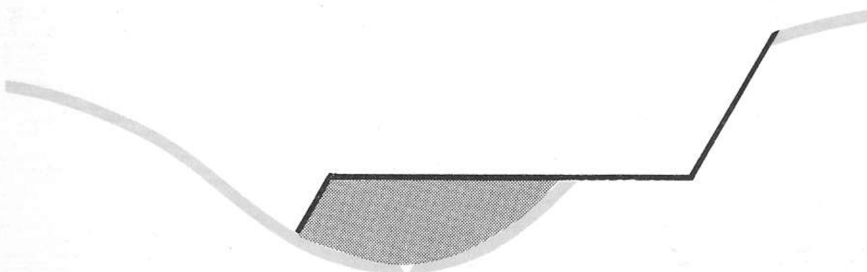
PROCEEDINGS of the
INTERNATIONAL SYMPOSIUM on

FLOW-THROUGH ROCK DRAINS



Convened at Inn of the South,
Cranbrook, British Columbia, Canada
September 8-11, 1986

THE INTERNATIONAL SYMPOSIUM ON FLOW-THROUGH ROCK DRAINS

Organized by the Technical and Research Committee on Reclamation with support from:

Canada/British Columbia Mineral Development Agreement
British Columbia Ministry of Energy, Mines and Petroleum Resources
Mining Association of British Columbia
British Columbia Ministry of Environment
CrowsNest Branch of Canadian Institute of Mining
Byron Creek Collieries
CrowsNest Line Creek
Fording Coal
Westar Mining

Symposium proceedings were assembled and edited by Grant Symons,
#7 - 210 Old Island Highway, Victoria, B.C.

Printing history
200 Printed-February 1987

CONTENTS

THE TECHNICAL AND RESEARCH COMMITTEE ON RECLAMATION

The Technical and Research Committee on Reclamation originated and first became active in the early 1970s in response to a demonstrated need for greater government - industry communication in the area of environmental protection and reclamation. Membership (see Section entitled 'Organization of Symposium' for names of members) is drawn from the corporate sector (several of the larger surface mines are represented); the Ministry of Energy, Mines and Petroleum Resources; the Mining Association of British Columbia; the University of British Columbia; the University of Victoria; and the Ministry of Environment and Parks. The Committee meets four or five times a year to discuss matters of joint concern and interest, exchange experience, and plan activities.

Since 1975, the Technical and Research Committee on Reclamation has annually sponsored the British Columbia Mine Reclamation Symposium and Awards. This event draws contributions from all disciplines engaged in the mine reclamation field and attendance from across Canada and the United States.

The proceedings of the symposium represent a record of mine reclamation achievement in British Columbia plus a state-of-the-art assessment of current practice.

In 1985, the committee sponsored a preliminary study of the practice of resloping waste dump faces, with support from the Canada-British Columbia Mineral Development Agreement.

In 1986, the Committee sponsored this International Rock Drain Symposium which draws attendance from the United States, France and Australia, again with support from the Canada-British Columbia Mineral Development Agreement.

Mr. A.W. "Tony" Milligan of Westar Mining Ltd., is the 1986/87 Chairman.

THE PROBLEM BEING EXAMINED AT THE SYMPOSIUM

A number of surface mines in mountainous regions of Western Canada are being forced by topographic constraints to dump large quantities of waste rock in valley fills. As stream diversions around dumps are often very costly, flow-through rock drains can be economical alternatives for conveying streamflow under and through the dump particularly if suitable mine waste rock is available. However, little published information exists on design methods and performance of these rock drains. This lack of available information is limiting and delaying approvals from government with resulting costs to mining companies.

THE OBJECTIVE OF THE SYMPOSIUM

The objective of the symposium is to assemble and review experience in the design, construction and operation of rock drains. From this it is hoped to develop guidelines that will allow mine development to be more cost effective and identify potential environmental impacts of rock drain construction both during operation and following abandonment.

Aspects to be considered in design, construction and operation of rock drains will include the following:

a. suitable methods of determining rock quality and gradation for use in drains;

b. hydraulic design of flow-through capacity;

c. hydrologic analysis and the damping effect of rock drains on flood peaks, also upper basin effects;

d. slope stability considerations;

e. water quality associated with rock drains (sediment, nutrients and heavy metal additions);

f. effect on water quality downstream of the rock drains;

g. long term performance of rock drains under various climatic and geologic conditions;

h. seasonal effects to be considered in design and construction;

i. instrumentation and monitoring;

j. naturally occurring rock drains.

ORGANIZATION OF SYMPOSIUM

(Note: Individuals noted [*] are members of the Technical and Research Committee on Reclamation)

Symposium Chairman

*Roger J. Berdusco, Fording Coal,
Elkford, B.C. V0B 1H0
Chairman, Research Subcommittee of Technical and Research Committee
Telephone: (604) 865-2271
Telex: 865-2271315

Symposium Co-ordinator

*D. Murray Galbraith,
Engineering and Inspection Branch,
B.C. Ministry of Energy, Mines and Petroleum Resources
525 Superior Street, Victoria, B.C. V8V 1X4
Telephone: (604) 387-3781
Telex: 049-7135 Fax: 387-5713

Hotel/Tour:

*Art L. O'Bryan,
Engineering and Inspection Branch
B.C. Ministry of Energy, Mines and Petroleum Resources
310 Ward Street, Nelson, B.C. V1L 5S4
Telephone: (604) 354-6125/30

Registration

*Terry Johnson, Mining Association of B.C.
#860-1066 W. Hastings Street
Vancouver, B.C. V6E 3X1
Telephone: (604) 681-4321
Telex: 04-507784

Liaison-Energy, Mines & Resources (Canada)

Al Clarke, Mineral Policy Sector
580 Booth Street, Ottawa, ON. K1A 0E4
Telephone: (613) 995-9466

Liaison-Coal Association-Canada

F. Nick Agnew
Suite 301, 1000 - 8Avenue SW
Calgary, AB T2P 3M7
Telephone: (403) 262-1544

Special Advisor

*J.D. (Jake) McDonald
6706 Tamany Drive, Victoria, B.C.
Telephone: (604) 652-2433

Liaison-B.C. Ministry of Environment

*John Dick, Planning & Assessment Branch
Parliament Buildings, Victoria, B.C.
Telephone: (604) 387-4441

Liaison-CIM CrowsNest Branch (District 6)

Brent Drensmore,
Crows Nest Resources Ltd.
P.O. Box 2003, Sparwood, B.C. V0B 2G0
Telephone: (604) 425-2555

Other Members of the Technical and Research Committee not noted above are:

Mr. R. Hillis
Utah Mines Ltd.
Island Copper Mine
P.O. Box 370, Port Hardy, B.C. V0N 2P0
Telephone: (604) 949-6326

Mr. J. Robertson
Teck Corporation
1199 W. Hastings Street,
Vancouver, B.C. V6E 2K5
Telephone: (604) 687-1117

Mr. J. Lant
Crows Nest Resources Ltd.
P.O. Box 2003, Sparwood, B.C. V0B 2G0
Telephone: (604) 425-2555

Dr. M. Bell
Environmental Studies
University of Victoria
P.O. Box 1700, Victoria, B.C. V8W 2Y2
Telephone: (604) 721-7107

Dr. L.M. Lavkulich
Faculty of Agricultural Sciences
University of British Columbia
248, 2357 Main Mall
Vancouver, B.C. V6T 2A2
Telephone: (604) 228-3477

Mr. R. Gardiner
Cominco Copper Division
Trail, B.C. V1R 2N2
Telephone: (604) 364-4457

LIST OF SPEAKERS AND TOPICS

Technical Speakers

Dermot Lane Reclamation Officer Fording Coal Limited Elkford, B.C. V0B 1H0 (604) 865-2271	"Five Years Experience with the Swift Creek Rock Drain at Fording Coal Limited"
David B. Campbell, P.Eng. Golder Associates Consulting Geotechnical and Mining Engineers 224 W 8th Avenue Vancouver, B.C. V5Y 1N5 (604) 879-9266	"Five Years Performance of the Swift Creek Rock Drain at Fording Coal Limited"
Louis W. Hamm Mining Engineer US Office of Surface Mining Reclamation and Enforcement 1020-15th Street Denver, Colorado 80202 USA (303) 844-2517	"Valley Fill Practices in Western United States Coal Mining"
Prof. Jack Lawson Head of Civil Engineering Dept. University of Melbourne Melbourne, Australia (613) 344-6792 Telex: AA35185	"Protection of Rockfill Dams and Cofferdams Against Overflow and Through-flow" - The Australian Experience
Peter C. Lighthall, P.Eng. Klohn Leonoff Consulting Engineers 10180 Shellbridge Way Richmond, B.C. V6X 2W7 (604) 273-0311	"An Intregrated Approach to Design of Rock Drains"
Jon D. Ventura, Civil Engineer US Office of Surface Mining Reclamation & Enforcement Eastern Technical Centre, Ten Parkway Center Pittsburgh, PA 15220 USA (412) 937-2808	"Design and Construction Practices in Rock Drains"
Chuck Brawner, P.Eng Dept. of Mining Mineral Process Engineering University of B.C. Vancouver, B.C. V6T 1W5 (604) 228-3986	"Geotechnical Considerations for Rock Drains"
Robert A. Welsh US Office of Surface Mining Reclamation & Enforcement Eastern Technical Center Ten Parkway Center Pittsburgh, PA 15220 USA (412) 937-2808	"Evaluation of Durability Testing Techniques for rock Underdrain" Material used in Appalachian Surface Coal Mining Valley Fills

David B. Campbell, P.Eng.
Golder Associates
Consulting Geotechnical and
Mining Engineers
224 W 8th Avenue
Vancouver, B.C. V5Y 1N5
(604) 879-9266

"Discussions of Concerns Regarding the Long-term Performance of Rock Drains"

Robert S. Nichols
Project Services Supervisor
Esso Resources Canada Ltd.
Calgary, Alberta
(403) 563-3833

"Rock Segregation in Waste Dumps"

R.W. Thompson
CTL Thompson
1972 W. 12th Avenue
Denver, Colorado 80204 USA
(303) 825-0777

"Rock Drains Spoil Disposal Areas Eckman Park Mine" Steamboat Springs, Colorado

Fred B. Claridge
Piteau & Associates
Geotechnical Consultants
206, 615-10th Avenue SW
Calgary, AB T3C 0J7

"Rock Drain Behaviour at Byron Creek Collieries Sedimentation Pond"

Dr. Brian Chappell
Mining Section - Bureau of Mineral
Resources, Gov't of Australia
Constitution Avenue
Canberra, Australia
(602) 499-554

"Rock Fill Dams in Handling Mine Waste Material"

Mr. W.A. McLaren, P.Eng.
President, Western Canada
Hydraulics Laboratories Ltd.
224 W. Esplanada
N. Vancouver, B.C. V7M 1A4
(604) 985-7741

"Modelling of Flow Through a Mine Waste Dump"

Mr. J. Gourdou
Tecminemet Ingenierie
1, Avenue Albert Enstein
B.P. 106
78191 Trappes cedex
France
(3) 050-61-88

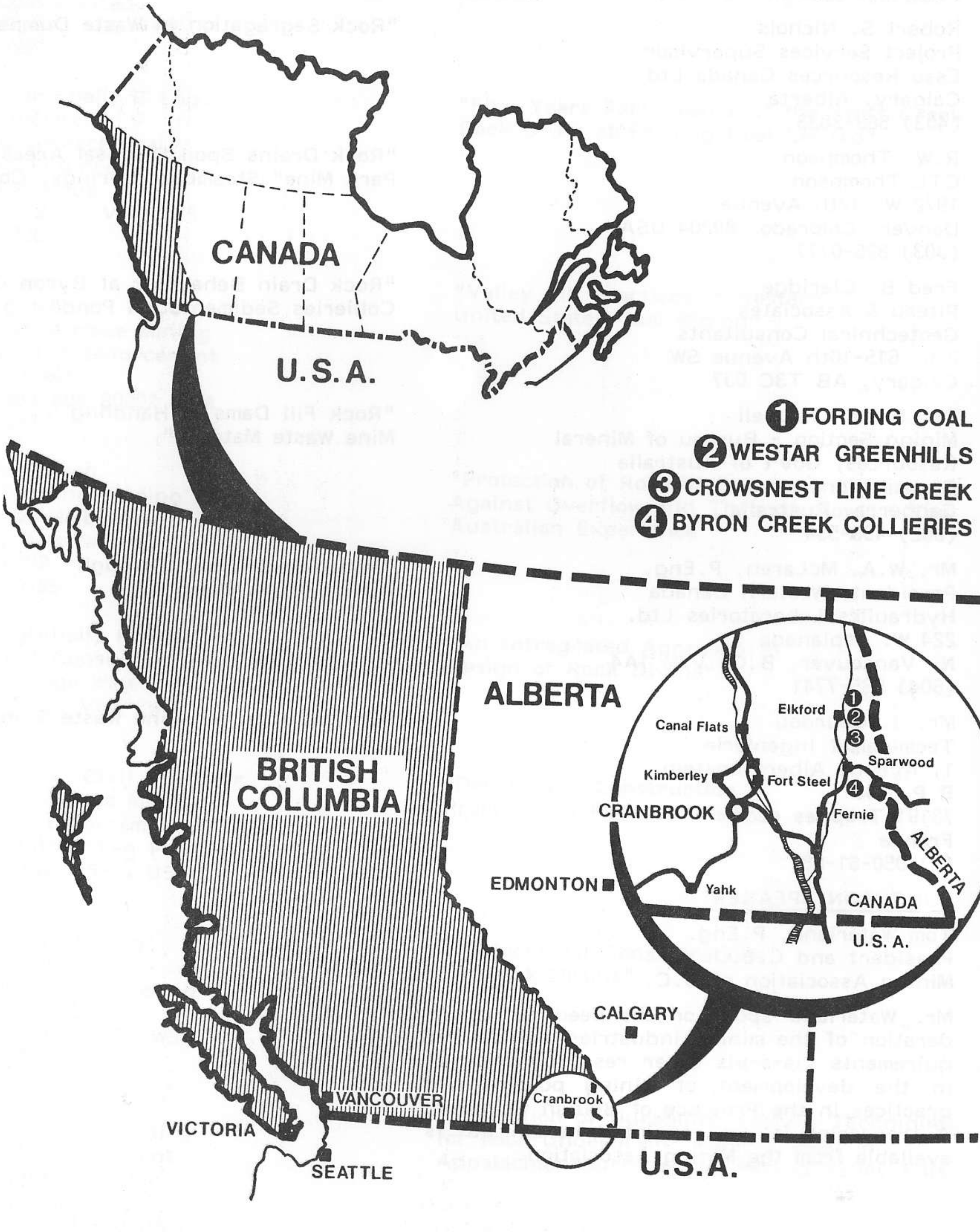
"Contribution for Mine Waste Stability"

LUNCHEON SPEAKER

Tom Waterland, P.Eng.
President and C.E.O.
Mining Association of B.C.

Mr. Waterland spoke on the need for consideration of the mining industries' critical requirements vis-a-vis other resource demands in the development of mining policies and practices in the Province of British Columbia. A transcript is not reprinted herein but is available from the Mining Association.

FIELD TRIPS OF KOOTENAY AREA COAL MINES



5 YEARS EXPERIENCE WITH THE SWIFT CREEK ROCK DRAIN AT FORDING COAL LIMITED

by

D. Lane
Reclamation Officer

R. Berdusco
Administrator, Regulatory and Public Affairs

R. Jones
Environmental Control Officer

INTRODUCTION

Fording Coal Limited operates the Fording River Operations Coal Mine located in southeastern British Columbia, Canada. The minesite, as shown in Figure 1, is within the medial range of the southern Canadian Rocky Mountains, approximately 136 km north of the United States - Canadian border and 6 to 12 km west of the British Columbia - Alberta provincial border.

The Fording River Operations produces an average of 4 million metric tonnes of cleaned coal per annum, primarily for export to Japan. Both thermal and metallurgical coal are produced at the minesite. Mining operations commenced in 1972 and are carried out on a continuous basis. The operations employ both truck/shovel and dragline mining techniques in multiple seam pits. Total material moved annually is approximately 50 million bank cubic metres (BCM) of waste and raw coal.

One of the major considerations in mine planning at such a large scale as employed at the minesite is the location of spoil waste dumps. Steep mountainous terrain and the occurrence of numerous streams places costly constraints on spoil location. Minimizing haul distances, development costs and reclamation requirements are critical components of an efficient mine plan.

Typical spoil from the coal mining at the Fording River Operations consists of sandstones, mudstones, shales and minor amounts of glacial till and colluvium overburden. Disposal of the waste material by end-dumping over the advancing face of a spoil dump results in segregation of the waste material as it slides and rolls down the face. The fine, least durable mudstone and shale material accumulates near the crest of the spoil face or breaks up moving down the face, while the large, most durable sandstone fragments come to rest at the base of the spoil. This results in the formation of a zone of coarse rock in the base region of the spoil which enhances stability and precludes the development of pore pressures within the dump. It is this characteristic of the Fording spoil dumps and the spoil planning requirements that led to the concept of the flow-through rock drain.

Alternatives to the rock drain include culverting through spoils, diversion of streams around spoil areas and long hauls to areas that do not have steam/spoil conflicts.

