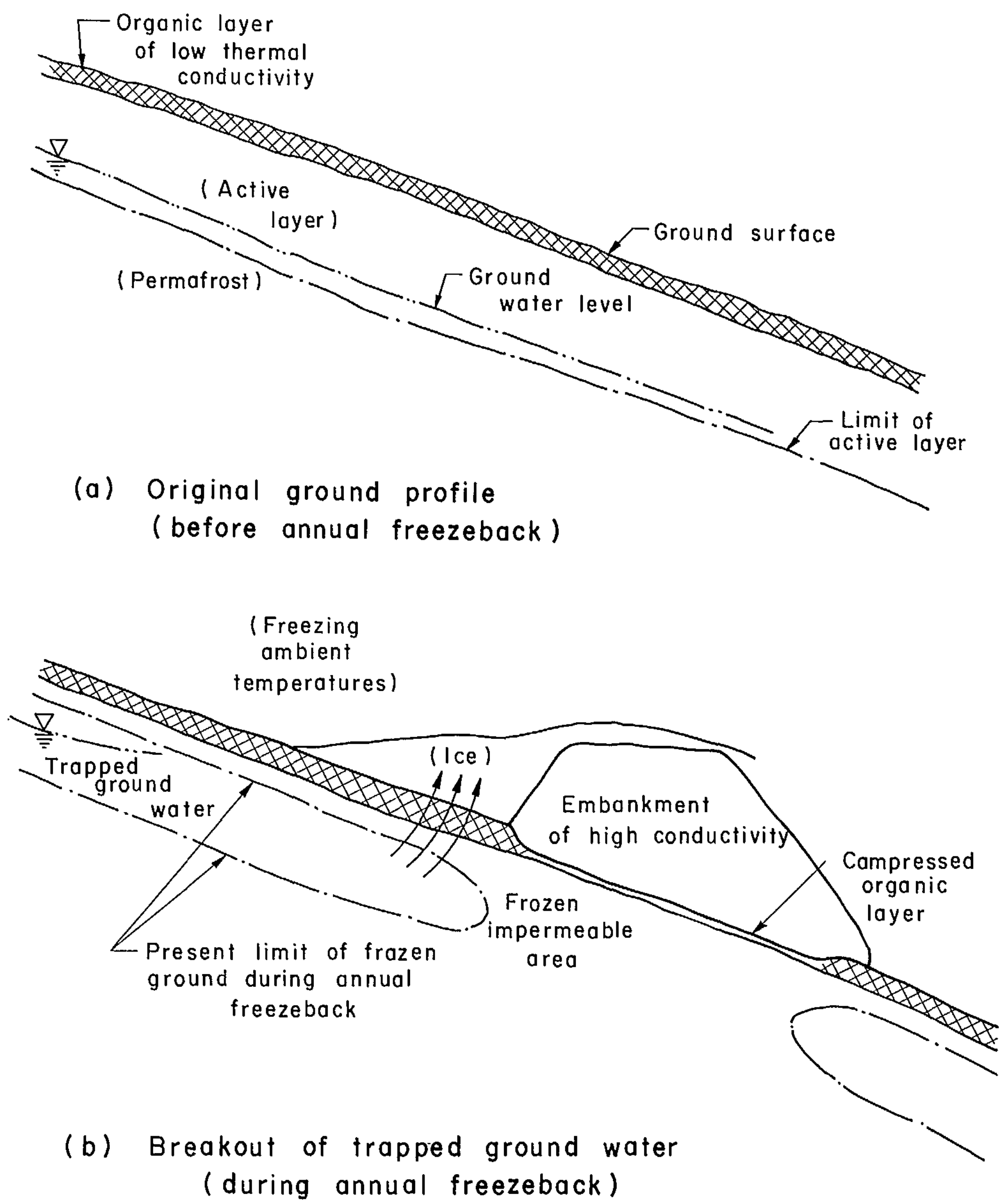
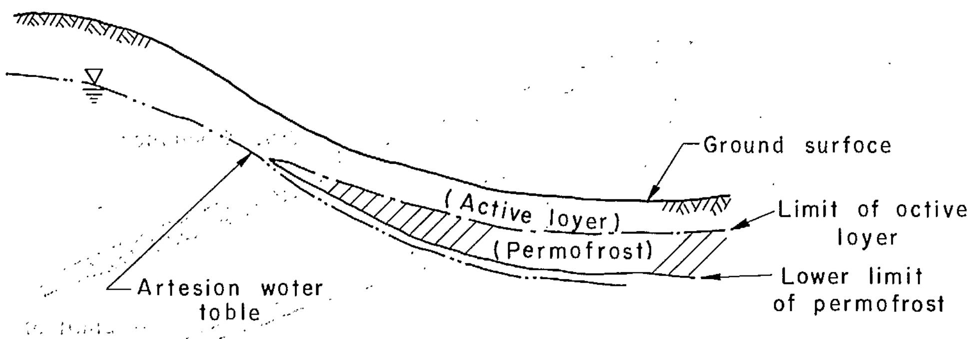
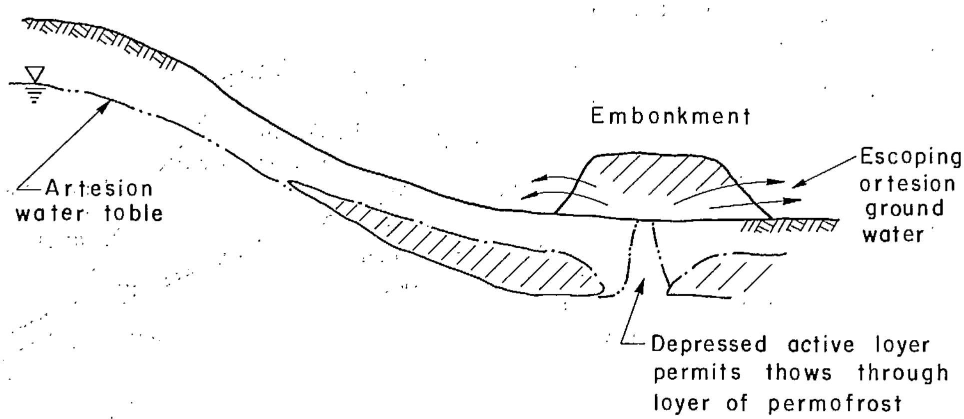


Fig 59 - Ice glaciering

the permafrost table before the adjacent uphill and downhill portions. The impermeable frozen area blocks the groundwater flow, causing water under hydrostatic pressure to break out and flow over the embankment. This condition is shown in Fig 59. "Aufeis" occurs when an embankment or structure effectively insulates an area, retarding the annual freezeback, thereby providing a permeable path for escape of artesian groundwater. It may also occur when artesian water is contained beneath an impervious frozen layer. This is not uncommon in areas of discontinuous permafrost. If an embankment causes active layer penetration to increase, thaw penetration may permit water to flow to the surface, resulting in icing at the surface during freezing temperatures (Fig 60). While "aufeis" and glaciering present no hazard to embankment security in themselves, the general in-



(a) Original ground profile



(b) Disturbed condition

Fig 60 - "Aufeis" conditions

convenience, coupled with a real hazard to any physical structures in the area, make them undesirable phenomena.

RECLAMATION

433. The process of reclamation essentially involves restoring the long term stability of the system and attempting re-vegetation. Vegetation frequently has important implications for increasing stability of the land form. One of the main controlling features of stability is the angle of repose. This depends on the nature, structure and size of particle forming the slope. Freeze-thaw phenomena in the active layer tend to decrease the angle of repose. Thawing of the winter ice in the active layer also may create problems in finer waste material or tailings since it is possible for layers to become over-saturated. In waste of low porosity, water may be impounded and this could lead to slumping, or "glides". It is to be expected that solifluction phenomena would be frequent and related to the movement of water through fine waste.

434. Liquefaction may also present a potential problem in seismic areas or where frequent large-scale blasts occur. These problems would not be encountered with coarse waste. In large waste deposits there may be thermo-karst problems - sinking of the ground surface due to degradation of permafrost. The causes of this could be a more positive heat budget related to greater heat input from radiative and convective sources, and higher peat permeability. The usual extent of thermo-karst phenomena is quite limited, being restricted to depressions of several yards across. In many instances, however, the heaping of material will mean a greater heat loss and would elevate the permafrost within the core, which is observed in road construction and natural mounds. However, heat budgets of these systems are complex and related to the local environment.

435. Proper design will minimize the problem of waste creep or slides. In certain instances the nature of the waste pile and its thermal regime may preclude the use of heavy equipment with high bearing loads.

436. The angles of slopes at which vegetation will grow are much lower than those regarded as physically stable. All mine wastes have definite characteristics with relation to acidity and content of organic material and heavy metals. Additional contamination of various kinds may have occurred during extraction and processing. Limestone is frequently used to neutralize acid waste. The cost of limestone for this purpose could become prohibitive in northern regions. Experimentation with overburden rich in limestone would be an alternative. Many wastes are poor in essential nutrients so that some fertilizer and organic material may be required. Organic material may be available locally in swamps. Careful pre-planning for segregation of top soil during overburden removal may be necessary to provide organic matter.

437. The choice of plant species is dependent on the final soil condition as well as geographical position. Many species in northern temperate regions are unable to survive Arctic conditions and species of high soil and climatic tolerance should be considered. Certain varieties of bromegrass (Manchar), red fescue and reed canary grass are frequently selected. Northern strains of Kentucky bluegrass will thrive except in the northern Arctic. Recently, strains of clover and alfalfa have been developed which tolerate permafrost conditions and also supply nitrogen to the soil. The planting or seeding of natural vegetation is frequently used. However, success in revegetating may take several years before a continuous cover is established, and a waste embankment could be subject to much erosion in that time. Additional information can be found in Supplement 10-1.

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APPENDIX A

ESTIMATING RUNOFF

ANNUAL AND SEASONAL RUNOFF VOLUMES

1. Forecasting runoff volumes for dam design is feasible. It is feasible also to estimate the probable recurrence interval of high precipitation values where long-term precipitation records are available. Methods of making such frequency analyses are described in references A-1, A-2, A-3, and A-4.

2. 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. To be reliable, such a precipitation runoff relationship would have to be determined for a catchment basin having characteristics similar to that being 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.

3. Snow surveys in the catchment basin are sometimes used for forecasting 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 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

4. 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 - amount, intensity and duration. For small catchment areas of a few square miles (several square kilometers), the areal distribution of storm rainfall can be assumed to be uniform. 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 showing precipitation amounts across Canada, as shown in Fig A-1. To obtain the corresponding rainfall for a 100-year return period multiply by 1.19. Figure A-2 gives information for the 100-year rainfalls in the USA, which may be useful for areas adjacent to the border. Where sufficient records of annual maximum rainfall for a specified duration are available, the maximum rainfall can be estimated by statistical analysis of the records. For most practical purposes, Fig A-1 can be used for determining the flood precipitation to use in design.

5. Studies have indicated that a fairly definite quantity of water lost by infiltration is required to satisfy initial field moisture deficiencies before runoff will occur, the amount of loss depending on 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 in. (5 cm) during dry summer and autumn months.

6. The initial loss for conditions usually preceding major floods in humid regions normally ranges from about 0.2 to 0.5 in. (0.5 to 1.3 cm) 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 only approximately.

7. 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. The most common value computed for a number of drainage basins in the Northeastern United States and considered to indicate approximately the minimum indices to be expected during major storms, is 0.05 in. (0.13 cm) per hour with some areas showing less than 0.02 in. (0.05 cm) and others more than 0.25 in. (0.6 cm).

8. A unit hydrograph is one representing one inch of direct runoff from a rainfall of some unit duration, say over six hours, and specific areal distribution. The basic premise implies that rainfall in excess of 2 in. (5 cm) 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 rainfall, or rainfall excess, that results in a unit hydrograph. The unit