Although each case has its own specific considerations, generally there is significant economic advantage to use rock drains where possible. At the Fording River Operations, the use of flow-through rock drains has led to greater availability of shorter haul areas for waste disposal, a reduction of spoil development costs, a reduction of impacts on streams, a reduction of the land area required for spoiling and reduced land reclamation costs.

REGULATORY CONSIDERATIONS

Regulatory requirements for rock drains on the Fording River Operations vary greatly depending upon the specific water course to be crossed.

Legislative requirements may or may not include approval undar section 7 or 8 of the "Water Act" and always require approval under section 6 of the "Mines Act".

The size of the stream, whether it is permanent or ephemeral, whether any modification of the stream is required prior to spoiling and whether the stream is fish-bearing are all considerations of the type of permitting required.

Since 1982, over twenty flow-through rock drains have been constructed in spoils at the Fording River Operations.

The following discussion focuses on the design, construction and operation of the Swift Creek rock drain and summarizes 5 years of operating experience.

THE SWIFT CREEK ROCK DRAIN PROJECT

The Swift Creek Rock Drain Project was initiated in 1980 with the primary purpose of testing the flow capacity of the zone of coarse rock that develops in the base region of free-dumped coal mine spoils. The site at Swift Creek was chosen as there was an immediate requirement to provide a means for conducting the surface flow of the creek through a fill ramp required to access a new spoil area south of Swift Creek. The project was considered timely as rock drains are expected to have a strong potential application for future developments throughout the minesite, and several years experience will be necessary to prove the long term usefulness of these structures.

Design Criteria

At the initiation of this project, it was not possible to accurately predict the ability of the zone of coarse rock at the base of free dumped spoils to channel water flow. This fact necessitated that an additional rock zone of predictable capacity be placed near the base of the spoil to ensure that the design peak flow of Swift Creek could be channelled through the spoil (Figure 2). This zone, constructed from select rock, is situated parallel to, and a minimum of 2 meters higher in elevation, than the existing creek channel, so that it will only be used if the naturally produced rock zone is unable to handle the water flow in Swift Creek.

The 200 and 1000 year flood discharges in Swift Creek were estimated to be 5.8 cumecs and 7.1 cumecs, respectively, based on a study concerning the prediction of flood flows in the Fording River (Kerr, Wood, Leidal Associates, 1980). The discharge capacity of the constructed rock drain was estimated to be 0.07 cumecs per square metre of cross-sectional area. It was designed to provide a 75 m^2 cross-section that would conduct approximately 5 cumecs. The zone of coarse rock at the base of the spoil, developed as a result of end-dumping, was estimated to have a discharge capacity of 0.08 cumecs per square metre of cross-sectional area. It was conservatively expected to provide a 50 m^2 cross-section that should be able to conduct approximately 4 cumecs. The combined capacity of these two rock zones was therefore designed to be approximately 9 cumecs.

Construction Details

Construction activities for the Swift Creek Rock Drain Project began in September, 1980 and were fully completed by March, 1982. The completed drain structure was built to the design specifications recommended by Golder Associates, the geotechnical consultant involved with this project.

Prior to the placement of spoil over Swift Creek, six piezometers were installed along the section of the Swift Creek Channel which would be covered with the spoil material. The piezometers enable the measurement of the elevation of the free-water surface within the base of the cross-over fill. At each piezometer location, surveys were made to establish the profile of the ground surface in a direction transverse to the axis of the stream channel.

Monitoring Program

The monitoring program implemented for the Swift Creek Rock Drain Project has the objective of measuring the flow capacity through the naturally produced coarse rock zone and quantifying the effects of the rock drain on the spoil fill stability and the water quality of Swift Creek. The specific data collected includes water flows with corresponding piezometric water elevations and survey of the elevations of pins located at the top of the crossover spoil ramp to measure spoil crest movements. Water samples are collected immediately upstream and immediately downstream of the rock drain and analyzed for pH, turbidity, suspended and dissolved solids, conductivity, hardness, alkalinity, nitrogen compounds, sulfates and dissolved and total metals.

Summary for Five Years of Results

The performance of the Swift Creek Rock drain has been satisfactory for five years of operation. The free dumped rock drain was adequate for all flows experienced and the constructed drain was not required to conduct the water flow of Swift Creek.

Stream Flow Data

The maximum measured water flow through the Swift Creek Rock Drain during the 5 years it has operated is 1.7 m^3/sec which occurred in 1982. Table 1 summarizes the maximum measured stream flows for the years 1982-1986, inclusive.

Piezometer Data

Piezometer data for the Swift Creek Rock Drain is available for 4 out of the 5 years of operation. The piezometers malfunctioned in 1986 and no data was collected. In order to ensure continued monitoring of water levels within the dump, two rotary drilled holes were installed at the sides of the haulroad located on the top of the crossover hill. These holes are centered over the original Swift Creek channel and will be used as wells to measure water levels within the crossover fill.

The available piezometer data indicates that the through-flow capacity of the naturally producted coarse rock layer at the base of the Swift Creek Crossover Ramp has remained essentially constant for 4 years and is not diminishing with time. Table 2 summarizes the range of average discharge capacities recorded for the years 1982 to 1985, inclusive.

The range of discharge capacities occurs because of differences in the height of spoil above each of the six piezometers. The highest discharge capacity occurs near the downstream toe of the crossover fill. The lowest discharge capacity occurs near the centre of the crossover fill beneath the haulroad. This variability in discharge capacity clearly indicates that loading from the spoil mass above the coarse rock layer is an important factor in determining the discharge capacity of a rock drain structure.

TABLE 1

MAXIMUM MEASURED STREAM FLOWS FOR SWIFT CREEK FOR THE FIRST FIVE YEARS OF OPERATION

YEAR	MAXIMUM MEASURED FLOW (m^3/sec)
1982	1.7
1983	0.801
1984	0.647
1985	0.516
1986	0.478

TABLE 2

THE RANGE OF AVERAGE DISCHARGE CAPACITIES (m^3/sec per square metre of Cross Sectional Area) FOR THE SWIFT CREEK ROCK DRAIN FOR THE YEARS 1982 to 1985

YEAR	RANGE OF AVERAGE DISCHARGE CAPACITY (m^3/sec per m^3 of cross sectional area)
1982	0.038 - 0.22
1983	0.053 - 0.12
1984	0.031 - 0.12
1985	0.058 - 0.091

Spoil Stability

The measured discharge capacities of the Swift Creek Rock Drain permit the prediction of water levels within the cross-over fill for stream flows greater than the flows that have occurred during the period where observations have been made. The water level within the cross-over ramp corresponding to the 1000 year flood discharge of 7.1 m^3/sec was estimated and the stability of the ramp assessed. The results of the assessment indicated that the 1000 year flood will not impair the stability of the spoil provided that a 3 m high fillet of coarse rock be placed at the location where water flows exit the toe of the cross-over ramp. This fillet was constructed in 1983.

Monitoring of the cross-over fill has been carried out to measure any settlement that is occurring at the crest of the spoil. Six survey pins were installed to record vertical displacements and three of these have provided a continuous record. The average vertical drop of these pins in the period of December 1982 to November 1985 is 1.93 metres with an average of 1.69 m occurring prior to April, 1984. This settlement rate is average to below average compared to the settlement of other stable spoils at the Fording River Operations.

Water Quality

A water quality monitoring program has been carried out at the Swift Creek Rock Drain since 1982 with the objective of quantifying the effects of the rock drain on the water quality of Swift Creek. The monitoring program consists of sampling above and below the rock drain to determine the net effect of the drain on various water quality parameters.

The Swift Creek Rock Drain produced an initial elevation of most water quality parameters during the early part of the first runoff period in 1982. This was an expected occurrence as the first water flows flushed-out the rock drain.

Nitrogen Compounds

There are three nitrogen compounds monitored at Swift Creek - ammonia (NH_3), nitrite (NO_2) and nitrate (NO_3). The source of these nitrogen compounds is the residual explosive in the blasted waste rock used in the construction of the rock drain. Explosives used at the minesite contain up to 33% nitrogen.

Ammonia levels have remained near detection limits for both above and below rock drain sampling locations and are not affected by the rock drain.

Except for the initial flushing-out of the rock drain, nitrogen in the form of nitrite, has remained below Ministry of Environment's receiving water objective applicable to the Fording River.

Peak concentrations of nitrate nitrogen also occurred in the first year due to the flushing-out of the rock drain. In subsequent years, peak concentrations have decreased substantially with minor fluctuations due to hydrological conditions.

Calculated nitrogen loadings indicate that the quantity of nitrate nitrogen released or discharged from the rock drain in 1985 is similar to the years 1983 and 1984 but substantially lower than 1982. The rock drain's contribution of nitrogen nitrate to the Fording River is relatively small, less than 5% of the total from the minesite.

Other Parameters

Sulphate, pH, conductivitity, alkalinity, hardness, dissolved solids and metals were monitored in the initial years to indicate general trends. Except for an initial elevation of sulphates, conductivity, alkalinity, hardness and dissolved solids due to the flushing-out of the rock drain, the effect of the rock drain on these parameters is negligible.

Sediment

There are two types of sediment that are transported by Swift Creek:

1. bed-material load, which includes all sizes of sediment found in appreciable quantities in the bed material;

2. suspended load, consisting mostly of fines, which are not found in appreciable quantities in the bed material.

Bed-material Load

The quantity and type of bedload transported by Swift Creek may be of concern to the performance of the rock drain structure. A buildup of bedload inside and upstream of the rock drain outlet may reduce the void space and affect the discharge capacity of the drain in the long term frame.

In order to study the effects of bedload, monitoring stations were set up in 1983 to measure the bedload quantities contributed by the two main forks of Swift Creek. In addition, monitoring of bedload deposited in the pond area formed upstream of the rock drain inlet has been carried out since 1982.

The results from bedload monitoring are preliminary but one important conclusion is apparent. The south fork of Swift Creek contributes 5% or less of the total bedload material in Swift Creek and the main fork contributed the rest, 95% or greater. The significance of this fact is that the south fork is an undisturbed drainage while the main fork has been disturbed by both logging and mining activities. Therefore the buildup of bedload upstream of the rock drain is a result of the disturbances in the main fork of Swift Creek. Regeneration of the forest on the logged area and reclamation of the mine disturbances should result in a significant decrease in bedload quantities as the watershed approaches a more natural state and thus ensure the long term viability of the Swift Creek Rock Drain. Future monitoring and analysis of bedload transport in Swift Creek is expected to confirm this hypothesis.

Suspended Load

The sampling results to-date suggest that the rock drain structure does not have an effect on suspended solids concentrations. In addition, an investigation of the water velocities through the void spaces within the drain indicates that the deposition of suspended load should not occur.

SUMMARY

Flow-through rock drains are an important and cost effective component of spoil planning at the Fording River Operations. Five years of experience with the Swift Creek Rock drain have clearly demonstrated that rock drains provide an effective means to conduct water flows through spoil dumps. The impact of rock drains on water quality is generally short-lived and within manageable limits.

FIVE YEARS PERFORMANCE OF THE SWIFT CREEK ROCK DRAIN AT FORDING COAL LIMITED

by

David B. Campbell, P. Eng.
Principal, Golder Associates
Consulting Geotechnical Engineers
Vancouver, British Columbia

INTRODUCTION

The Swift Creek rock drain represents the first rock drain developed at a mining project in British Columbia. This rock drain, which was developed during the winter of 1981-82 conducts the surface flows in Swift Creek beneath the base of a 55 m high rockfill causeway that was required to provide access to a new dumping area located on the southside of the drainage.

The materials comprising the Swift Creek rock drain consist of large rock fragments that separated on the face of the advancing causeway fill, rolled to the toe, and came to rest within the depression of the Swift Creek drainage channel. Because no precedent data were available regarding the through-flow capacity of coarse gravity-separated rock fragments, an auxilliary placed rock drain, with its base at a level approximately 2 m above the invert of the natural creek channel was constructed parallel to the drainage course. The material used to construct the placed rock drain consisted of coarse angular fragments of waste rock garnered from the toe regions of nearby waste rock dumps.

The data collected at the Swift Creek rock drain over the past five years have shown that the placed rock drain has remained completely dry, and that the Swift Creek flows have remained within the lower rock drain comprising gravity-separated rock fragments. This paper summarizes the observed performance of the lower rock drain i.e. the drain comprising gravity-separated fragments, over the past five years.

INSTRUMENTATION

Before the rockfill causeway was advanced across the Swift Creek drainage channel, six piezometers were installed at the bottom of that segment of the drainage course to be covered by the causeway fill. The piezometers consist of wellpoint screens complete with flexible plastic tubes that extend to a pressure measuring station located a short distance downstream of the downstream toe of the causeway fill. The piezometer tubes were threaded through a steel pipe, which was buried at a depth of approximately 2 m below surface to provide protection against potential damage as a result of impact by the large rocks that gain significant kinetic and rotational energy in the course of transit down the face of the advancing causeway fill.

At each of the piezometer locations, detailed surveys were made to establish the configuration of the channel cross-section. These survey data were used to calculate cross-sectonal areas, and to establish the relationship between area and elevation, employing the type of plot shown on Figure 1.

The piesometric elevations at piezometer locations are determined from measurements made at a closed-end monometer. Provided the water levels at the monometer are read correct to plus or minus 1 mm, the elevations of the free water surface within the rock drein at individual piezometer locations can be determined to plus or minus 20 mm. Thus, the piezometer data, together with the relationship between elevation and area provide a means by which the gross area of the wetted cross-section can be determined at the location of an individual piezometer.

Each time that a set of piezometer data is obteined, measurements are also made of the rate of flow through the rock drain. The data are plotted in the form of rate of discharge versus the area of the wetted cross-section at each of the piezometers. A typical plot is shown on Figure 2. Although the plotted points show a degree of scatter, the relationship between rate of discharge and wetted area can be represented by a straight line, the slope of which is a measure of the rate of flow per unit of gross wetted cross-section within the drain. This straight line relationship is in conformance with that which should be expected for turbulent flow through coarse, broken rock, where the hydraulic gradient is controlled by the slope of the natural drainage course.

INTERPRETATION OF PERFORMANCE TO DATE

The long-term performance of the rock drain, and the potential for reduction in through-flow capacity as a result of deposition of sediment within the rock drain has been the chief concern expressed by members of regulatory agencies responsible for assessment of permit applications.

Comparison of rates of flow and the corresponding plezometric levels within the drain at individual piezometer locations is one method of assessing performance over time, and assessing whether reduction in through-flow capacity is occurring. Figure 3 is a plot of the measured rate of flow through the drain vs the elevation of the free water surface at the location of piezometer No. 2 for the years 1982 and 1985.

The data points are scattered within a horizontal band, which has a width representing a range of approximately 0.5 m in elevation. An explanation of the scatter in the plotted points representing the 1985 data is provided by Figure 4 which is a plot of rate of discharge versus apparent area of wetted cross-section. The data points for May 23rd indicate a through-flow capacity of approximately $4.2 \times 10^{-2} m^3/sec$ per m^2 of wetted cross-section. Between May 23rd and June 3rd, the apparent unit rate of discharge through the drain increase progressively to a value of $6.8 \times 10^{-2} m^3/sec$ per m^2. The most probable explanation for the increase in apparent discharge capacity during the interval May 23rd to June 3rd is melting of ice that accumulated within the Swift Creek rock drain during the previous winter.

Observations on the faces of waste rock dumps in eastern British Columbia show that seasonal exchange of air occurs within the dumps at the onset of winter. As the ambient temperature drops at the beginning of winter, field observations show conclusively that convection currents result in expulsion of warm air on dump surfaces, particularly on the sloping faoes of dumps. Warm air expelled from a dump is replaced by higher density cold air that invades the dump through its toe region. The field observations show that cold air does invade waste rock dumps, and that some buildup of ice within a rock drain should be expected.

The data plotted on Figure 4 indicate that on May 23rd, 1985, ice remained present within the Swift Creek rock drain and that the presence of this ice raised the elevation of the free water surface at the location of piezometer No. 2. This increase in the indicated elevation of the free water surface resulted in an over-estimation of the area of the wetted cross-section through which flow was taking place.

During the 11-day interval May 23rd, 1985 to June 3rd, the flow of weter through the drain resulted in melting of ice from approximately 4.5 square meters of the drain cross-section. By June 3rd, 1985, the indicated unit rate of flow through the drain had increased by more than 50 per cent relative to the unit through-flow rate indicated for May 23rd.

The discharge capacities of the drain per unit of wetted cross-section at the locations of piezometers 1 to 4 inclusive for the years 1982 to 1985 are summarized in graphical form on Figure 5. As indicated on Figure 6, piezometers 3 and 4 are located beneath the causeway fill platform, and are covered by approximately 55 m of waste rock. Piezometers 1 and 2 are located beneath the downstream fill slope.

The data on Figure 5 indicate that through-flow capacity per unit of wetted cross-section is lower at piezometers 3 and 4, located beneath the platform, than at piezometers 1 end 2. The increase in through-flow capacity from piezometers 3 and 4 to piezometer 1 is indicated on Figure 6(b). The data indicate that through-flow per unit area of wetted cross-section within the drain increases exponentially in the down-stream direction for that segment of the rock drain located beneath the down-stream shoulder of the fill.

At the time the toe of the causeway fill began to advance over the zone of coarse segregated rock that forms the Swift Creek rock drain, both rock size and rock type were uniformly distributed along the length of the drainage course extending from the upstream to the downstream limits of the drain. Consequently, the reduced through-flow capacity beneath the platform of the causeway is not related to differences in rock type or in rock size at the time the rock drain became covered.

The lower value of through-flow capacity beneath the causeway platform, and the increase in through-flow capacity in the downstream direction is related to variations in thickness, and therefore in the weight of the waste rock above the drain. The stresses that result from the weight of the waste rock above the drain are transferred through the rock drain by point-to-point contacts between neighboring blocks of rock. Initially, these contact areas were small, with the result that as the drain became covered by causeway fill, the point-to-point contact stresses increased to high values, and undoubtedly resulted in crushing at the contacts, as well as fracturing of some of the blocks. Crushing and fracturing results in a slight reduction in the mean effective size of the fragments comprising the drain, and the fracture fragments can be expected to have resulted in a modest reduction in void ratio. Since both void ratio and particle size govern hydraulic radius (reference Campbell, 1986) this reduction in hydraulic radius is responsible for in a reduction in the through-flow capacity per unit of cross-sectional area beneath the causeway platform.

Figure 6 is a vertical cross-section coincident with the longitudinal axis of the Swift Creek rock drain. The locations of piezometers are indicated, together with the level of the measured piezometric profile coresponding to a discharge of 0.77 m^3 per sec. through the drain, the maximum rate of discharge for which data are available. This piezometric profile is labelled curve 'A' on Figure 6(a). The measured rate of discharge per unit area of wetted cross-section at each of the piezometers was used to estimate the area of the wetted cross-section corresponding to a discharge rate of 7.1 m^3 per sec, the predicted 1,000-year flood event. By imploying the relationship between cross-sectional area and elevation at each of the piezometers, the piezometric level correaponding to the 1,000-year flood event was estimated. This predicted level is labelled curve 'B' on Figure 6(a).

To guard against ravelling at the outlet end of the drain during intervals of high discharge, a fillet consisting of individual rock fragments approximately 1 m in size was placed at the downstream toe. The surface of this fillet slopes at 5 horizontal to 1 vertical, and intersects the downstream face of the causeway fill at a level 3 m above the base of the natural drainage channel.

CONCLUSION

The data that have been collected at the Swift Creek rock drain over the past 5 years have permitted an assessment of the rate of discharge through the drain per unit area of wetted cross-section. At piezometer No. 1, where the thickness of fill above the drain is approximately 10 m, the measured rate of through-flow per unit of wetted area is slightly greater than the flow the value predicted at the time the initial design studies were carried out. Beneath the roadway portion of the causeway, the measured through-flow capacity per unit of wetted cross-section is somewhat lower than the values originally predicted. These lower values of through-flow capecity beneath the roadway are believed to be the result of crushing and fracture of constituent rock fragments comprising the drain in response to the stresses imposed by the overlying fill.

The data to date indicate that the through-flow capecity of the Swift Creek rock drein is not decreased with time.

REFERENCE

Campbell, D.B. Discussions of Concerns Regarding the Long-Term Performance of Rock Drains. International Symposium on Flow-Through Rock Drains, Cranbrook, British Columbia, September, 1986.

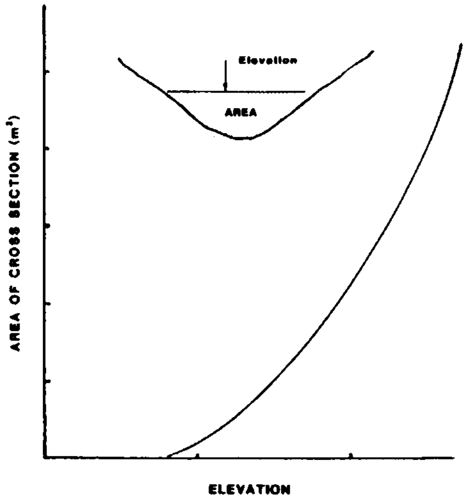


FIGURE 1
An example of the relationship between area and elevation at an individual piezometer, as established by survey data.

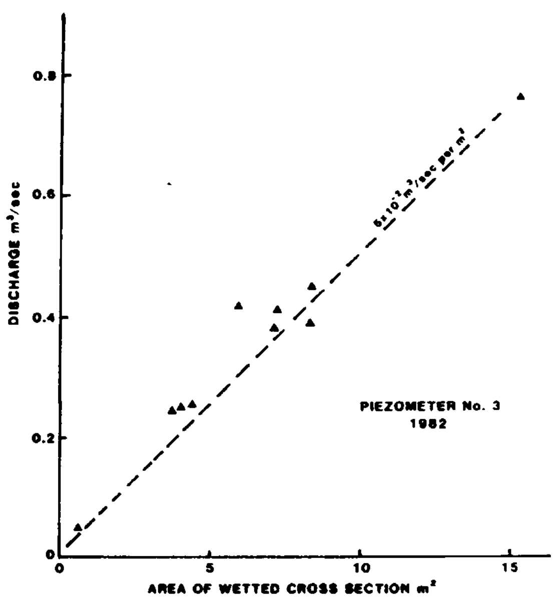


FIGURE 2
A typical plot of rate of flow vs area of wetted cross section at an individual piezometer location.

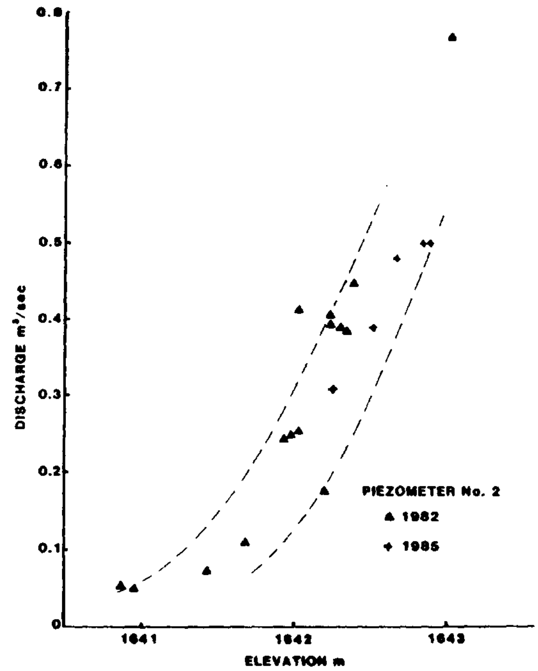


FIGURE 3
Plots of discharge vs water surface elevation for 1982 and 1985 at the location of piezometer No. 2.

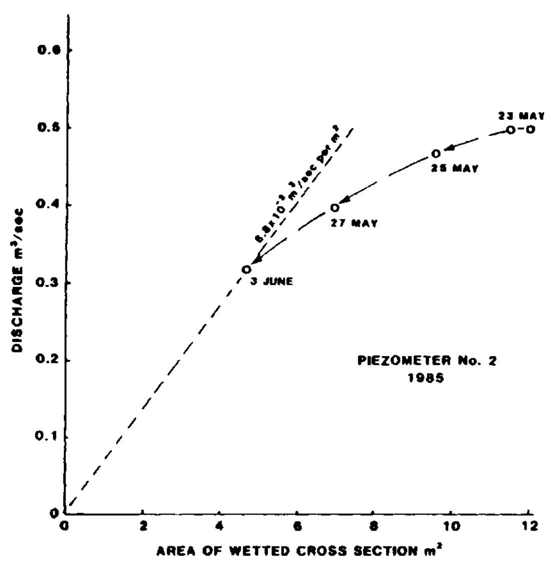


FIGURE 4
Rate of flow plotted against wetted area at piezometer No. 2 during the interval 23 May to 3 June '85. The data indicate that during this interval, flow through the drain resulted in melting of ice from approximately 4.5 square metres of the cross section.

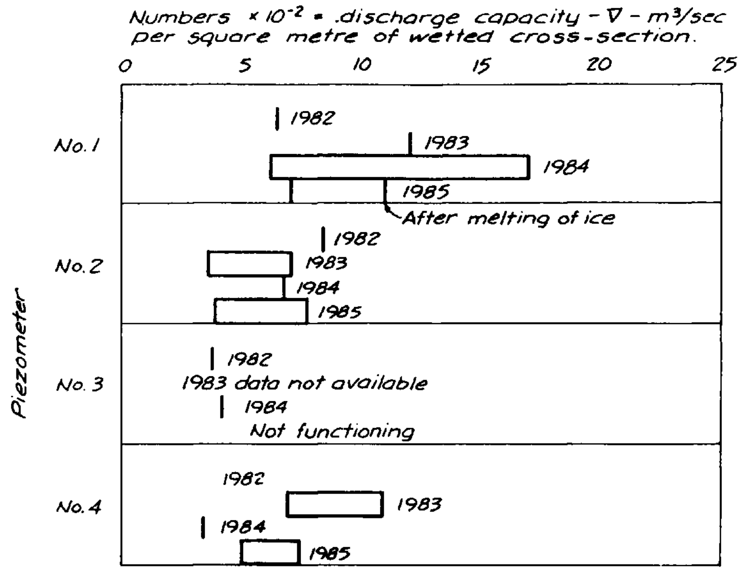


FIGURE 5
A graphical presentation of indicated through-flow capacities for 1982 to '85 inclusive. The data indicate that through-flow capacities are not decreasing with time.

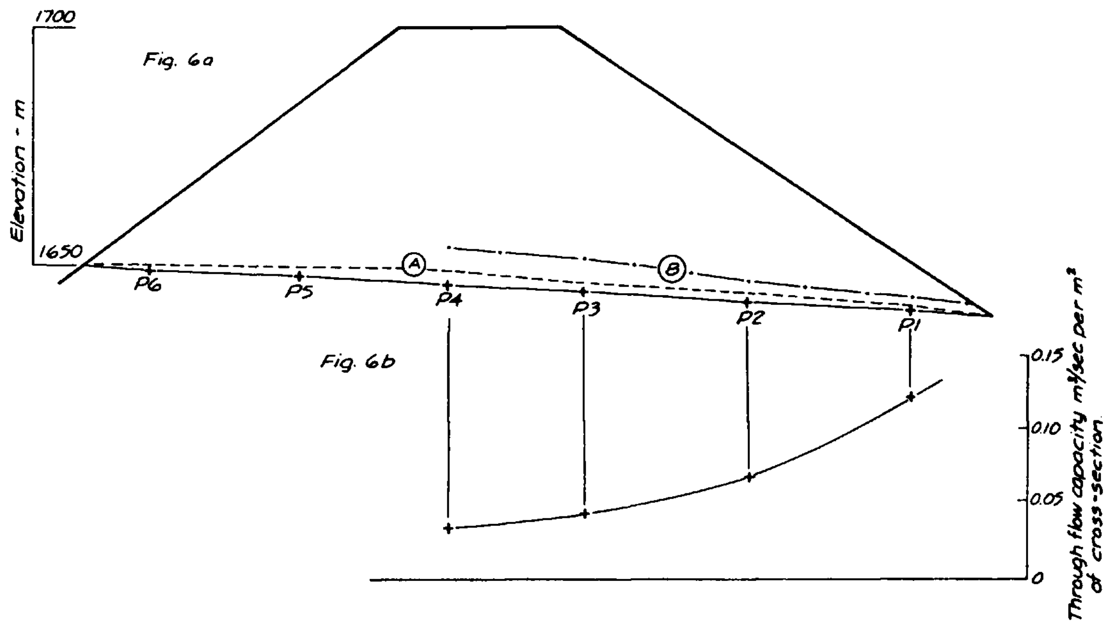


FIGURE 6
Section coincident with the axis of the Swift Creek rock drain showing the observed piezometric profile at the base of the causeway fill corresponding to a flow of 0.77 cumecs (curve A), and the predicted profile corresponding to 7.1 cumecs, the predicted 1000-year event (curve B). The manner in which through-flow capacity increases in the downstream direction is illustrated by Figure 6(b).

VALLEY FILL PRACTICES
IN WESTERN UNITED STATES
COAL MINING

by

Louis W. Hamm
Mining Engineer
United States Office of Surface Mining
Reclamation and Enforcement
Western Field Operations
Denver, Colorado

ABSTRACT

This paper is intended to present a representative sampling of the occurrence and usage of valley fills in Western United States coal mines. Valley fills observed in the field, and design data was reviewed from the mining permit applications submitted in response to the regulatory requirements of the Surface Mining Control and Reclamation Act of 1977 (SMCRA).

Western underground mining operations involve longwall and/or room and pillar mining methods in rugged mountain areas where steep topography provides no natural location for surface fecilities. At some Western underground mining operations a surface pad is created by excavating valley wall material and depositing it in the valley bottom, creating a valley fill to support the necessary surface facilities. These fills are frequently established in perennial or intermittent stream valleys. Stream drainage is temporarily diverted through the fill in culverts while the mine is operating. The temporary diversions are later removed during reclamation and the fill material regraded.

Surface operations near mountain areas, such as the Yampa River Basin of Colorado, sometimes deposit excess spoil as valley fills in small drainages adjacent to the mining operation. A combination of rock drains and surface diversions are used to convey flow in these drainages through and around the valley fill spoil.

OVERVIEW

The coal mining regions of the Western United States make limited use of valley fill spoil disposal. Most of the open pit coal mining occurs in the Great Plains or areas of low topographic relief. The mining method most commonly used in the mountainous areas of the west is underground mining, which generates relatively little spoil or waste rock.

The contour strip and mountain top removal mining common in the Eastern United States generates considerable spoil in the mountains of Appalachia, resulting in frequent use of valley fill disposal. The dipping, deep beds of the rugged mountains in the west are best suited to underground mining in room and pillar or longwall operations.

For the purposes of this paper, five western operations will be discussed. Three deep mines in Utah and two surface mines in Colorado. These mines typify valley fill utilization in the Western United States coal fields.

The underground mining operations aro located in rugged mountain areas where steep topography provides no natural location for surface facilities. Therefore, a surface pad is created by excavating adjacent valley wall material and depositing it in the valley bottom thereby creating a valley fill to support the necessary surface facilities. The deep coal seams are usually exposed in the deeply incised valleys of perennial or intermittent streams. Therefore, the valley fill for support facilities must accomodate the passage of perennial or intermittent streams at flood stage. Culverts are constructed along the existing stream channel before construction of the valley fill begins, so that the stream flow will be maintained through the fill. During reclamation the fill is either removed or the reclaimed stream channel is diverted over and/or around the fill.

Surface mine operations near mountain areas, such as the Yampa River Basin of Colorado, sometimes deposit excess spoil as valley fills in small drainages adjacent to the mining operation. A combination of rock drains and surface diversions are used to convey flow in these drainages through and around the valley fill spoil.

REGULATIONS

Under the Surface Mining Control and Reclamation Act of 1977 (SMCRA), valley fills are defined as a "structure consisting of any material other than organic material, that is placed in a valley where side slopes of the existing valley, measured at the steepest point are greater than 20 degrees, or were the average slope of the profile of the valley from the toe of the fill to the top of the fill is greater than 10 degrees." (Code of Federel Regulations, 30 CFR 701.5 Definitions)

Both Federal and State regulations (Code of Federal Regulations, 30 CFR 816.71 through .73, and 817.71 through .73; Utah Underground Mining Code, UMC 817.71 through .74; and the Code of Colorado Regulations, Colorado rules 4.09.1 through 4.09.4) require that valley fills be constructed so that leacheate and runoff from the fill will not degrade surface or ground waters or exceed required effluent limitations. All vegetation and organic materials must be removed from the disposal area and topsoil shall be removed, segregated, and stored. The fill must be shown to be stable with a minimum long-term safety factor or 1.5. Subdrains are required wherever there is the potential for drainage or seepage, and State regulations require runoff from above the fill to be diverted into stablized diversion channels designed for a 100-year, 24-hour precipitation event. Federal regulations require these diversions be designed for the 100-year, 6-hour event. Temporary diversions are required to pass the 10-year, 24-hour event (State) or the 10-year, 6-hour event (Federal). Excess spoil is required to be placed in lifts not to exceed 1.2 meters (4 ft) each, except for fills with spoil that meets requirements for classification as "durable rock". Durable rock spoil must be at least 80 percent, by volume, durable, non-acid and non-toxic forming rock that does not slake in water and will not degrade to soil.

Current Utah regulations are unique from most other Western State regulations in that they specifically prohibit diversion of a stream over a valley fill. Federal regulations require that diversions over the fill be placed in channels designed to safely pass the runoff from a 100-year, 6-hour precipitation event. Most State regulations require the channel be designed for the 100-year, 24-hour precipitation event. No uncontrolled drainage over the fill is allowed in Federal or State regulations.

CASE STUDIES

Convulsion Canyon

The Convulsion Canyon Mine, near Salina, Utah, is an underground room and pillar operation (Figure 1). The mine is beginning expansion to include longwall methods as well. The portal area was constructed at the junction of an ephemeral and an intermittent stream. Both are incised into steep canyons of the Southern Wasatch Mountains. The surface facilities pad was constructed in the late 1960's on a 260,000 cubic meter (340,000 yd^3) valley fill covering approximately 5 hectares (13 acres). The valley-fill pad was constructed by excavating material from the adjacent valley walls.