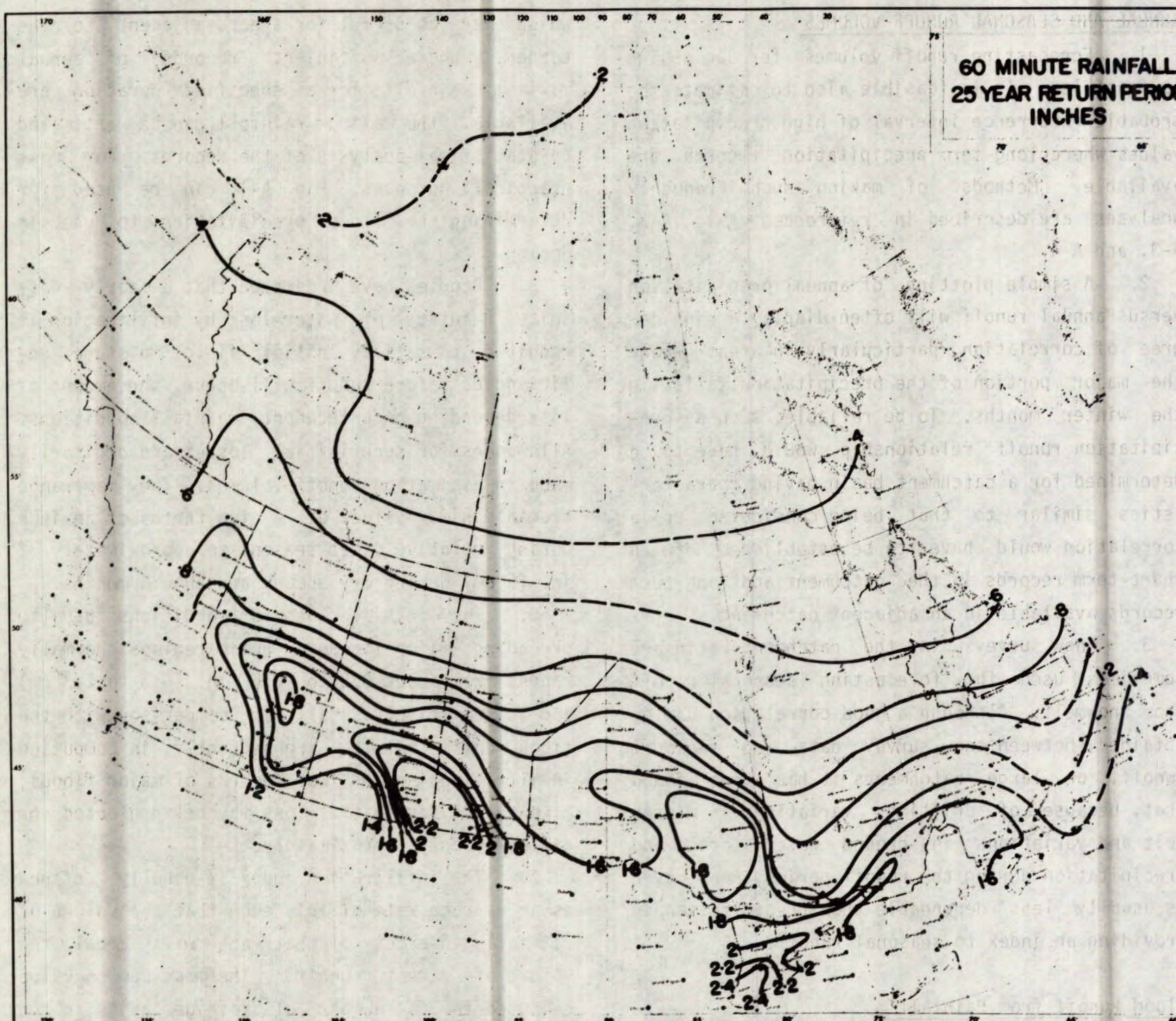
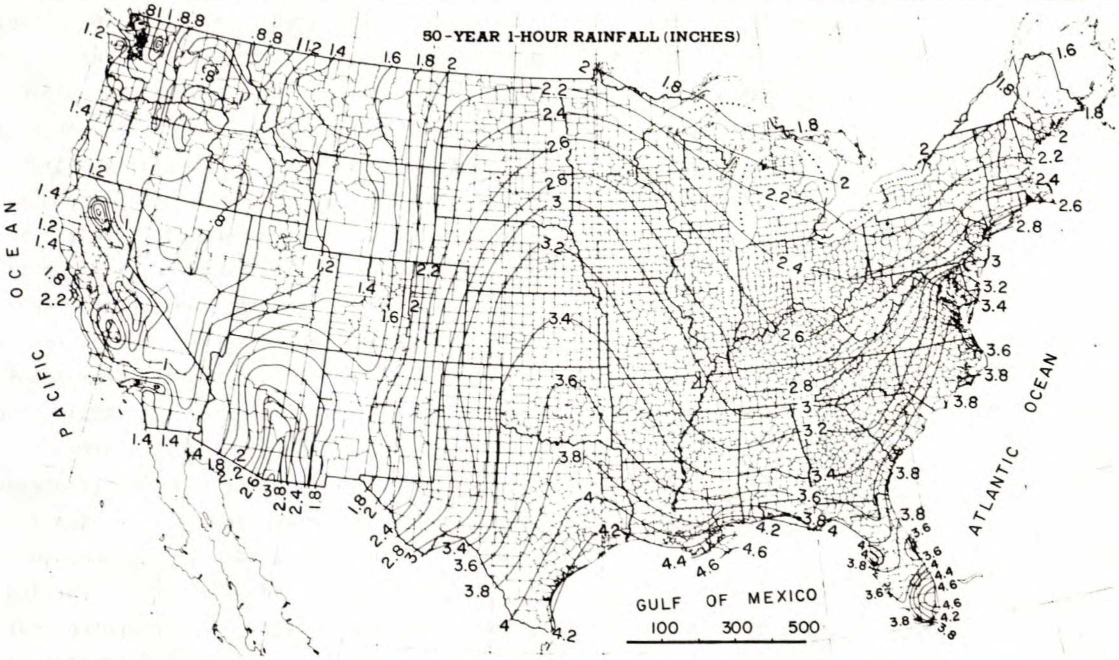


Fig A-1 - Twenty-five year return period for sixty-minute rainfall

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 determining the unit hydrograph should not exceed about half the lag.

9. Two basic methods are used for developing unit hydrographs. Analysis of rainfall-runoff records are made for isolated storms occurring over the catchment basin. This requires rainfall and stream-flow records for the actual catchment basin. Alternatively, computation of synthetic unit hydrographs are made 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. These empirical relations have proven particularly useful in the study of runoff



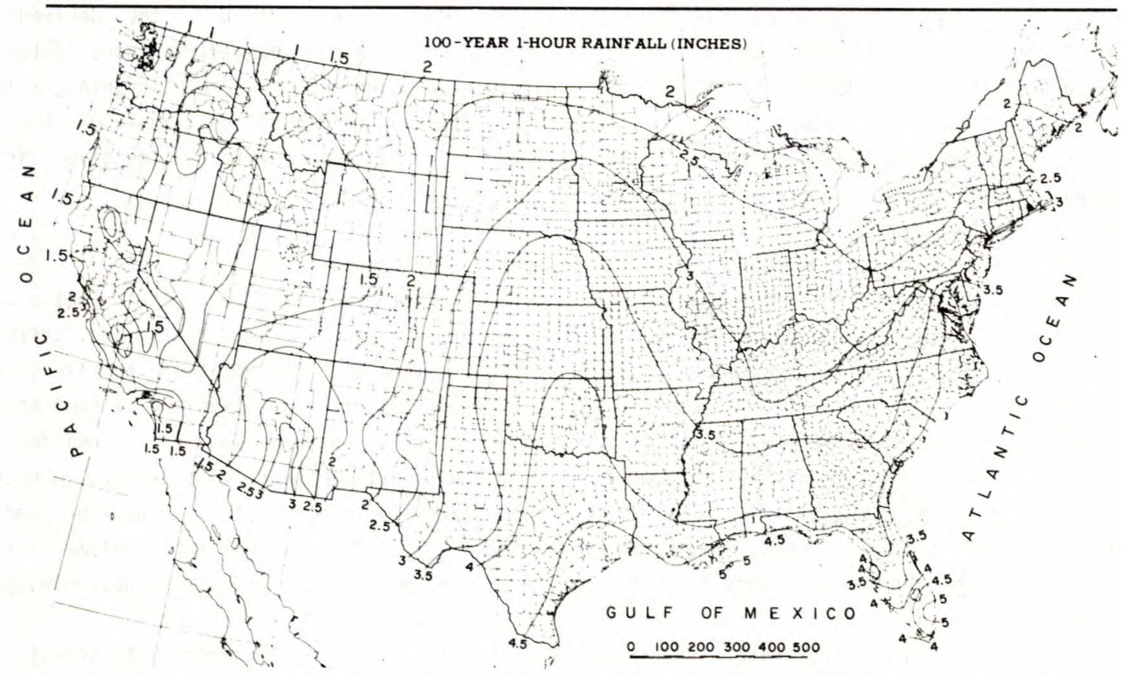


Fig A-2 - Fifty- and one hundred-year return period for sixty-minute rainfall in continental U.S.A.

characteristics of drainage areas where stream flow records are not available. The basic equations (after F.F. Snyder) for deriving a synthetic unit hydrograph by this method are as follows (A-1):

$$t_p = C_t \, (LL_{ca})^{0.3} \qquad \text{eq A-1}$$

$$t_r = t_p / 5.5 \qquad \text{eq A-2}$$

$$q_p = 640 \, C_p / t_p \qquad \text{eq A-3}$$

$$t_{pR} = t_p + 0.25 \, (t_R - t_r) \qquad \text{eq A-4}$$

$$q_{pR} = 640 \, C_p / t_{pR} = q_p t_p / t_{pR} \qquad \text{eq A-5}$$

$$Q_p = q_p A \qquad \text{eq A-6}$$

Where t_p is the lag time of t_r unit hydrograph in hours, t_r is the unit-rainfall duration in hours, t_R is the unit-rainfall duration other than standard unit t_r in hours, t_{pR} is the lag time of t_R unit hydrograph in hours, q_p is the peak discharge rate of the t_r unit hydrograph in cfs/sq mile, q_{pR} is the peak discharge rate of the t_R unit hydrograph in cfs/sq mile, Q_p is the discharge rate of t_r unit hydrograph in cfs, A is the drainage area in square miles, L_{ca} is the stream mileage from site to centre of gravity of the drainage area, L is the stream mileage from site to upstream limits of the drainage area, and C_t and C_p are coefficients depending upon units and basin characteristics (the range of 640 C_p is 200 - 600 with an average of 400, the range C_t is 8.0 - 0.4 with an average of 2.0).

10. The general procedure should be to: (a) analyze hydrologically for the drainage area to determine approximately the peak discharge, lag and general shape of unit hydrographs; fragmentary hydrological data that are not adequate for unit hydrograph-derivation in the usual manner may be very useful in connection with synthetic analyses; (b) evaluate, if adequate hydrological records are available, 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; (c) estimate, by general comparison of the runoff characteristics involved, whether the unit hydrograph peak discharge values computed for the particular area are consistent with values for comparable basins.

11. Studies have been made to determine the degree of accuracy inherent in the use of unit hydrographs derived from records of minor floods. These studies indicated that unit hydrographs for major floods had peak discharge ordinates consistently higher than those computed from records of minor floods. 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 in.) were 25% to 50% higher than values computed from records of minor floods (1-2 in.). 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.

12. 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 Fig A-3 are given in the references (A-1).

Runoff from Snowmelt

13. Unlike rainfall, snowmelt is not a quantity that can be measured directly and, therefore, must be estimated. In relation to flood hydrograph analysis, this involves the determination of snowmelt rates under various conditions of terrain, vegetal cover and weather. Secondly, evaluating the effect of the snowpack on runoff is necessary. Predicting the total volume of runoff during the melt season requires determining the water equivalent of the snowpack.

14. Snowmelt results from heat transfer involving radiation, convection and conduction. The relative importance of each of these processes is highly variable, depending on conditions of weather and local environment. The natural sources of heat in melting snow are: solar