A 183 cm (72 in.) diameter culvert conveys flow from intermittent East Spring Creek along a sandstone ledge beneath the valley fill. Flow from ephemeral Mud Spring Hollow is diverted through a 107 cm (42 in.) culvert to join the larger culvert beneath the fill (Figure 2). The average slope of the East Spring Creek diversion is 4.2 percent, and the slope of the Mud Spring Hollow diversion is 7 percent. The peak flow during a 100-year, 24-hour precipitation event for East Spring Creek has been calculated in the mine permit application to be 21.55 cubic meters per second (761 cfs). The same event in Mud Spring Hollow has been calculated to be 12.83 cubic meters per second (453 cfs), for a total peak flow through the fill area of 34.4 cubic meters per second (1,214 cfs). (Sergent, Hauskins, and Beckwith, 1986)

During reclamation, the fill will be excavated to the culvert which is situated on a sandstone ledge approximately 2.4 meters (8 ft) below the surface of the fill. Once the culvert is removed the fill will be regraded with stable side slopes designed for positive drainage, and the stream channel will be diverted around the fill, over bedrock outcrops, and into a splash basin where it will join the natural stream channel (Figure 3). The proposed reclaimed main stream channel is designed with eight reaches over approximately 400 meters (1,300 ft), with a total vertical drop of about 67 meters (220 ft). The reclaimed channel will be trapezoidal with side slopes of 1:1 and a bottom width of 5.3 meters (17.5 ft). Over the bedrock ledges to the splash basin, the channel will have a bottom width of 3 meters (10 ft) and side slopes of 0.75h:lv, with a bed gradient of .571 to .546. The reclaimed channel will include a layer of filter sand and filter fabric with riprap. Hydraulic characteristics were designed using a Mannings roughness coefficient of 0.035. (Sergent, Hauskins and Beckwith, 1986)

Currently the fill is stable and there have been no major problems with conveyance or runoff through the culvert diversion. Reclamation is scheduled for the year 2010 (Southern Utah Fuel Company, 1980).

Deer Creek

The Deer Creek Mine, near Huntington, Utah is an underground longwall operation with some room and pillar mining as well (Figure 4). The portal facilities are constructed on a 190,000 cubic meter (250,000 yd^3) valley fill at the junction of Deer Creek a perennial stream, with two intermittent streams, Deer Drainage, and Elk Creek. The valley fill occupies approximately 3.5 hectares (8.5 acres). Material to construct the fill was obtained from the south slope of the Deer Creek drainage and from sediment pond construction.

Deer Creek, Deer Drainage, and Elk Creek are passed underneath the facilities area in a 2.5 meter x 1.75 meter (8' 2" x 5' 9") pipe-arch culvert. The culvert and the associated diversions collect runoff from 1,250 hectares (3,100 acres) of the Deer Creek basin. The main Deer Creek culvert is 850 meters (2,800 ft) long with a vertical drop of 128 meters (420 ft). Intermittent Deer Drainage is diverted into a parallel system of 91 cm (36 in.) and 137 cm (54 in.) culverts which feed into the main Deer Creek Culvert. Similarly, a parallel system of 76 cm (30 in.) to 107 cm (42 in.) culverts at Elk Creek feed into the main culvert (Figure 5). Flow during a 100-year precipitation event has been calculated to be 20.6 m^3/s (728 cfs) for the Deer Creek watershed.

Reclamation at the Deer Creek Mine facilities will consist of removing the temporary drainage system and facilities, and recontouring slopes for positive drainage. Riprapped channels with 3 to 6 meter (10 to 20 ft) base widths and 2h:lv side slopes are proposed for reconstructing the Deer Creek, Deer Drainage, and Elk Creek drainages. These channels are designed for the 100-year, 24-hour storm event. Reclaimed channel design (Utah Power and Light Company, 1985) calls for routing the Deer Creek channel over the valley fill to a sandstone outcrop at the downstream extent of the fill. A 9 to 12 meter (30 to 40 ft) wide channel will be cut into the sandstone at that point. Water will flow from the riprapped channel on the fill to the channel cut into sandstone and over the edge of the sandstone cliff, cascading over sandstone outcrop as it falls. A riprap-lined splash basin will be used at the base of the cliff to dissipate energy and transition the Deer Creek flows into those of Elk Creek (Figure 6).

The design for the reclaimed stream channel was approved as an experimental practice* to develop reclamation technology in Western U.S. mountain mines since the Utah regulations prohibit standard permitting of diversions across a fill. The alternative to permitting the diversion as designed was to create much more environmental disturbance by removing, transporting, and depositing the fill material at a new location. The configuration and geology of the valley walls does not readily lend itself to diversion of the stream channels around the fill.

Skyline

The Skyline Mine, near Scofield, Utah, is an underground room and pillar mine. The mine is situated in the high valleys of the Wasatch Mountains at an elevation of approximately 2,620 meters (8,600 ft) above

*Experimental Practices are addressed under Section 711 of SMCRA, "...to encourage advances in mining and reclamation practices..." so long as they "...are potentially more or at least as environmentally protective, during and after mining operations, as those required by promulgated standards..." and "...the experimental practices do not reduce the protection afforded public health and safety below that provided by promulgated standards."

sea level (Figure 7). The portal area facilities are construction on a terraced 558,000 cubic meter (730,000 yd^3) fill over epproximately 12 hectares (30 acros) at the junction of two intermittent forks (North Fork and Middle Fork), and one ephemeral fork (South Fork) of Eccles Creek (Figure 8). The combined drainage area for these streams is approximately 332 hectares (820 acres). Peak runoff during a 100-year, 24-hour precipitation event would be about 3.7 m^3/s (130 cfs). (Coastal States Energy Company, 1979)

Culverts to temporarily convey the stream flow while the mine is operating are located in the original stream channels. The South Fork tributary culvert is 142 cm (56 in.) in diameter and approximately 290 meters (950 ft) long. It is joined by the Middle Fork tributary culvert whioh is 122 cm (48 in.) in diameter and approximately 305 meters (1,000 ft) long. Both culverts join a 152 cm (60 in.) diameter culvert for 43 meters (140 ft) where it is joined by diverted flow from the North Fork tributary. The North Fork tributary flow is diverted through a 122 cm (48 in.) diameter culvert for approximately 283 meters (930 ft) where it joins the other two tributaries to connect with a 183 cm (72 in.) main conveyance culvert for approximately 280 meters (920 ft) to the outlet below the fill (Figure 9). (Coastal States Energy Company, 1979)

The North Fork tributary is unique because the inlet was clogged several times following construction by earth slides from upstream undisturbed areas following precipitation events. Such earth slides are common in the Wasetch mountains. To mitigate the problems, the mine operators constructed a rock drain to a depth of approximately 2.4 meters (8 ft) below the surface at the mouth of the small tributary valley. The rock drain, constructed flush with the ground surface over a 9 meter (30 ft) diameter area, allows some drainage to occur even after an earth slide covers the drain inlet. The previous culvert inlet structure was blocked and covered by each earth slide which would create a mound over the culvert structure. The mounded slide material would effectively divert subsequent runoff around the culvert inlet and onto the facilities area.

The rock drain inlet at the North Fork inlet is constructed of 20 cm (8 in) diameter durable sandstone wrapped in a fabric filter with 5 cm (2 in) diameter filter rock over the fabric. (Keith Zobell, Utah Fuels Company, verbal communication, 1986)

The valley fill occupies approximately 10 hectares (25 acres). Material to construct the fill was obtained from the walls of the valleys at their confluence.

Reclamation at the Skyline Mine will consist of excavating the valley fill to remove all diversion culverts, and regrading the fill material to establish positive drainage through riprap protected channels located approximately in the original stream channels (Figure 10).

Colowyo

The Colowyo Mine located between Craig and Meeker, Colorado, is a surface dragline plus truck and shovel operation. The mine is situated at an elevation of approximately 2,225 meters (7,300 ft) above sea level in the Yampa River Basin of Western Colorado. Topography is rolling and moderately incised by surface drainage.

Excess spoil* from the mine operation is hauled to a 41,000,000 cubic meter (54,000,000 yd^3) valley fill established in an ephemeral stream drainage at the east edge of the mine area (Figure 11). The fill covers approximately 70 hectares (175 acres). There is no constructed drain in the fill. Surface runoff is directed around the fill by means of diversion channels.

Construction of the fill began in early 1977. Spoil material was first deposited in a 61 meter (200 ft) lift with subsequent lifts of 15 meters (50 ft) each. The spoil material was deposited by end dumping from trucks, thereby creating a natural sorting of rock material size through tbe action of gravity (Figure 12). Coarse rock was established at the bottom of the fill with decreasing rock size toward the top of each lift (Colowyo Coal Company, 1981). In this manner a natural rock drain of coarse spoil was established in the valley bottom. Monitor wells installed after completion of the eastern most side of the fill indicate the presence of a piezometric surface at depth in the fill, although there is no apparent

*Federal regulations define excess spoil as ". . . spoil material disposed of in a location other than the mined-out area. . ."

outflow at the toe of the fill (James Kiger, Colowyo Coal Company, verbal communication, 1986).

Disposal of excess spoil into the valley fill continued in an upstream direction as mining progressed until 1985, when the fill reached the pit area and normal backfilling of the pits continued. The ephemeral valley into which the spoil is being placed is unique because the mouth of the canyon is more narrow than the upstream areas. The natural configuration of the valley makes the disposal of spoil convenient because the graded outslope is at a narrow part of the valley, and runoff diversions can be localized to reduce construction costs.

The Colowyo Mine valley fill was begun before the enactment of SMCRA in 1977. State and Federal regulations promulgated subsequent to SMCRA require that valley fills be constructed in 1.2 meter (4 ft) lifts. Since the valley fill had already been significantly constructed prior to promulgation of the regulations, and the fill did not meet the "durable rock fill" requirements of the regulations, the fill was permitted under experimental practice regulations so that the industry might benefit from the fill's design. The burden to the operator of permitting the structure was increased because of special design parameters and mitigation plans necessary to permit under the experimental practices regulations.

The fill is a permanent structure constructed with a 3h:lv outslope from the mouth of the valley at an elevation of 2,009 meters (6,590 ft) to an elevation of 2,195 meters (7,200 ft). A 9 meter (30 ft) wide bench is placed at every 30 meters (100 ft) in elevation in the lower 90 meters (300 ft) of the structure. The benches are 6 meters (20 ft) wide in the upper 90 meters (300 ft) of the structure. Above 2,195 meters (7,200 ft) the spoil was placed at a flatter slope to blend with backfilling of the mine pit (Figure 13). (James Kiger, 1986, verbal communication; CTL/Thompson, Inc., 1979).

Surface drainage of the valley has been relocated in a channel constructed along the north abutment of the fill. The channel intercepts disturbed area runoff from above the spoil pile and conducts it to the toe of the fill and a siltation dam.

Eckman Park

The Eckman Park Mine is located south of Steamboat Springs, Colorado in the Yampa River Basin of Western Colorado (Figure 14). The mine is a surface dragline operation, situated at an elevation of approximately 2,130 meters (7,000 ft). Excess spoil is hauled to valley fill areas established during 1981 as permanent structures. At least two fills with durable rock underdrains were established to accomodate excess spoil (Figure 15). The rock underdrains are constructed along the natural drainage bottom with graded filter material and durable rock drain material. The fills are approximately 2,300,000 cubic meters (3,000,000 yd^3) in size over approximately 32 hectares (80 acres) each. The spoil is composed of sandstone with a specific gravity of 2.13 and shales with specify gravity ranging from 2.29 to 2.39 (Energy Fuels Corp., 1979).

The average unit weight of the loose spoil material is approximately 2,000 kg/m^3 (125 pcf). Gradation of the spoil was found to range from 87.1 percent passing a 20 cm (8 in) sieve, down to 2.3 percent passing 0.074 mm (No. 200 U.S. Standard Sieve Series) sieve.

Based on the above gradation, the drain filter material was sized ranging from 100 percent passing a 61 cm (24 in) size, to 3 percent passing a 0.074mm (No. 200) size.

The drain rock was sized ranging from 100 percent passing a 1 meter (3 ft) size to 10 percent passing a 0.5 meter (1.5 ft) size.

The drain rock and filter material are composed of durable, non-slaking sandstone quarried from the site. The spoil fill is compacted in 1.2 meter (4 ft) lifts, with an overall 3h:lv slope, and 9 meter (30 ft) wide benches every 15 meters (50 ft) in height.

Similar spoil-pile valley-fills are currently proposed for an additional mining area adjacent to the Eckman Park Mine, known as the Little Middle Creek Tract. The Little Middle Creek Tract is at an elevation of 2,400 meters (8,000 ft). A total of eight rock underdrains below excess spoil is proposed for the new area. The underdrains will extend from the toe of the spoil structure to the low wall of the box cut (Figure 16). Additional underdrains are proposed for any steep area encountered. All stream flows will be intercepted and diverted around the underdrains. The proposed underdrains are designed to transport ground water under the excess spoil, and prevent saturation of the fill.

All proposed underdrains at the Little Middle Creek Tract will have a minimum width of 3 meters (10 ft) and a minimum height of 1.2 meters (4 ft). No more than 10 percent of the drain material will be less than 30 cm (12 in) in size. The maximum size of the drain material will be 76 cm (30 in). To ensure that the underdrain does not clog, a graded filter of durable, non-toxic, on-site materials or an appropriate filter fabric is proposed to be constructed on all sides of the underdrain as well as the upslope end. (Golder Associates, 1986)

A total length of 2,210 meters (7,250 ft) in underdrains are re-proposed for the Little Middle Creek Tract. Each will be constructed 1.2 meters (4 ft) in height with a 3 meter (10 ft) wide top and 2h:lv side slopes.

CONCLUSION

Valley fills in Western Coal Mining are not always created as a means for excess spoil disposal. Some underground mines in Utah excavate adjacent valley walls to create a valley fill as a pad to support surface facilities. Western surface mine areas in moderately incised topography may use the incised drainage valleys as a convenient means of excess spoil disposal without inordinate surface disturbance. Although Western United States coal mining methods and geography are not as conducive to valley fill use as Eastern mines, the design and function of each valley fill is largely governed by regulatory constraints brought on through the enactment of the SMCRA on August 3, 1977.

ACKNOWLEDGEMENTS

The author is deeply grateful to the employees and management of the following coal mining companies for their cooperation in development of this paper: Coastal States Energy Company and their affiliates Southern Utah Fuel Company and Utah Fuel Company, Utah Power and Light Company, Colowyo Coal Company, and the Yampa Valley Coal Corporation. In particular, I am grateful for the assistance provided by the following individuals: Messrs. Wes Sorenson, Chris Shingleton, Val Payne, Keith Zobell, Scott Childs, James Kiger, Michael Kondelis, and Pat Sollars. I am also grateful to the Western Field Operations Office of the Office of Surface Mining Reclamation and Enforcement, and Mr. D. M. Galbraith of the Ministry of Energy, Mines and Petroleum Resources in the Province of British Columbia for the opportunity to prepare this paper.

REFERENCES

Code of Federal Regulations, 1985, 30 CFS Part 700 to End: U.S. Government Printing Office, Washington.

Coastal States Energy Co., 1979, Skyline Mine Permit Application: For public inspection at the Office of Surface Mining Reclamation and Enforcement, Denver, Colorado; and the Division of Oil, Gas and Mining, Salt Lake City, Utah.

Code of Colorado Regulations, 1980, Regulations of the Colorado Mined Land Reclamation Board for Coal Mining: Mined Land Reclamation Div., Colorado Dept. of Natural Resources.

Colowyo Coal Co., 1981, Colowyo Mine Permit Application: For public inspection at the Office of Surface Mining Reclamation and Enforcement, Denver, Colorado; and the Colorado Mined Land Reclamation Division, Denver, Colorado.

CTL/Thompson, Inc., 1979, Consultation Slope Stability Colowyo Streeter Canyon Fill, near Meeker, Colorado: For Office of Surface Mining, Job No. 5368.

Energy Fuels Corp., 1979, Permit Application for the Eckman Park and Energy Mines No. 1 & 2: For public inspection at the Office of Surface Mining Reclamation and Enforcement, Denver, Colorado; and the Colorado Mined Land Reclamation Division, Denver, Colorado.

Golder Associates, 1986, Slope Stability Analyses Excess Spoil from Box Cuts Little Middle Creek Tract: For, Colorado Yampa Coal Company, Routt County, Colorado, 863-2010.

Sergent, Hauskins and Beckwith, 1986, Submittal of Drainage Plan and Slope Stability for Reclamation for Convulsion Canyon Mine, ACT/041/002, No. 2 No. 3, and No. 14, Sevier County, Utah: For, Southern Utah Fuel Co., Job No. E83-2022.