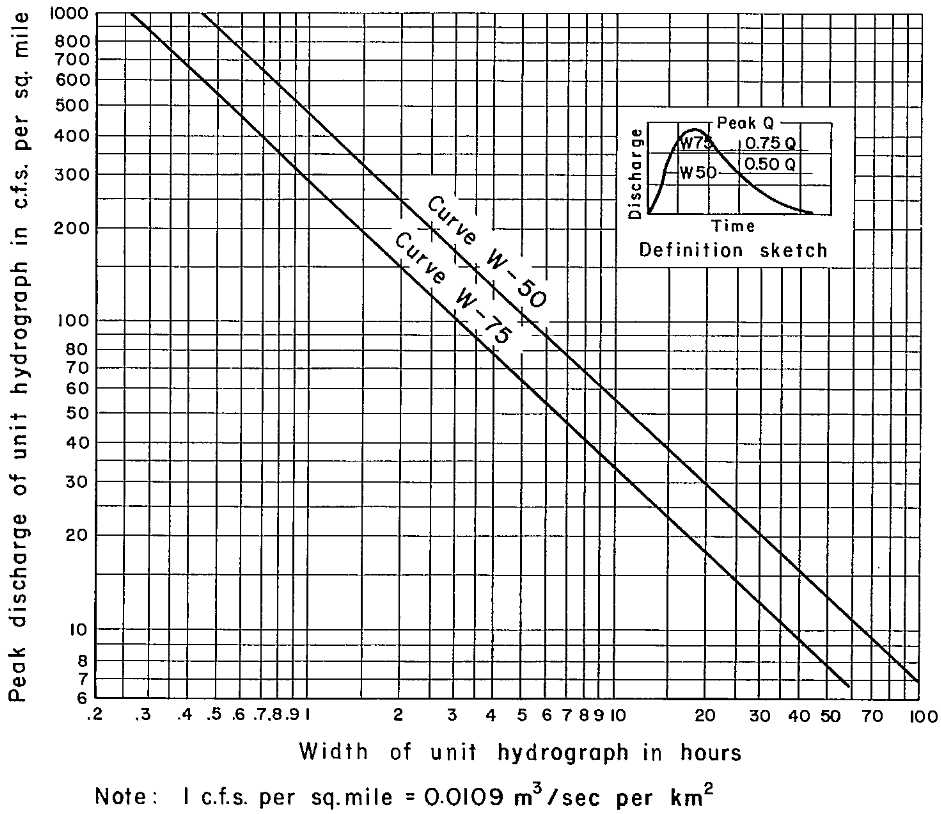


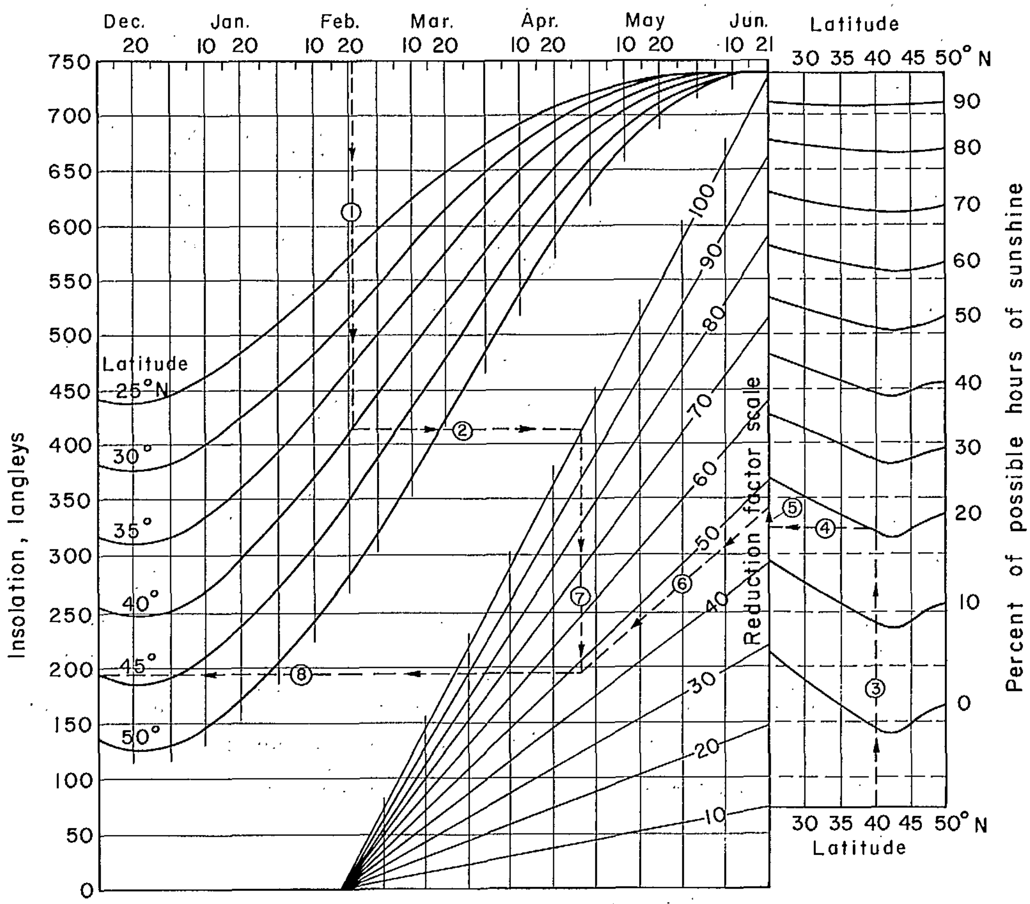
Fig A-3 - Unit hydrograph peaks vs widths

radiation, terrestrial radiation, convection heat transfer from the air, latent heat of vaporization by condensation from the air, conduction of heat from the ground (usually negligible), and heat from rain water.

15. The amount of heat transferred to the snowpack 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 are caused by clouds. It is usually necessary to estimate incoming radiation 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-2). A nomograph for estimating incoming solar radiation at latitudes below 50°N is shown on Fig A-4. Graphs showing variation with season and latitude of solar radiation outside the earth's atmosphere, and the seasonal variation of incident radiation on north-south facing slopes, are included on Fig A-5.

16. 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% for new fallen snow to as little as 40% for late season snow.

17. 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. The melt coefficients represent the actual melt of the snowpack, expressed as daily ablation in inches (cm) 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% free water content). The equations are as follows



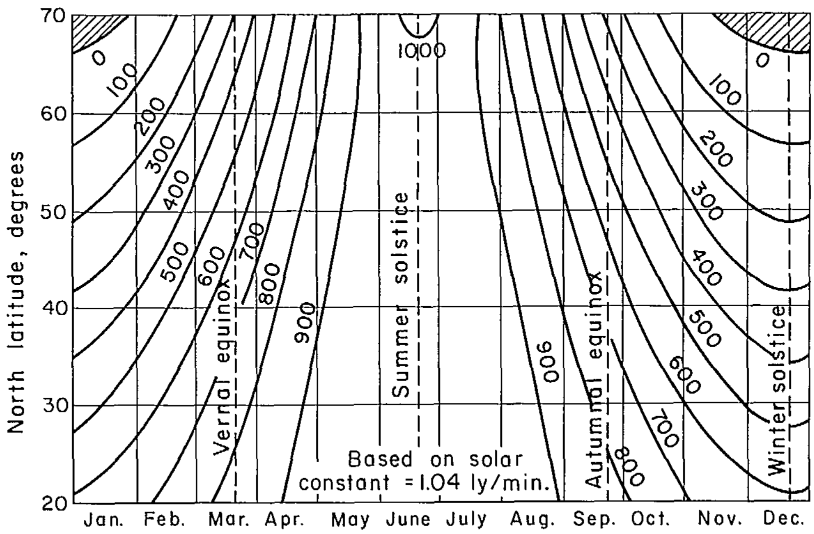
NOTES:

1. The sample shown by dashed lines estimates the daily total insolation at latitude 40°N on February 21, with 20 per cent possible sunshine. Consecutive steps are numbered. Step 4 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.

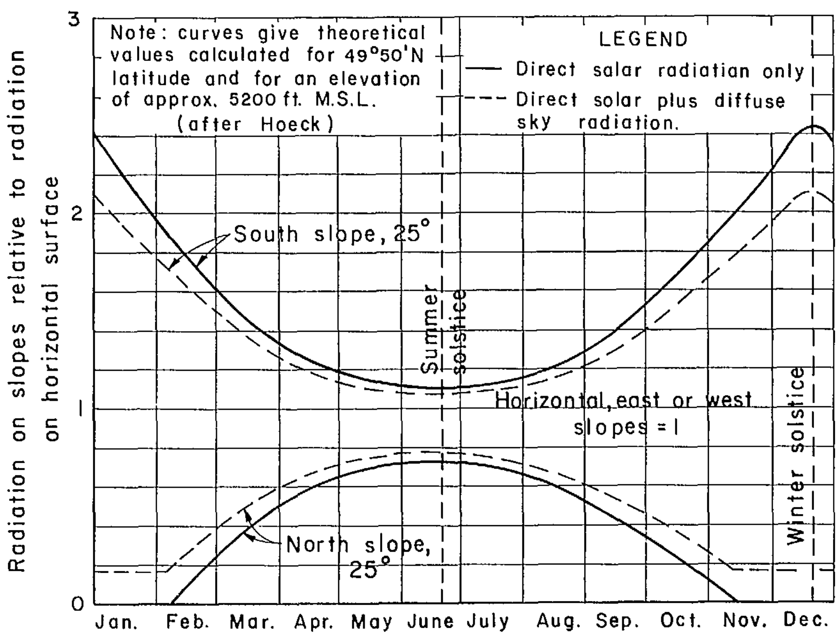
SEASONAL CORRECTION TO REDUCTION FACTOR

	Per cent 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

Fig A-4 - Solar radiation vs latitude and sunshine



DAILY RADIATION OUTSIDE EARTH'S ATMOSPHERE
(Langleys)



ON NORTH AND SOUTH SLOPES

Fig A-5 - Solar radiation. Top: Daily radiation outside earth's atmosphere; Bottom: Clear sky radiation received on north and south slopes

(U.S. Corps of Engineers):

Heavily forested area: (> 80%)

$$M = 0.074(0.53T'_a + 0.47T'_d) \qquad \text{eq A-7}$$

Forested area: (60-80%)

$$M = k(0.0084v)(0.22T'_a + 0.78T'_d) + 0.029T'_a \qquad \text{eq A-8}$$

Partly forested area: (10-60%)

$$M = k'(1-F)(0.0040I_i)(1-a) + k(0.0084v)(0.22T'_a + 0.78T'_d) + F(0.029T'_a) \qquad \text{eq A-9}$$

Open area: (< 10%)

$$M = k'(0.00508I_i)(1-a)+(1-N)(0.0212T'_a-0.84) + N(0.029T'_c)+k(0.0084v)(0.22T'_a+0.78T'_d) \qquad \text{eq A-10}$$

where M is the snowmelt rate in in./day, T'_a is the difference between air and snow surface temperatures in °F, T'_d is the difference between dewpoint and surface tempertures in °F, v is the wind speed in open areas in miles/hour, I_i is the solar radiation on horizontal surface (insolation) in langleys, a is the average snow surface albedo, k' is the basin shortwave radiation melt factor (Fig A-5; it would be 1.0 for a basin essentially horizontal or whose north and south slopes are areally balanced, and usually falls within the limits of 0.9 and 1.1 during spring), F is the average basin forest cover, expressed as a decimal fraction, T'_c is the difference between cloud base and snow surface temperatures in °F [air temperature drop of 3-5°F/1000 ft elevation; where cloud base is less than 1000 ft (300 m), its temperature can be assumed equal to surface air temperature], N is the cloud cover, expressed as a decimal fraction, and k is the basin convection-condensation melt factor.

18. For clear weather in spring, heat exchange by air turbulence is of secondary importance to radiation. In winter during rainfall, however, turbulent exchange is the dominant heat exchange process. It involves the transfer of sensible heat from warm air over the snowfield and also the heat of condensation of water vapour from the atmosphere condensing on the snow surfaces.

19. For this reason, melting due to solar radiation during rainstorms is relatively small and the basic equations can be simplified by assuming an average rate. The simplified equation for estimating snowmelt during rain are as follows (A-2):

Heavily forested areas: (> 80%)

$$M = (0.074 + 0.007\ P_r)\ (Ta - 32) + 0.05 \qquad \text{eq A-11}$$

Open or partly forested areas: (< 60%)

$$M = (0.029 + 0.0084\ kv + 0.007\ P_r)\ (T_a - 32) + 0.09 \qquad \text{eq A-12}$$

$$W_c = \rho D T_s/160 \qquad \text{eq A-13}$$

where M is the snowmelt rate in in./day, T_a is the mean temperature of saturated air in °F, v is the mean wind speed in miles/hour (for partly forested areas, wind values should be those representative of the open portions of the basin), P_r is the rate of rainfall in in./day, k is the basin convection-condensation melt factor (this allows for basin exposure to wind and would be 1.0 for unforested plains, but could be as low as 0.3 for densely forested areas), W_c is the cold content equivalent in in. of liquid water, ρ is the snow density in grams/cubic centimetre, D is the snowpack in in., and T_s is the average snowpack temperature deficit below 0°C.

20. Values of wind speed and temperature used in the equations should represent average conditions over the snow covered area of the basin. The reduction in wind speed in the forested portion of the basin is accounted for in the selection of the basin convection-condensation melt coefficient.

21. Runoff analysis for winter or early spring periods requires consideration of the storage effect of the snowpack. 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.

22. Observations have indicated that the liquid water-holding capacity of snowpacks generally falls within the range of 2%-5% of the total water equivalent. For an initially cold (sub-freezing) snowpack, the equivalent cold content (in. 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 simplified equations A-11, A-12 and A-13. 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% and being as much as 10% during the day due to melt water in transit.

23. The time delay for runoff in 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.

24. In snow hydrology, it is assumed that no direct runoff occurs until the soil storage is filled to capacity. For typical mountain soils, the theoretical maximum storage capacity ranges from 4 to 8 in. (10 to 20 cm) 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.

25. 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% of the water equivalent of the snowpack.

26. Loss by evaporation from the snowpack itself is usually small, observations in the United States indicating it to average less than 0.5 in. (1.0 cm) of water per month, during the winter and early spring. During late spring there is usually condensation on the snow surface.

27. Groundwater 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% of the snowmelt.

28. 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.

SIMPLIFIED SPILLWAY DETERMINATION

29. The above analyses are appropriate for dams representing a considerable investment, which in many cases will include tailings dams. However, a relatively simple approach to determining the volume of flood water to be handled will be appropriate for a tailings basin on many mining properties. A spillway or overflow weir must have sufficient capacity to pass the peak runoff plus the maximum volume of process and mine water.

30. The peak runoff is calculated from the following equation:

$$Q = CiA \qquad \text{eq A-14}$$

where Q is the runoff to be handled by the spillway in cfs, i is the average rainfall intensity in

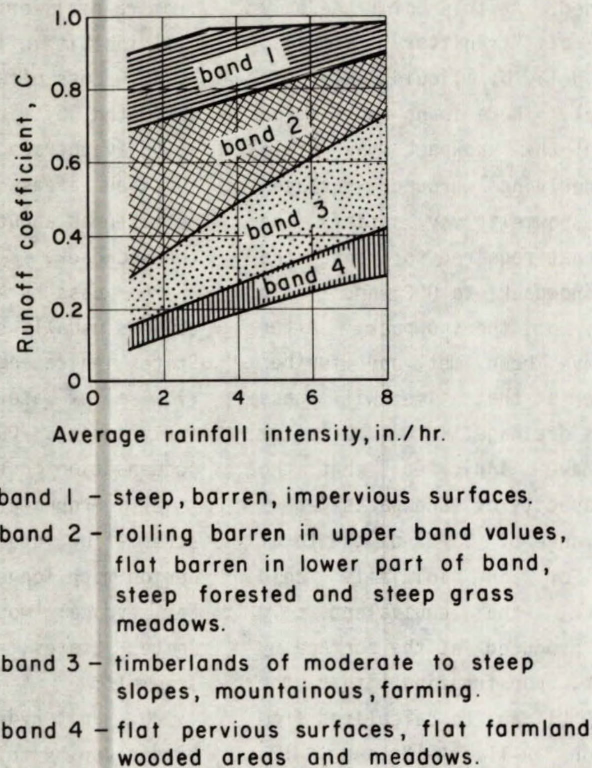


Fig A-6 - Relationship between runoff coefficient and rainfall intensity and topography

in. per hour of a 1-hour storm with a recurrence interval of 100 years obtained from Fig A-2, C is the runoff coefficient obtained from Fig A-6, and A is the area of the drainage basin in acres.