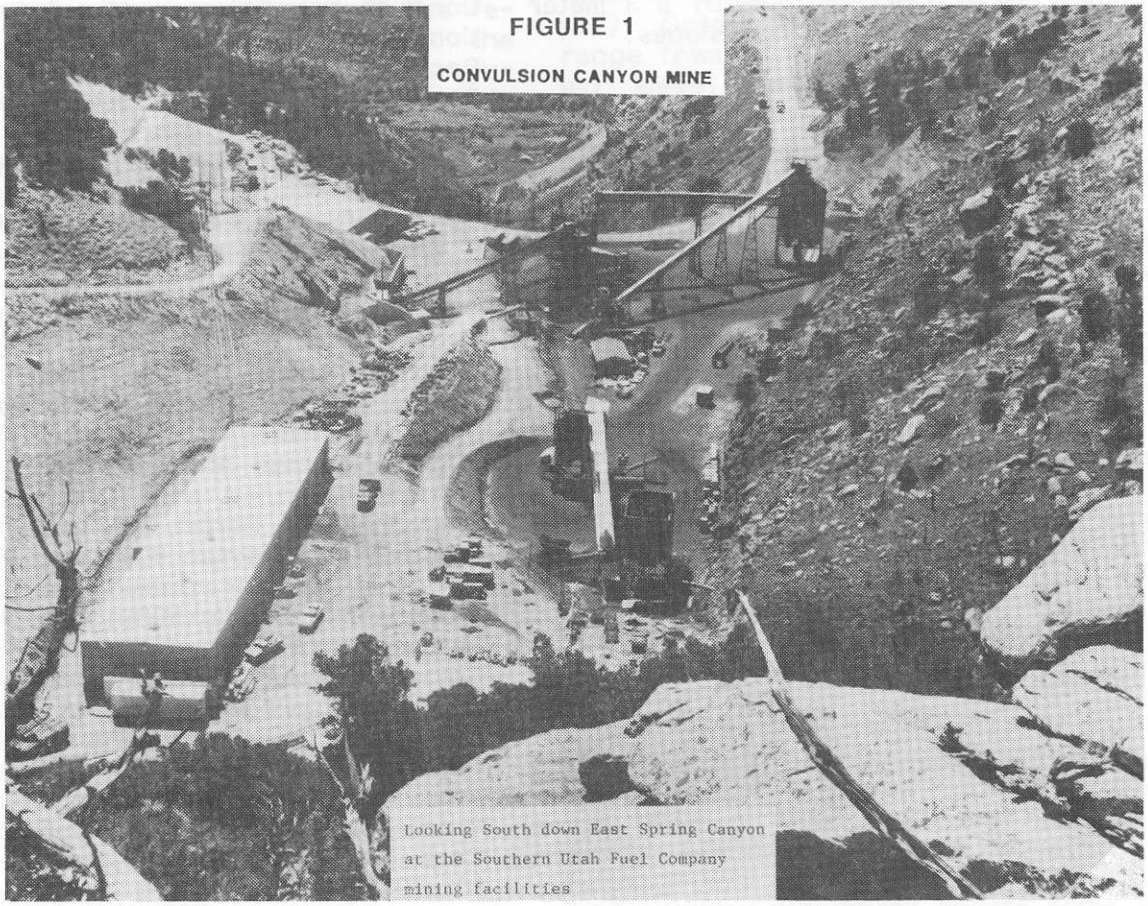
Southern Utah Fuel Co., 1980, Convulsion Canyon Mine Permit Application: For public inspection at the Office of Surface Mining Reclamation and Enforcement, Denver, Colorado; and the Division of Oil, Gas and Mining, Salt Lake City, Utah.

Utah Power and Light Co., 1985, Revised Deer Creek Mine Permit Appliction: For public inspection at the Office of Surface Mining Reclamation and Enforcement, Denver, Colorado; and the Division of Oil, Gas and Mining, Salt Lake City, Utah.

Utah Underground Mining Code, 1982, Coal Mining and Reclamation Permanent Program: Utah Division of Oil, Gas and Mining.

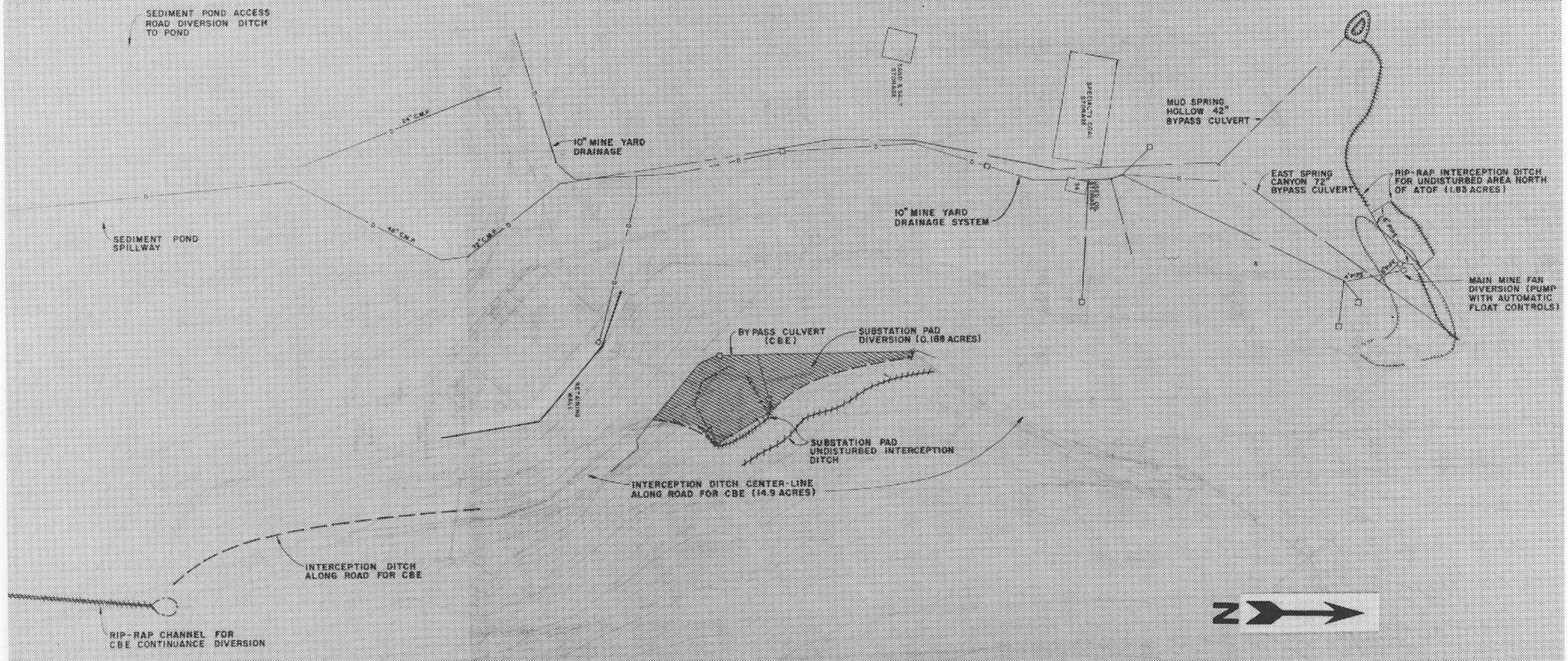
FIGURE 1

CONVULSION CANYON MINE



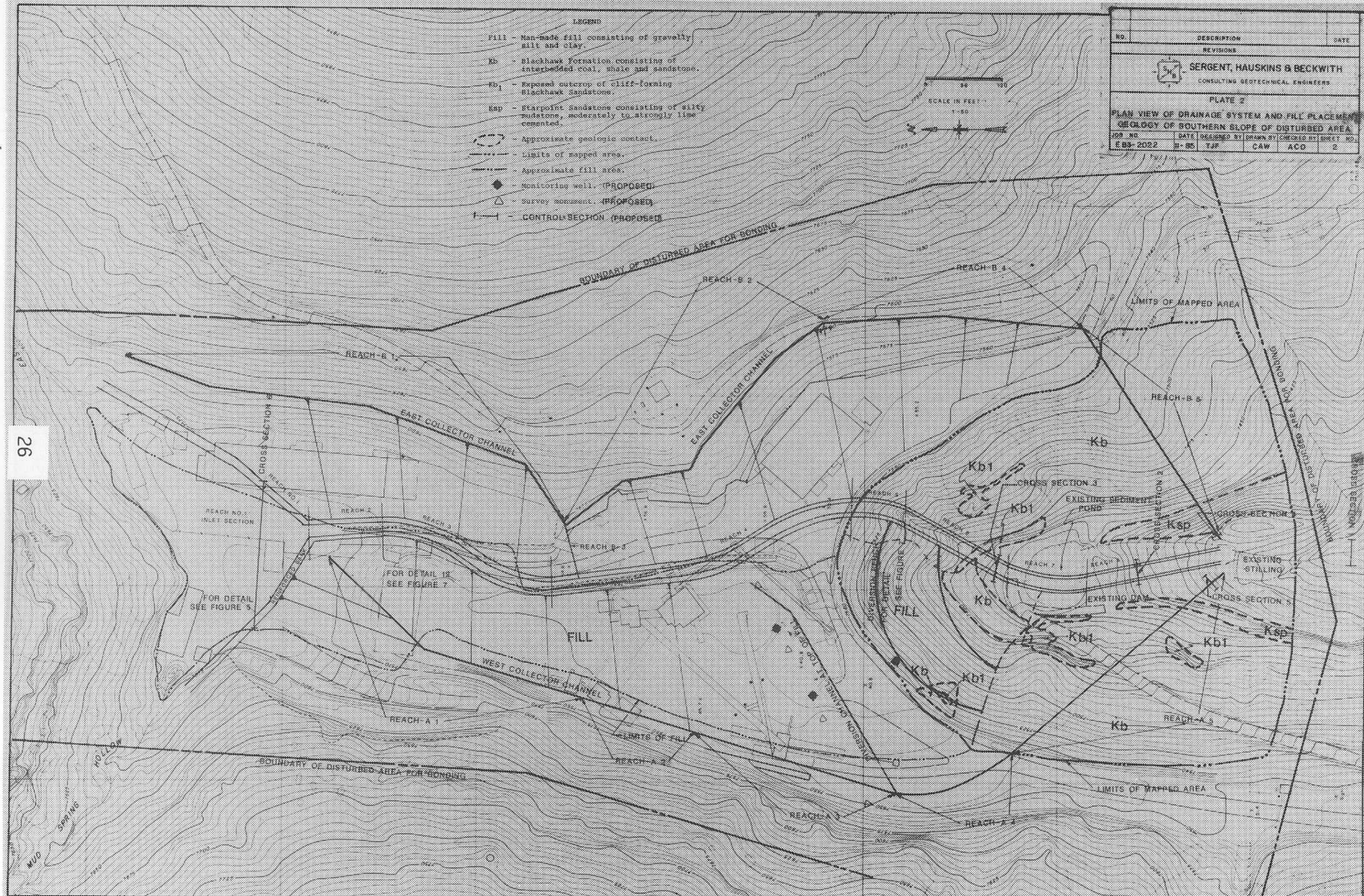
Looking South down East Spring Canyon at the Southern Utah Fuel Company mining facilities

FIGURE 2



CONVULSION CANYON MINE CULVERT LAYOUT

FIGURE 3



CONVULSION CANYON MINE RECLAIMED STREAM CHANNEL LAYOUT

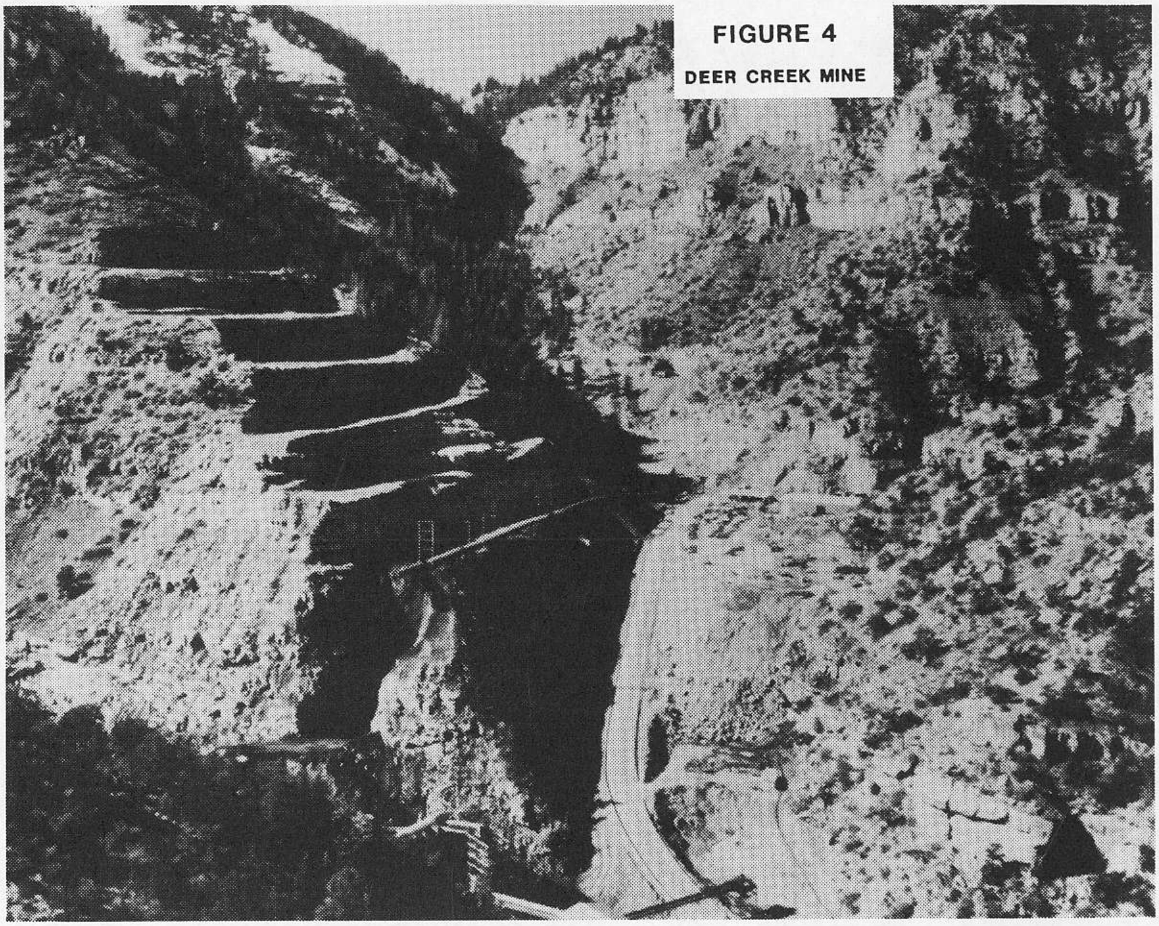
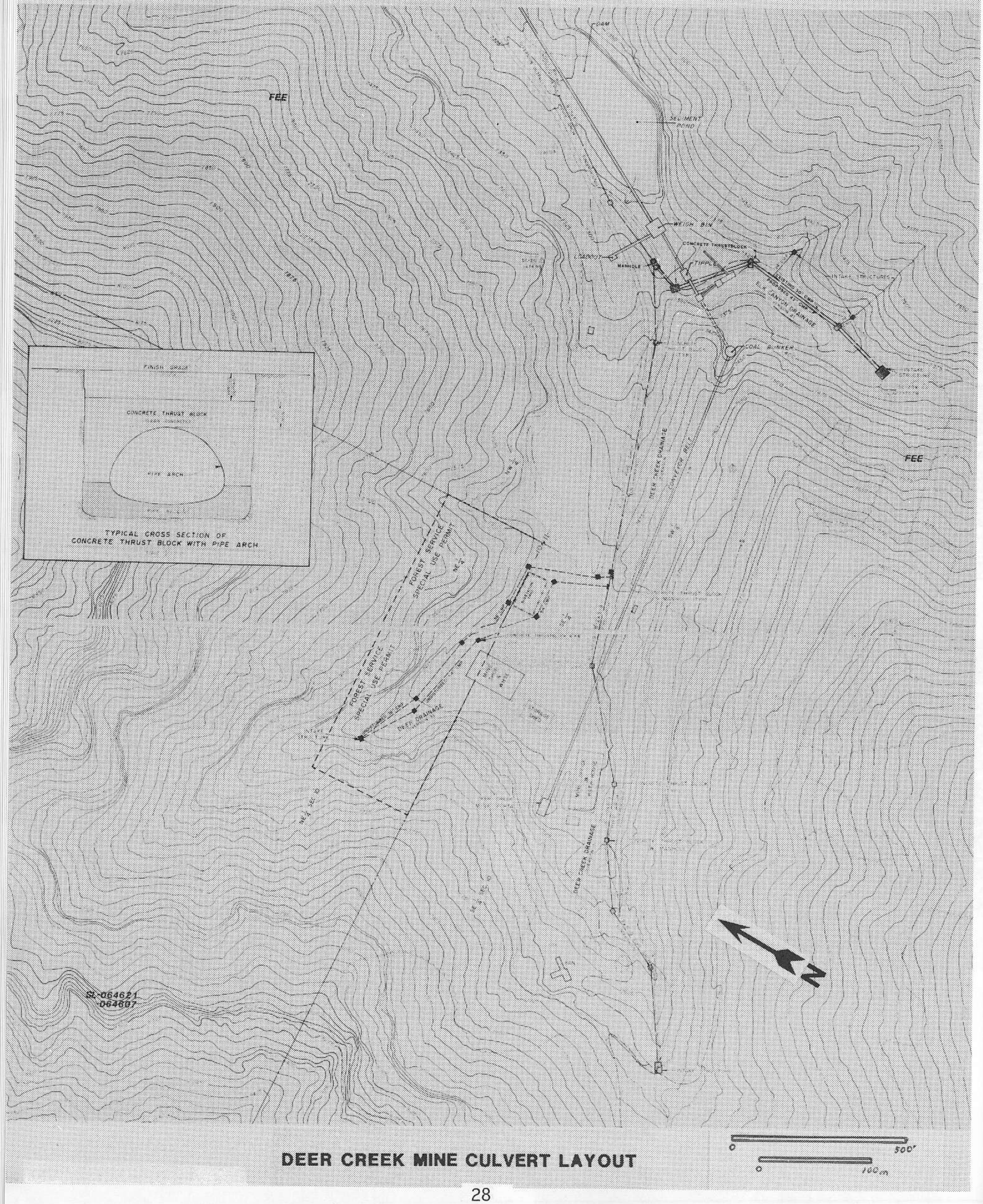


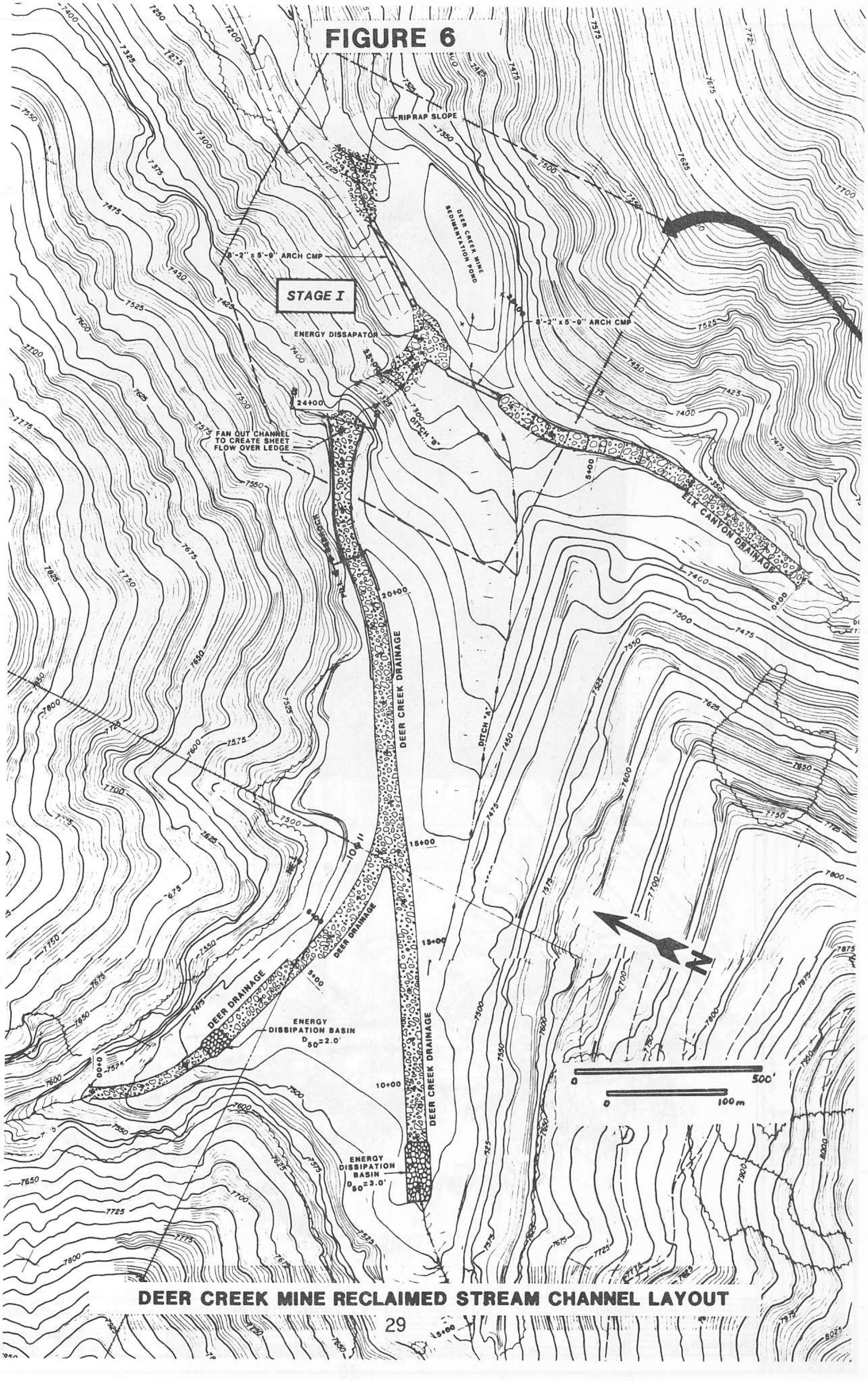
FIGURE 4
DEER CREEK MINE

FIGURE 5



DEER CREEK MINE CULVERT LAYOUT

FIGURE 6



DEER CREEK MINE RECLAIMED STREAM CHANNEL LAYOUT

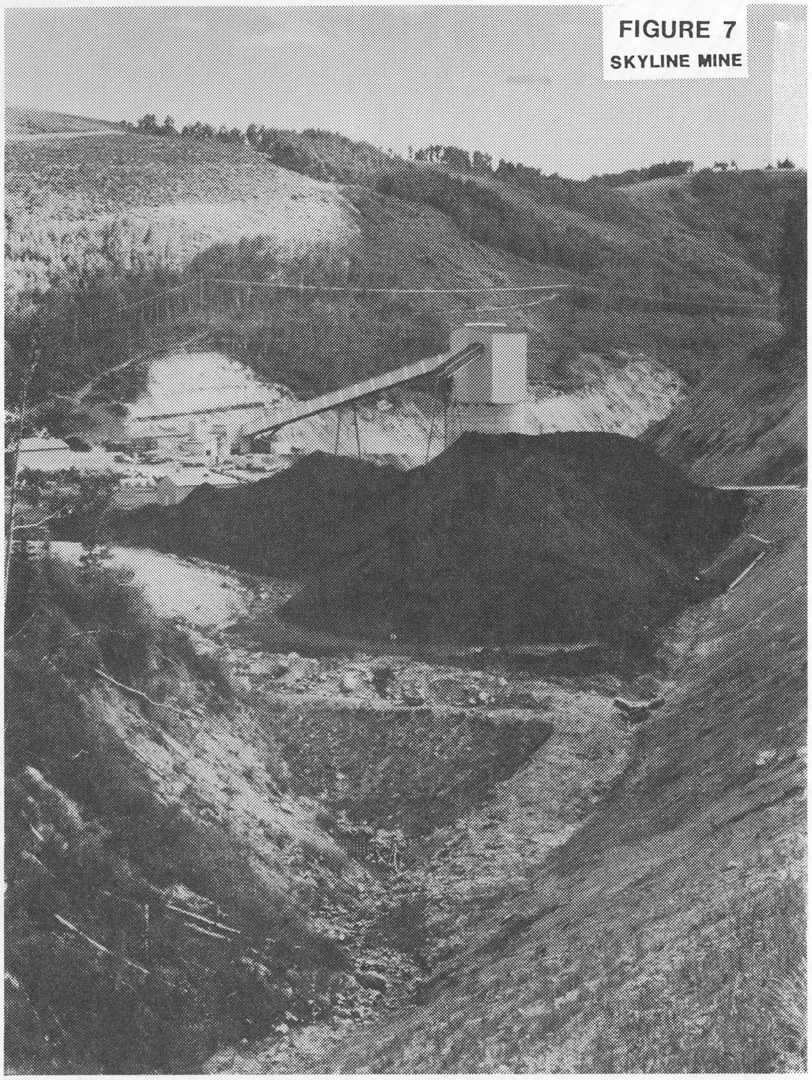


FIGURE 7
SKYLINE MINE

FIGURE 8

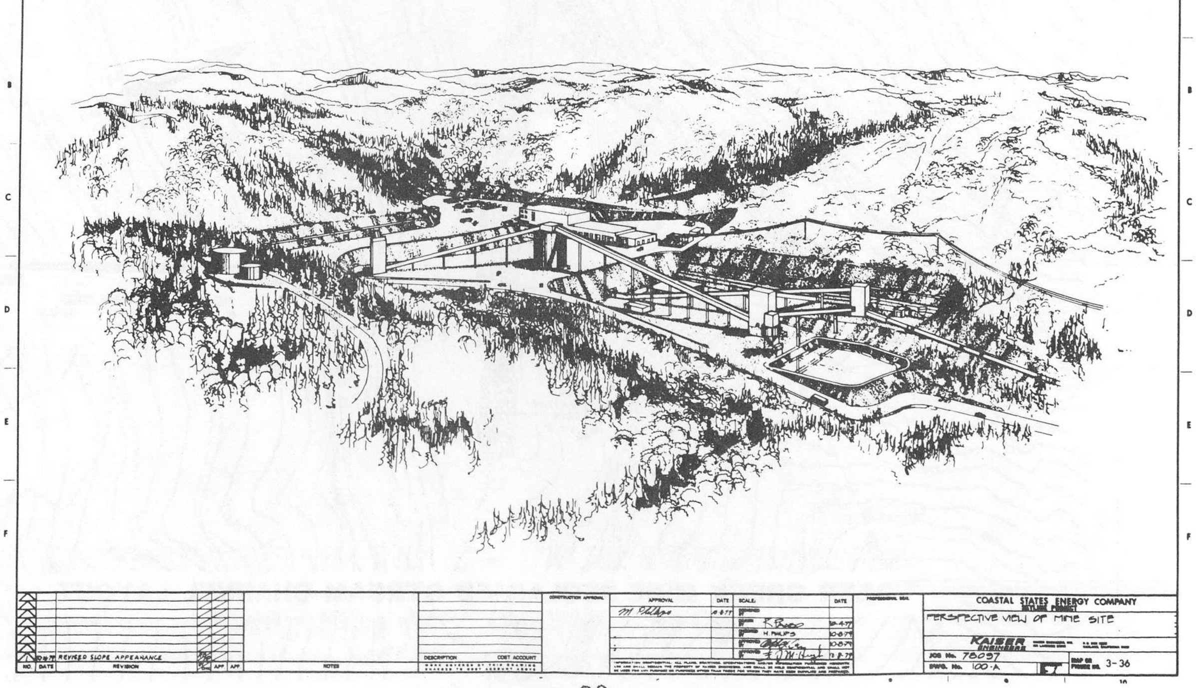
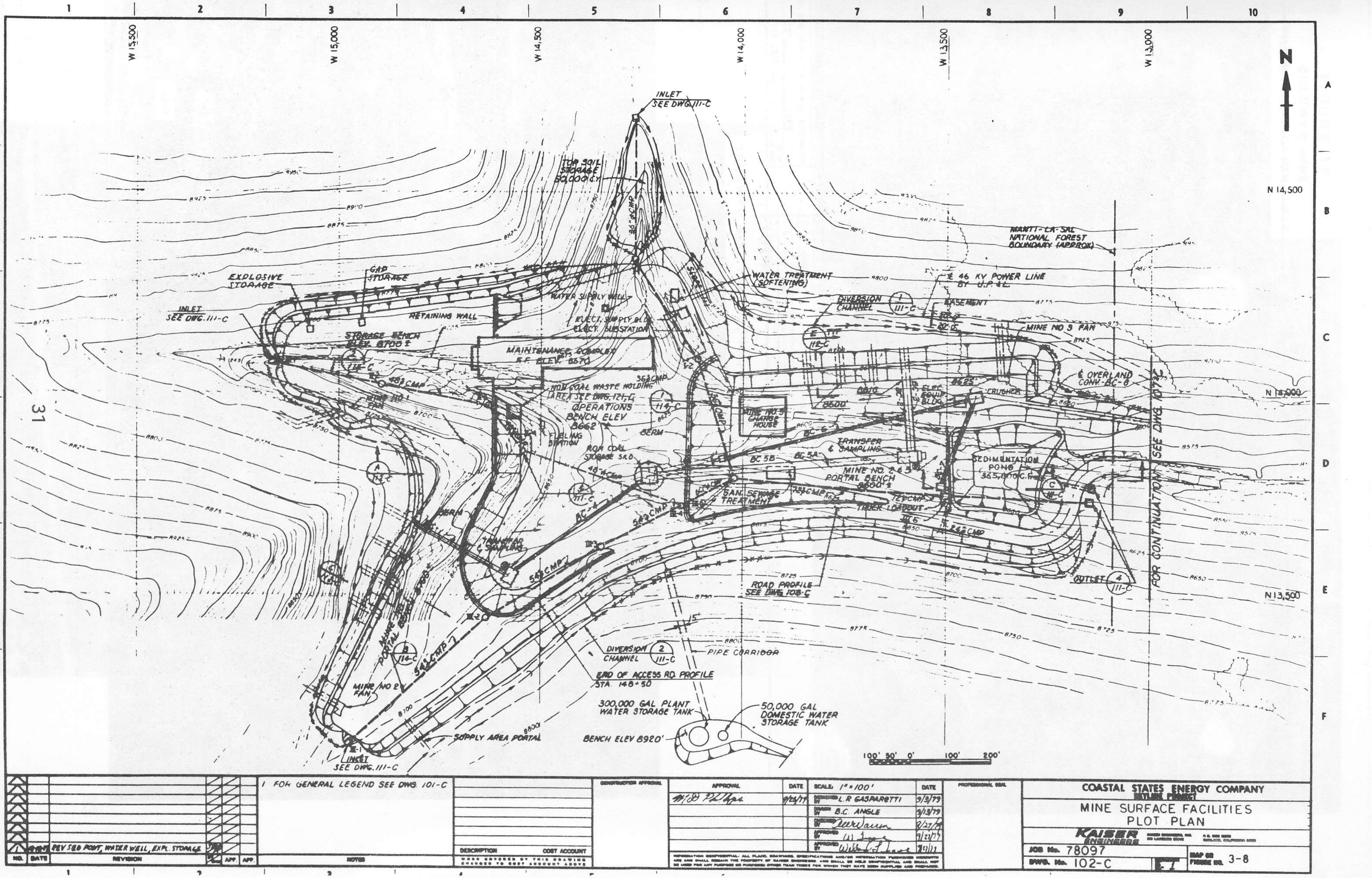
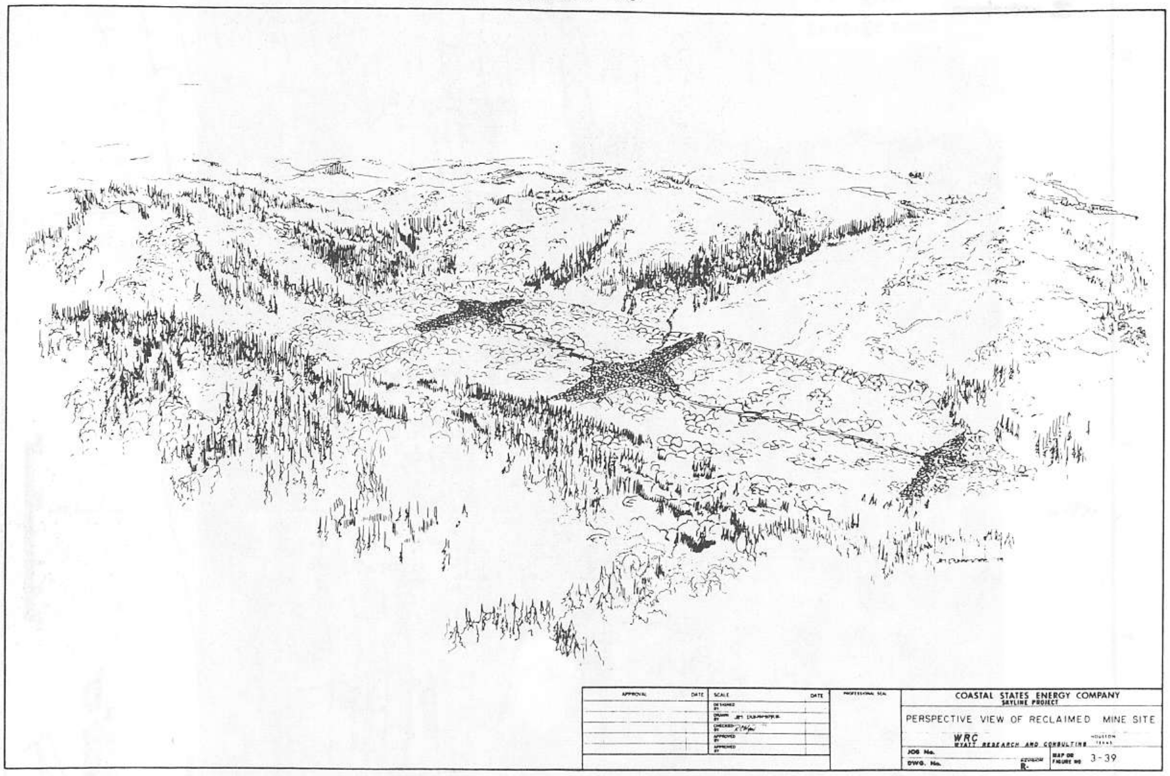