31. The broad-crested weir formula is as follows:

$$Q = 3.33\ (L - 0.2H)H^{3/2} \qquad \text{eq A-15}$$

where Q is the flow in cfs, L is the length of the weir and H is the head of water over the crest in feet.

Example: Suppose the average rainfall intensity, i, in a mining area is 1.87 in. and the area of drainage basin, A, is 165 acres, where the runoff coefficient, C, is estimated as 0.3. The mine dewatering produces a maximum of 110,000 gallons in 8 hours and mill tailings disposal is at a maximum rate of 590 tons/day of solids at a slurry/solids ratio of 33%. What length of the spillway would be required to handle the total water flow?

Solution: The peak runoff Q can be calculated from eq A-14:

$$\begin{aligned} Q &= Ci\ A \\ &= 0.3 \times 1.87 \times 165 \\ &= 92.56 \text{ cfs} \end{aligned}$$

The combined mine water plus mill water is less than 1 cfs (say 1 cfs). Assume the maximum desired head of water over the crest, H, is 1.5 ft, then the length of spillway, L, required to pass a flow of 93.56 cfs is obtained from eq A-15.

$$93.56 = 3.33\ (L - 0.2 \times 1.5) \times 1.5^{1.5}$$

Therefore L = 15.2, say 15.5 ft

Note: If SI units were used, eqs. A-14 and A-15 should be converted as follows:

$$Q = 0.0275\ CiA$$
$$Q = 1.8366\ (L - 0.2\ H)H^{3/2}$$

where Q is expressed in m^3/sec; i is in cm/hr; A is in ha; L and H are in metres.

Conversion factors: 1 cfs = 0.0283 m^3/sec,
1 acre = 0.4046 ha(4046 m^2),
1 in./hr = 2.54 cm/hr,
1 ft = 0.304 m.

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GLOSSARY

ANGLE OF INTERNAL FRICTION, ϕ

The maximum angle of obliquity between the normal and the resultant stress acting on a surface within a soil or rock.

ANGLE OF REPOSE

The angle with a horizontal plane at which loose material will stand on a horizontal base without sliding.

ATTERBERG LIMITS - see plasticity index.

CATCHMENT

An area designed to provide a sufficient width and length to catch runoff from the tailing ponds.

COEFFICIENT OF CONSOLIDATION, C_v

A coefficient indicating the rate of compression in soil under load.

COEFFICIENT OF INTERNAL FRICTION

The tangent of the angle of internal friction.

COEFFICIENT OF PERMEABILITY, k

The rate of discharge of water under laminar flow condition through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature (20°C).

COEFFICIENT OF UNIFORMITY, U

The ratio of the 60% passing size of the material to the 10% passing size of the material, $U = D_{60}/D_{10}$.

COHESION, C

The portion of the shear strength (s) indicated by the term c in Coulomb's equation: $s = c + \phi \tan \phi$, where ϕ is the angle of internal friction. It has the nature of an intergranular binding force.

COMPRESSION INDEX, C_c

An index indicating the compressibility of the soil, which represents the slope of the curve of void ratio versus logarithm of effective pressure.

CONSOLIDATION

The process of transient flow of water through a soil structure which compresses or expands in time is called consolidation in soil mechanics.

CREEP

An increase in plastic strain with time is usually called creep.

CREST

The top of an embankment slope.

CULVERT

Tunnel drain for water across the embankment.

CRITICAL SURFACE (CRITICAL FAILURE SURFACE)

The sliding surface for which the factor of safety is a minimum in an analysis of a soil slope or embankment in ductile ground when average stresses can be used.

DEFORMATION

The change in linear dimension of a body or the absolute movement of a point on a body.

DEGREE OF SATURATION, S

The ratio of the volume of water in the soil voids to the total volume of voids.

DENSITY (MASS DENSITY)

Mass per unit volume. Relative density - the density relative to its two limiting values.

DISPLACEMENT

The straight line distance between two points or positions.

DRAWDOWN

The vertical distance the free water elevation is lowered or the reduction of the pressure head due to removal of free water.

DECANT PIPE

Pipes buried in the tailings embankment for the purpose of water drainage.

EFFECTIVE STRESS - SEE STRESS

EQUIPOTENTIAL LINE

Line along which water will rise to the same elevation in piezometric tubes.

FACTOR OF SAFETY

The ratio of available shear strength to shear stress on the critical failure surface.

FINGER DRAINS

A drainage system which consists of strips of pervious drainage materials.

FILTERS

Well-graded material which is 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. It must have a gradation such that its voids are sufficiently small to prevent the passage of the fine soil particles from the protected soil.

FLOW LINE

The path that a particle follows under laminar flow conditions.

FLOW NET

A grid which is formed by the intersection of two sets of orthogonal lines. One set, the flow lines represents the loci of particles of liquid as they pass through the porous medium; the other set, known as equipotential lines are piezometric contours representing the loci of points having the same potential. The flow net is used to estimate the rate of seepage flow and to predict the piezometric pressure at any point within the embankment cross-section.

FREEBOARD

The height measured from water level to the crest of embankment. The minimum freeboard should be measured from the maximum projected flood level to the crest of the embankment.

GROUND WATER LEVEL

The level below which the pores and fissures of the rock and subsoil, down to indefinite depth, are full of water.

HYDRAULIC GRADIENT

The loss per unit distance of elevation head plus pressure head.

HYDROCYCLING

A procedure used for dam and embankment construction in which hydrocyclones or cyclones can be used to separate sands from the slime. A series of cyclones can be placed along the crest of the embankment or a group of cyclones can be mounted in parallel as a mobile unit which travels along the longitudinal axis of the dam.

LIQUID LIMIT, W_L

The water content at which the soil exhibits a small shearing strength is taken to be the boundary between liquid and plastic behaviour and this water content is called liquid limit.

LIQUIFACTION

A transformation in which the soil particles become temporarily suspended in the pore water and the soil mass suddenly acquires the property of a viscous liquid.

METHOD OF SLICES

A general procedure used for slope stability analysis in soil. A trial surface is chosen and the potential sliding mass is divided into a number of vertical slices. Each slice is acted on by its own weight which produces shearing and normal forces on its vertical boundaries and along its base.

METHOD OF INFINITE SLICES

A procedure involves a circular sliding surface for which the stability of the potential sliding mass is considered as a whole rather than stability of each individual slice.

MODULUS OF ELASTICITY (MODULUS OF DEFORMATION)

The slope of the tangent (hence tangent modulus) of a stress-strain curve. The use of the term modulus of elasticity is recommended for materials that deform in accordance with Hookes Law; the term modulus of deformation for materials that deform otherwise.

MOHR ENVELOPE (FAILURE ENVELOPE)

The envelope of a series of Mohr circles representing stress conditions at failure for a given material. According to Mohr's strength theory, a failure envelope is the locus of points the co-ordinates of which represent the combinations of normal and shear stresses that will cause a given material to fail.

NON-CIRCULAR SLIDING

The failure surface in a soil slope or embankment may follow a non-circular path.

OVERTOPPING

Water flowing over the crest of the tailings embankment.

PIEZOMETER

A device for measuring the hydrostatic pressure at a point in the ground. Simple piezometers are open holes for measuring the groundwater table.

PIPE DRAINS

A drainage system which consists of pipes in the embankment cross section.

PIPING

The result of errosion which starts at the point of exit of a flow line that has passed below or around the embankment or its foundation.

PLASTIC LIMIT, W_p

A boundary region of water content representing a change in characteristic of the soil from those of a plastic to those of brittle material. This water content is called the plastic limit.

PORE WATER PRESSURE, u

Stress transmitted through the pore water.

PERMEABILITY - SEE COEFFICIENT OF PERMEABILITY

PHREATIC SURFACE

The surface along which the pressure in the fluid equals atmospheric pressure.

PHOTOGRAMMETRIC MAPPING

The mapping of surface exposures or geological structures done by photogrammetric techniques.

RELIEF WELLS

Wells drilled for control pore water pressure beneath embankment or soil slope.

ROTATIONAL SHEAR SLIDE

A slide resulting from the yielding and redistribution of shear stresses in a soil so that a more or less circular surface of failure envelope develops before the cohesion breaks down and permits a comprehensive, circular sector of the slope to fail by rotating.

SHEAR FAILURE

Failure resulting from shear stresses.

SHEAR STRENGTH

The internal resistance offered to shear stress. It is measured by the maximum shear stress, based on original area of cross section, that can be sustained without failure. Peak shear strength - at a certain level of shear stress, the shear strength of the surfaces is exceeded and further displacement will take place without any further increase in shear stress. This limiting value defines the peak shear strength at that particular normal stress.

Residual shear strength - as the peak strength is exceeded, fracturing of interlocking projections on the surface occurs and the broken pieces are ground into detrital material as shear displacement continues. After a certain amount of displacement, the surfaces becomes slickensided and covered with gouge material and shear displacement takes place at a constant shear stress level. This shear stress is called residual shear strength.

SHRINKAGE LIMIT, W_s

The decrease in volume stops at the water content at which the surface of soil becomes lighter and this water content is known as the shrinkage limit.

SPECIFIC GRAVITY

The ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of 4°C.

SPIGOTTING

A procedure generally used in the upstream method of construction for dams or tailings embankments for which spigots are employed and the tailing slurry is discharged from a series of spigots along the crest of dam or embankment. The slurry meanders in a series of loose streams that result in discontinuous horizontal stratification.

STRESS

The force per unit area, as the area approaches zero, acting within a body.

Effective stress - The average normal force per unit area transmitted from grain to grain in a granular mass. It is the stress that is effective in mobilizing internal friction.

Principal stresses - Stresses acting normal to three mutually perpendicular planes, intersecting at a point in a body, on which no shear stresses act.

Shear stress (Shearing stress) - The stress component tangential to a given plane.

Total stress - The total force per unit area acting within a granular mass. It is the sum of neutral and effective stresses.

TAILINGS

The waste product from a milling operation in which the valuable minerals have been recovered.

THIXOTROPY

When some materials are kneaded without altering the water content, cohesion decreases considerably. This effect is known as thixotropy.

UNIT WEIGHT

Weight per unit volume.

Dry unit weight - the unit weight that, when multiplied by the height of the overlying column of ground, yields the effective pressure due to the weight of overburden.

Saturated unit weight - The wet unit weight of a granular mass when saturated.

Submerged unit weight - The weight of solids in air minus the weight of water displaced by the solids per unit volume of mass

VOID RATIO, e

The ratio of void volume to solid in a soil mass.

WASTE EMBANKMENTS

Refers to all mine waste materials placed on surface but excludes those materials placed underground as backfill.

WASTE PILES

Refers to all waste embankments other than tailings or ore, resulting from a mining operation.

SYMBOLS

- A - Amplitude of the temperature swing
- b - the width of a slice
  - the time phase term for which the temperature may be related to the first day of the year
- C - cohesion of the material
- C_c - compression index
- C_v - coefficient of consolidation
- C' - effective cohesion of the material
- D - total depth of flow
- D_r - relative density
- D_{10}, D_{60} - the diameter of the 10% passing size of the material and the diameter of the 60% passing size of the material respectively
- D_{15_F}, D_{15_B} - the diameter of the 15% passing size of the material; suffix F refers to filter material and suffix B refers to protected material
- e - the void ratio of the soil as it exists
- e_{max} - the void ratio of the soil in its loosest state
- e_{min} - the void ratio of the soil in its densest state
- f - frequency, cycle per year
- FS - factor of safety
- G_s - specific gravity of the soil particles
- h - stope height or height of the waste embankment or pile
  - the difference in piezometric head between the point of seepage entry and the point of seepage exit
- i - slope angle of the foundation
- I_p - plasticity index
- k - coefficient of permeability
  - stone diameter
- L_w - liquid limit
- n - porosity
- n_d - the number of equipotential drops determined from the flow net
- n_f - the number of flow paths determined from the flow net
- N - number of blows for standard penetration test
- N' - effective normal force acting on the base of a slice
- p' - total stress normal to the plane of failure
- $\bar{p}$ - effective normal stress = $(p - u)$
- q - flow of water
  - rate of seepage per unit length perpendicular to the plane of the flow net
- S - shear strength
  - degree of saturation
- t - the day of the year expressed as a fraction
- t_m - the mean annual soil surface temperature
- t_s - the soil surface temperature
- u - pore water pressure on the failure plane
- U - coefficient of uniformity
- $\bar{V}_c$ - average velocity against channel
- $\bar{V}_s$ - average velocity against stone
- W - water content when saturated in percent dry weight
  - the crest width of embankment
- Z - height of the embankment crest above the foundation at its lowest point in ft
- α - the angle of inclination at the centre of the base of the slice
  - the thermal diffusivity
- β - slope angle
- γ - unit weight in saturated state
- γ_d - unit weight in dry state
- γ_w - unit weight of water
- δ - the friction angle between the base of the pile and its foundation
- ϕ - angle of internal friction
- ϕ' - effective angle of internal friction
- η - the time of the year under consideration