FIGURE 9



N
W 15,500
W 15,000
W 14,500
W 14,000
W 13,500
W 13,000
N 14,500
N 14,000
N 13,500
INLET SEE DWG 111-C
TOP SOIL STORAGE 50,000 C.Y.
MANTI-LA-SAL NATIONAL FOREST BOUNDARY (APPROX)
EXPLOSIVE STORAGE
CAP STORAGE
INLET SEE DWG. 111-C
RETAINING WALL
WATER SUPPLY WELL
ELECT. SUPPLY BLDG
ELECT. SUBSTATION
WATER TREATMENT (SOFTENING)
DIVERSION CHANNEL
46 KV POWER LINE BY U.P.&L.
EASEMENT
MINE NO 3 FAN
STORAGE BENCH ELEV 8700 ±
MAINTENANCE COMPLEX F.F. ELEV 8670
NON COAL WASTE HOLDING AREA SEE DWG. 121-C
OPERATIONS BENCH ELEV 8662' ±
BERM
FUELING STATION
ROM COAL STORAGE SILO
MINE NO. 5 CHANGE HOUSE
TRANSFER & SAMPLING
CRUSHER
OVERLAND CONV. BC-8
SEDIMENTATION POND 365,000 C.F.
MINE NO. 2 & 3 PORTAL BENCH 8600' ±
SAN. SEWAGE TREATMENT
TRUCK LOADOUT
FOR CONTINUATION SEE DWG. 101-C
OUTLET
ROAD PROFILE SEE DWG 108-C
DIVERSION CHANNEL
PIPE CORRIDOR
END OF ACCESS RD. PROFILE STA. 148+50
MINE NO 2 FAN
300,000 GAL PLANT WATER STORAGE TANK
50,000 GAL DOMESTIC WATER STORAGE TANK
BENCH ELEV 8920'
SUPPLY AREA PORTAL
INLET SEE DWG. 111-C
100' 50' 0' 100' 200'
1 FOR GENERAL LEGEND SEE DWG. 101-C
REV. SED POND, WATER WELL, EXPL. STORAGE
NO. DATE REVISION APP. APP.
NOTES
DESCRIPTION COST ACCOUNT
APPROVAL DATE
SCALE 1" = 100'
L.R. GASPARETTI 9/3/79
B.C. ANGLE 9/13/79
COASTAL STATES ENERGY COMPANY
MINE SURFACE FACILITIES PLOT PLAN
KAISER ENGINEERS
JOB No. 78097
DWG. No. 102-C
FIGURE NO. 3-8

FIGURE 10



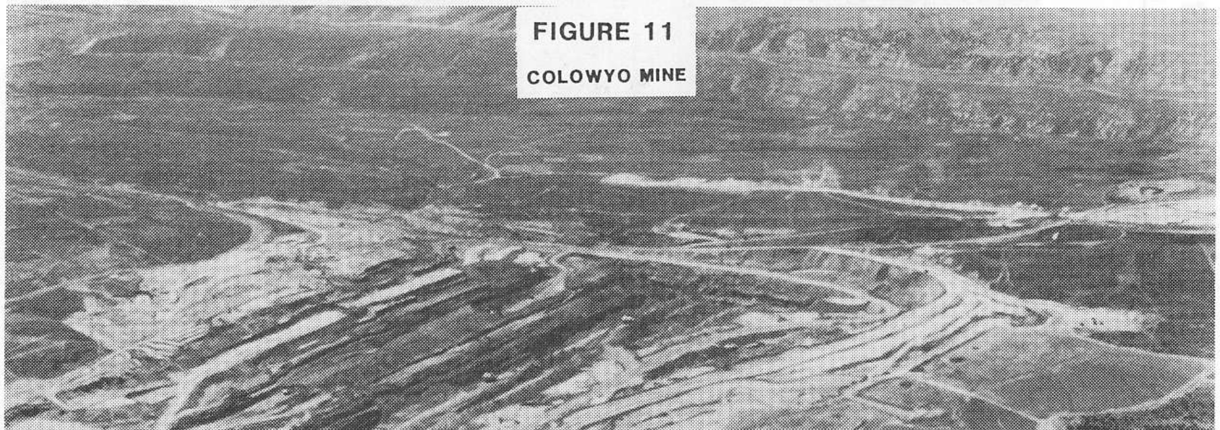


FIGURE 11

COLOWYO MINE

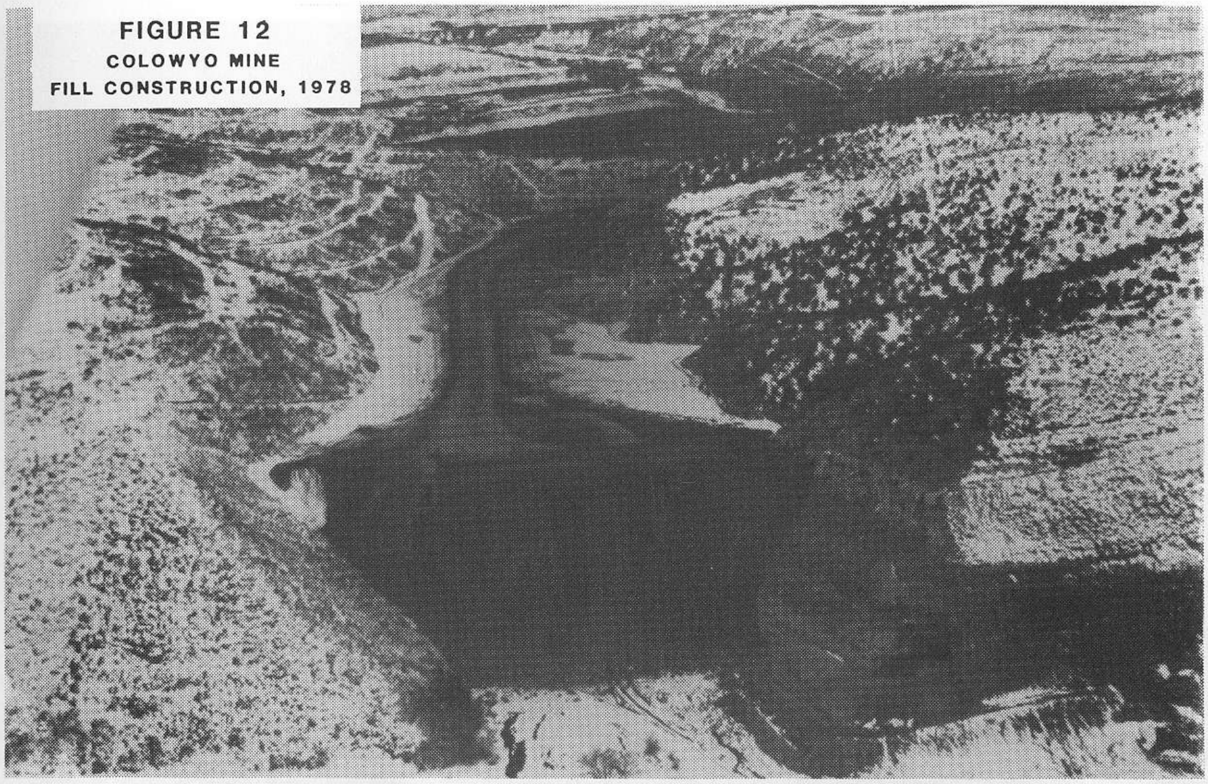


FIGURE 12
COLOWYO MINE
FILL CONSTRUCTION, 1978

FIGURE 13
COLOWYO MINE VALLEY FILL, 1985

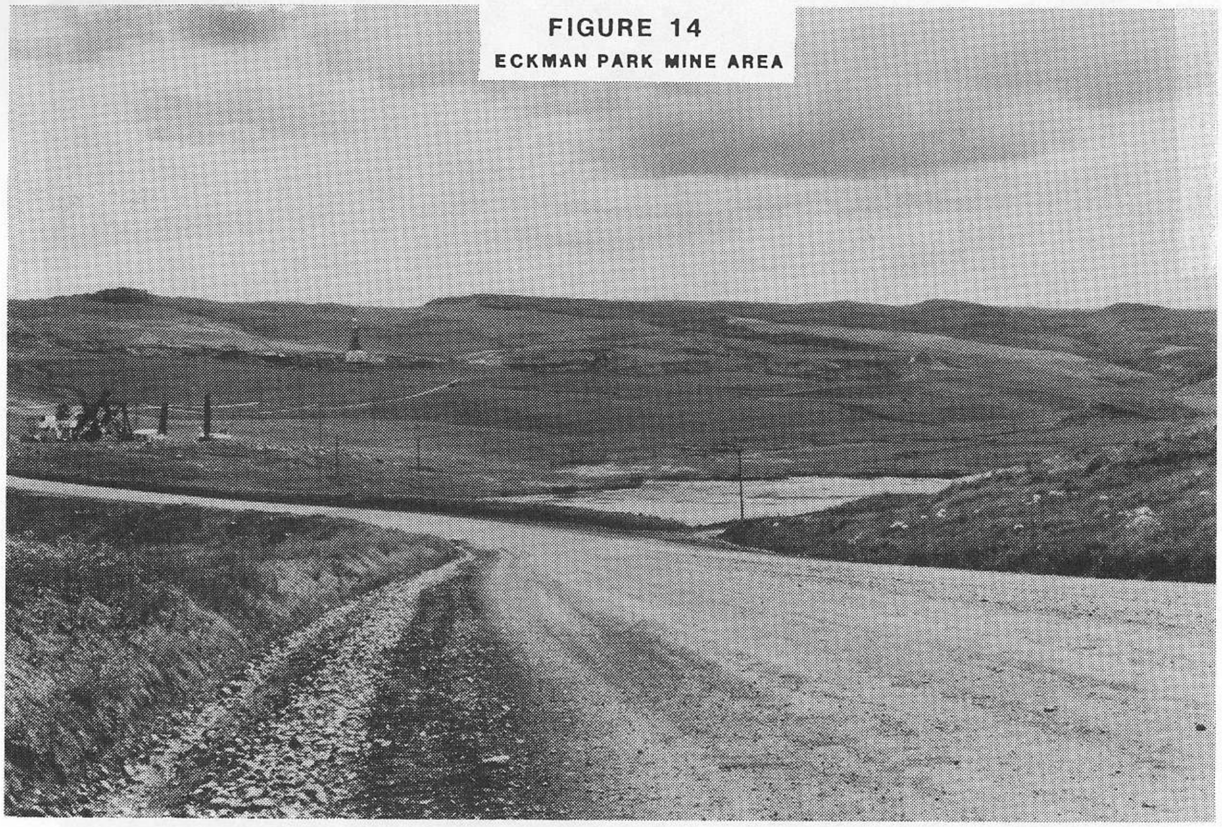
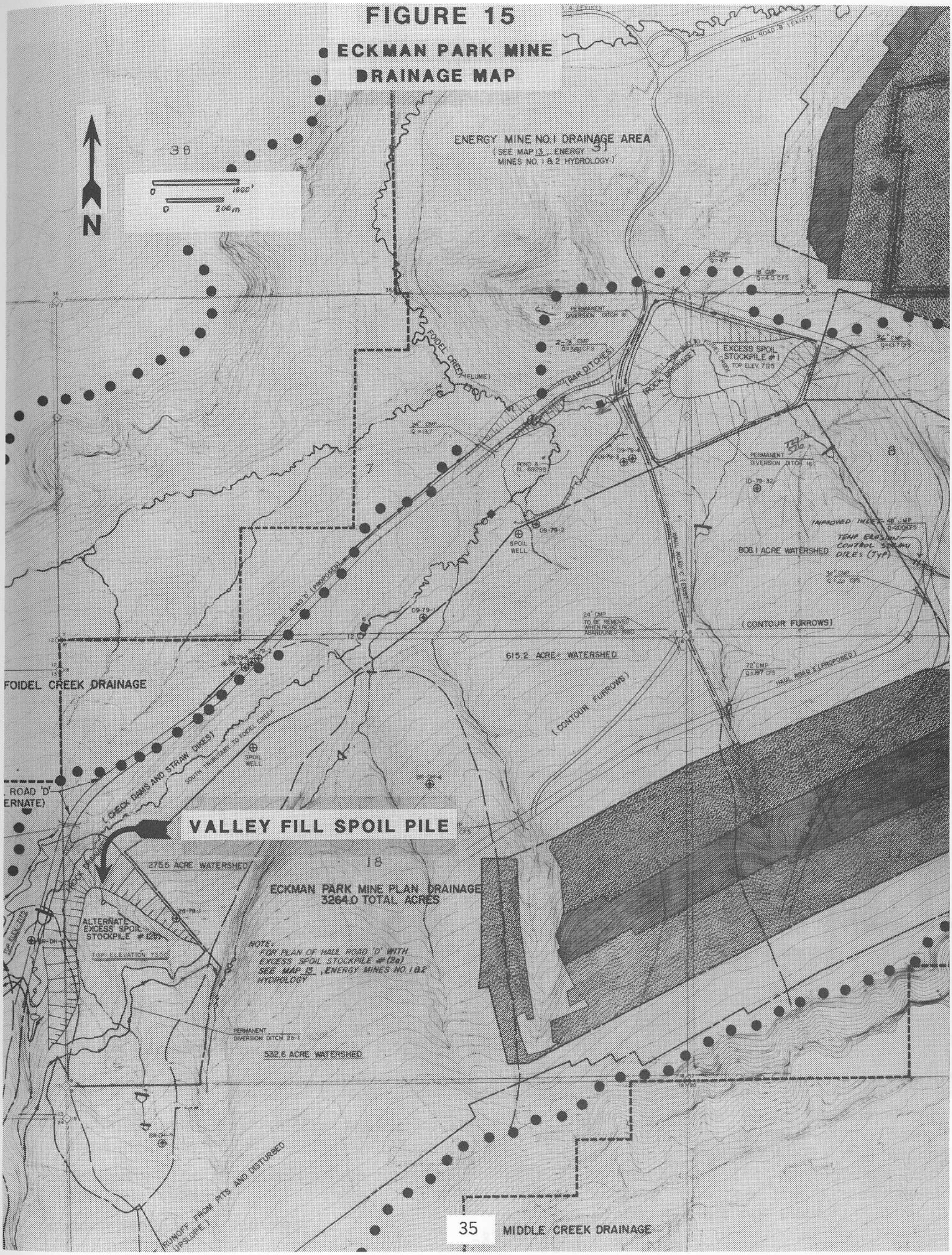


FIGURE 14
ECKMAN PARK MINE AREA

FIGURE 15

ECKMAN PARK MINE DRAINAGE MAP



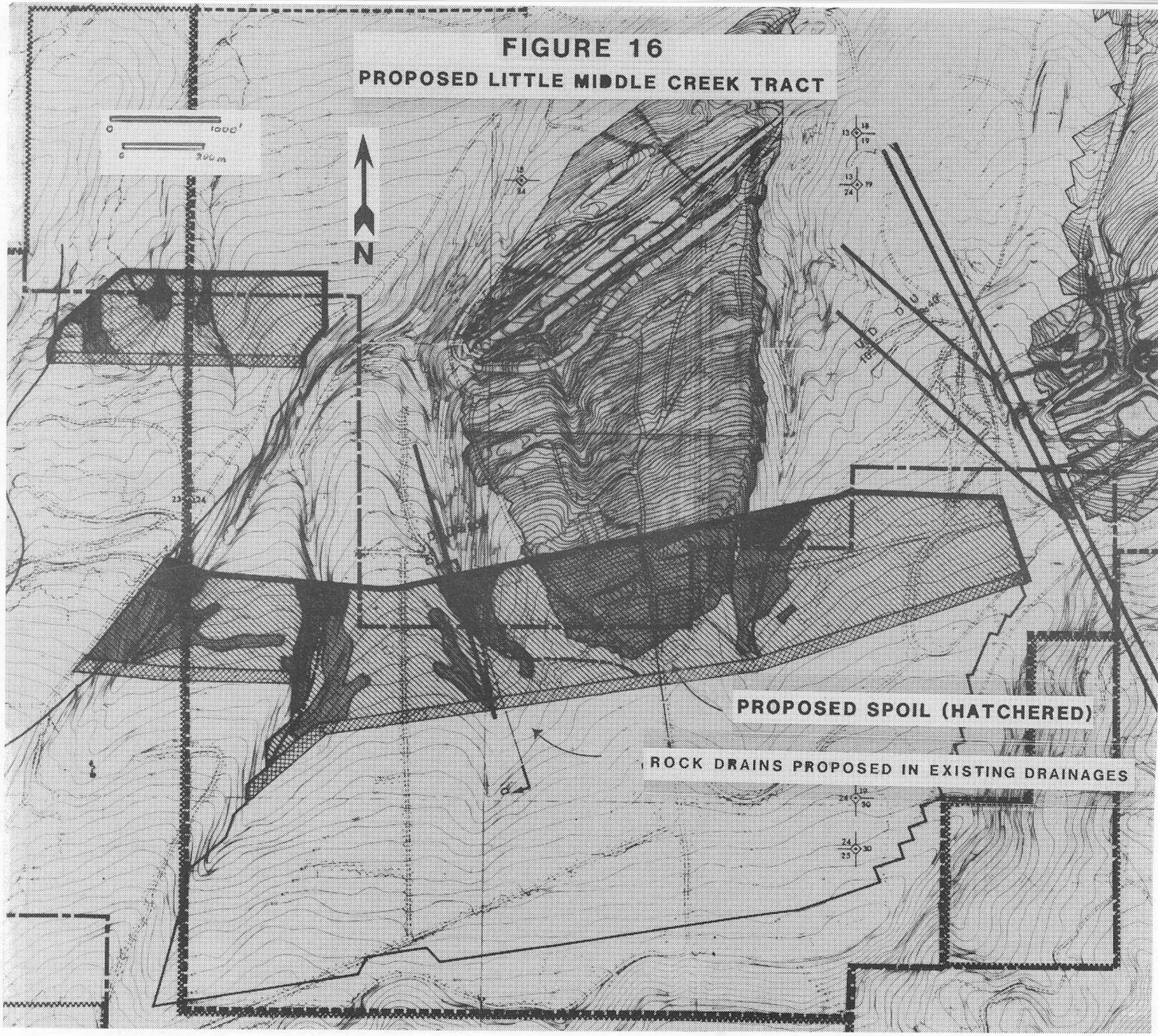


FIGURE 16
PROPOSED LITTLE MIDDLE CREEK TRACT

PROTECTION OF ROCKFILL DAMS AND COFFERDAMS AGAINST OVERFLOW AND THROUGHFLOW - THE AUSTRALIAN EXPERIENCE

by
J. D. Lawson
Professor of Civil Engineering
University of Melbourne
Australia

INTRODUCTION

A small unconventional rockfill dam was built in Tasmania, Australia, in the 1950s and reported by Wilkins.(1) This started an interesting and productive chain of events which included research at several places on the hydraulic and stability characteristics of rock banks subjected to overflow and through-flow, and culminated in the wide-spread use of steel-mesh and anchorage protection of cofferdams and partly-completed large conventional rockfill dams against unravelling and slip failures of the downstream rock slope. 50 such structures have been built in the 40 years to 1982, with 41 of them in Australia in the 20 years since 1963. (31)

An attempt is made in this paper to describe the developmants in theory and in practice, with case studies.

It should be noted that early developments in the techniques of passing weter over rockfill dams and through filtering dams and causeways (used instead of bridges and culverts) took place in the U.S.S.R.(2)

ROCKFILL DAMS WITH INBUILT SPILLWAYS

The unique feature of this type of dam is that the permanent spillway is contained within the rockfill bank, and flows pass through the main body of the rockfill (Figure 1) thus eliminating conventional spillways and energy dissipation structures.

The design problems include -

- the prediction of water surface profiles,
- pressure and velocities throughout the dam,
- stability of the structure against over-stopping,
- erosion of the downstream face from seepage flow, and
- deep-seated slip failures.

By laboratory testing of model banks 1, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14 and numerical analysis 12, 13, 16, 17, 18, 19 design criteria have been derived for proportioning a rockfill dam with inbuilt spillway capable of safely passing a flood flow of given magnitude.

It is relevant to note that the discharge capacity of such a structure is poor compared with a free overfall spillway (Q @ h compared with Q @ h3/2) and as overtopping of the rockfill is not acceptable, it follows that if the design discharge is large, then the height of rock above the buried spillway crest may preclude the use of the technique for economical reasons, unless operated in conjunction with an emergency overfall spillway which caters for the infrequent lerge flows.

Laughing Jack Marsh Dam (Figure 2) in Tasmania, discussed by Wilkins(1), is a small 12.2m high rockfill structure with sloping upstream cley core, the central part of which is lower and protected with a concrete slab to provide a buried-in-rock rectangular weir to accept overflows. For a maximum flood of $28.3 m^3/sec^{-1}$ the rise in reservoir level above the crest was estimated to be 1.5m and the critical depth over the broadcrested buried weir to be 0.5m with a velocity through the voids of $2.2 m/sec^{-1}$.

Special design considerations of this dam include:

- provision of adequate freeboard of rock above the crest to avoid overtopping (including wave action),
- removal of fines from the body of rockfill subjected to freefall and throughflow,
- provision of large rocks to above the phreatic surface at the base of the downstream slope - rocks of a size that would not be moved by the throughflow and free surface flow on the slope. Steel mesh protection was not used.

OVERTOPPED PARTLY-COMPLETED ROCK-FILL DAMS

The practice of allowing floods to pass over conventional rockfill dams during the construction period has become popular in recent years, especially in Australia. (20,21, 22,31) Provided the downstream face of the structure is suitably protected against surface erosion (unravelling) and deep-seated slip failures, large floods can be safely passed over and through the rockfill, thus effecting economies from decreased costs of diversion works. The unravelling phenomenon is usually associated with a combination of outflowing and overflowing water on the downstream rockfill slope which becomes a seepage face. In this case, unravelling depends on the geometry and flow characteristics of the rockfill which govern the extent of the seepage face. It is, of course, well known that stable submerged slopes exist for granular materials(23) but these are mild slopes compared with conventional rockfill dams which, for economic reasons to save rock volume, have much steeper slopes. It is important to recognize, however, that for an assemblage of unbonded particles, the danger of uncontrolled collapse caused by overtopping or throughflow is real and must be adequately taken into account in design. Many failures of unprotected steep banks of rockfill are recorded in the literature.(12,20, 24) The laboratory model testing and numerical analysis of overtopped rockfill banks logically followed the inbuilt spillway studies associated with Figure 1.

THEORY

A study of rockfill dams embraces -

- an understanding of the basic laws governing flow, the development of field equations from these laws, and the interpretation of flow behaviour through model and prototype dams,
- computational techniques, including finite difference and finite element solutions of field problems,
- methods of analysis for stability of rockfill slopes.

REGIMES OF FLOW

At the outset, it is generally accepted that there are two principal types of flow, laminar and turbulent. For most cases of flow through soils of practical interest, the flow velocities and particle sizes are small enough for laminar conditions to apply and for Darcy's law to describe adequately the flow regime. However, as the flow velocities, particle sizes, and Reynolds numbers (variously defined) increase, Darcy's law becomes inaccurate.(25,26,27) The close approximation to the linear relationship between velocity and energy gradient, as expressed by Darcy's law, no longer applies and another flow law is required.

Turbulent flow through porous media is encountered in several engineering situations. Cases in civil engineering include river engineering applications of rockfill, well flows, and flows through filters (for example, water and sewage treatment). The chemical engineers are involved with flow of gases in packed beds; petroleum engineers with operation of gas wells; agricultural engineers with drying of grain in storage bins(19); mining engineers with rock drains.

Numerous attempts have been made to describe the full range of flows in porous media by development of a correlation between energy loss and particle properties, in an analogous fashion to the Moody resistance diagram for pipe flow, but this correlation is complicated by the many other parameters influencing flow in porous media. (27,28)

BASIC FLOW LAWS

Many different hypotheses and theories have been postulated to describe the laws of flow through porous media.(1,3,25,27,29) Of the many empirical forms of energy loss equation that have been derived, two of the most common are of the form

$$S = a'v + b'v^2 + c'v^3$$

and $$S = a\,v^n$$

where v = discharge velocity

$a = \frac{1}{k}$, where k = coefficient of permeability, or transmission constant

S = gradient of piezometric head

and a', b', c', a and the exponent n vary with the particular porous media and the flow conditions pertaining.

By analogy with pipe flow, it could be expected that exponent n will vary from 1 for laminar flow conditions (satisfying Darcy's law, $v = kS$) to 2 for fully turbulent conditions.

As mentioned previously, a generalized correlation of energy loss data in the form of a diagram of friction factor versus Reynolds number is not yet available. Instead, presently, energy loss behaviour through a particular porous medium is often expressed in the form of a simple exponential relationship, developed from tests on the material, under conditions which simulate the field situation. It should be emphasized that these exponential formulae are only good approximations over certain limited flow ranges; the constants n and a being functions of other variables, including the rate of flow, particle shape, surface roughness and size distribution.(12)

TURBULENT FLOW FIELDS

Analytical solutions of turbulent flow fields may be derived within a system of defined boundaries by development the differential equation for the piezometric head and flow distribution, using a flow law (such as an exponential energy loss equation of form $S = av^n$) in conjunction with the continuity equation, and assuming steady two-dimensional flow.

Expressed in Cartesian derivatives, a generalised field equation has been developed(12) of the form

$$(\phi_{xx} + \phi_{yy})(\phi_x^2 + \phi_y^2) + \left(\frac{1-n}{n}\right)$$
$$(\phi_x^2\phi_{xx} + 2\phi_x\phi_y\phi_{xy} + \phi_y^2\phi_{yy}) = 0$$

where

ϕ = a scalar function representing S/a

ϕ_x, ϕ_y = derivatives in x and y directions

a, n = coefficients, constant within any local region of flow

For laminar flow conditions (n = l) the equation reduces to Laplace's equation

$$\phi_{xx} + \phi_{yy} = 0$$

Further, if a can be consideration constant, then ϕ can be replaced by the piezometric head h and the distribution of piezometric head within the system is a function only of the exponent n.

COMPUTATIONAL TECHNIQUES

Finite-Difference Solutions of the above generalized field equation have been developed using square grids(13) and rectangular computational units.(16) In each case, the total piezometric head at each node is determined by relaxation methods to give residuals at every point below a certain acceptable level when the field equation is satisfied throughout the region.

These finite-difference equations have their limitations, because they can be used only in problems where all boundaries exactly coincide with node points and elements of the grid.

In the general case of seepage flow, few boundaries are likely to coincide with node points since all boundaries in free surface problems are curved or sloping.

Finite Element Solutions, applied to non-linear flow through rockfill, have been developed(16,18,19) using triangular elements which are well suited to anisotropic fields and curved boundaries (Figure 3).

Using the variational principle in the calculus of variations as applied to a region divided into triangular elements, with an assumed linear head distribution within each element, then solutions in terms of piezometric head are available. Boundaries of the flow fieid, such as seepage faces, free surfaces, impervious faces are readily dealt with by selecting nodes of the triangular net to coincide with the boundaries and assigning appropriate piezometric heads to these node points.

Computer programs are available(19) to deal with flow through and over rockfill banks of various geometries. A typical graphical output of streamlines, equipotentials and isobars for a not-unusual 36° rockfill bank is depicted in the set of three diagrams (Figure 4).

It will be noted that the turbulent flow net has a higher phreatic surface and a longer seepage face than the laminar flow net, reflecting the higher discharge and higher exit gradients for turbulent flow. The isobar pattern shows that turbulent flow causes higher pore pressures in most parts of the dam.

STABILITY OF ROCKFILL SLOPES

Making use of the turbulent flow analysis combined with a stability analysis(30) of overtopped banks, a series of downstream bank angles between 33° and 45° have been analyzed for a range of tailwater levels(16). The envelope to all slip circles having a factor of safety of 1.0 can be closely approximately by a line drawn parallel to the downstream face and at a horizontal distance from the face equal to two-thirds of the bank height (Figure 5). This gives a reasonable estimate of the zone requiring tension reinforcement to ensure adequate resistance to failure by overtopping or throughflow.

It is of interest to note that

- for given factor of safety, the amount of reinforcing necessary, if taken to the slip circle envelope, is independent of bank angle. Conversely, for similarly reinforced banks, the factor of safety is independent of the bank angle, hence it is economically advantageous to use as high a bank angle as possible,
- the exponent n in the flow law (when varied from n = 1 to 2) has little effect on stability for either a free surface slope flow or an overtopped bank (see Figure 6),
- the height of the tailwater has no effect on the stability. Increases in tailwater level merely cause different slip circles at a higher elevation (with about the same factor of safety) to become critical,
- the density of the rock material has a significant effect on the stability of a slope.

It should be recognized that as well as the Bishop slip circle method, used for the above analyses, other techniques for slope stability determination have been examined, such as the sliding wedge analysis with modifications(31).

The stability of rockfill banks subjected to both overflows and throughflows (creating free surface seepage flows down the downstream slope) has been extensively examined using hydraulic model tests on embankments of scaled down rockfill.(3,9,31) Such experimental testing suffers from scaling problems with respect to size, shape and density of model particles required to reproduce similar flow, pore pressure, and stability characteristics. To model the mesh protection and anchor reinforcement is another problem. However, useful results (qualitative if not quantitative) are often possible, particularly if models of several scales (say, 1:50 and 1:100) are used and extrapolation techniques employed.

Whether stability studies are carried out experimentally or analytically, in all cases of overtopping or downstream slope seepage flows (see Figure 6), the critical failure mode is invariably unravelling of the surface rock particles. The design procedure for stability of a rockfill slope subjected to flow must therefore be

- provision of adequate mesh protection to prevent unravelling of the surface particles due to the flow, and
- tension (or anchor) reinforcement to guard against deep-seated slips, as well as retain the surface mesh in place.

PRACTICE

The types of mesh protection and anchor reinforcement that have been employed in practice are many and varied.(31,20) Most of what follows has been extracted from A.N.C.O.L.D. Report(31) and other referenced A.N.C.O.L.D. Bulletins.

Examples of surface mesh used on the downstream face include

- 20mm bars with a mesh size 1200mm by 300mm (between horizontal bars),
- 10mm and 7mm bars with a mesh size 75mm by 200mm resp. together with 25mm bars in mesh 1500mm by 2800mm,
- 4mm wire chain-link fencing mesh of 50mm square opening with 22mm bars at 1350mm vertical spacing,
- cylindrical gabions 900mm dia. by 2440mm long (and 940mm dia. by 2400mm) made of 4mm chain-link fencing mesh of 50mm square opening,
- 8mm wire mesh of 100mm square opening with 20mm sloping bars at 500mm centres.

Anchor reinforcement systems associated with the surface mesh protection include:

- 20mm bars, 4m long, spaced 900mm vertical and 1200mm horizontal,
- 25mm bars, 11.6m long at base to 7m long at top (+ 12m), spaced 2100 mm vertical and 1500mm horizontal,
- 20mm bars, 4.6m long, spaced 900mm vertical and 1200mm horizontal,
- 25mm bars, 19m long at base to 12mm bars, 7m long at top (+ 23m), spaced 1350m vertical and 1500mm horizontal,
- 38mm bars, 45m long spaced 3000mm by 225mm at base to 20mm bars, 12m long spaced 3000mm by 1500mm at top (+ 40m),
- 25mm bars, 23.5m long at base to 20mm bars, 12m long at top (+ 34m) spaced 1350mm by 1350mm,
- 20mm bars, 11.1m long spaced 1000mm by 1000mm,
- 1-24mm bar per 940mm by 2400mm gabion, 20m long from base to + 18m, then 1-20mm bar per gabion, 20m long from + 18 to + 26m.

Progressive downward failure of mesh protection systems can be avoided by designing and constructing the anchorage system to secure the top layer at any stage, such as by using crank-shaped anchors, inclined anchors, anchors fixed to grouted dowels in the rockfill.

Experience has shown that where light to medium duty mesh is used to retain small size rock particles at the downstream face, it should be protected from damage by eroded rocks or debris with an overlay of 20mm, or larger, sloping bars at 300-500mm spacing (e.g. Googong Dam). Horizontal bars, if used, are preferably placed beneath the sloping bars to avoid obstruction to debris. By completing the protection of a minimum size wedge at the downstream edge at the start of each layer by using, for example, cranked or inclined anchor bars, eroding rocks (and therefore damage to the downstream mesh) are prevented because unprotected rock is never placed above a completed level of protection. Weathered rock in abutments should be protected from erosion by avoiding concentration of flows in these regions.

For the past few years, the Hydro-Electric Commission, Tasmania, has opted to use cylindrical gabions and tie bars almost exclusively as downstream protection for cofferdams and main dams which are subject to overtopping. For cofferdams, the gabions are stacked "bottle fashion" (i.e. their axes in an upstream/downstream direction) on the downstream face, whereas for main dams the gabions are normally stacked with their axes parallel to the dam axis. The former arrangement is often quicker to construct, needs more gabions, and is more costly. The tie bar anchorage for main dams is necessary as for earlier protection techniques.

MESH PROTECTION LEVEL

This is a term used to mean the flood level at which the mesh is terminated on the downstream face of the main dam. In most cases, the mesh is terminated at the 10 year flood level (or less), meaning that at this level a 10 year recurrence interval flood can be passed through the river diversion works without overtopping the dam, if constructed to this level.

The decision as to the level at which to terminate mesh protection (i.e. mesh protection level) is likely to be based on the following major considerations

- the reliability of the hydrological data,
- the estimated time and the season of the year in which it is planned to construct the embankment from the level at which the mesh is terminated to a higher level at which there is a very low probability of being overtopped,
- the probability of being overtopped while constructing the dam above the mesh protection level,
- the incremental cost of mesh protection,
- the value of damage and delay if overtopped when building above the mesh protection level,
- the reliability of construction programs in which it is planned to build above the mesh level in the dry season, taking account of adverse foundation and abutment conditions, interruptions due to industrial disputes, and less-than-planned rockfill production.

In some cases, accelerated construction may permit the mesh to be terminated at a level lower than the design level, such as when towards the end of the dry season construction period it can be reliably forecasted that a "safe" wet season level will be reached by the onset of the wet season. Obviously, very careful examination of the flood storage and estimation of overtopping flood probability is necessary in this case since, at the higher levels of construction, although the probability of overtopping reduces, the volume of water stored and therefore the potential flood wave are both considerably greater - and the dam is vulnerable without mesh protection (Figure 6) c.f. Hell Hole Dam.

OVERTOPPING

With respect to overtopping of mesh-protected rockfill dams or cofferdams, most of the recent applications in Australia have been designed for a 3m discharge depth (equivalent to about $15m^3/sec^{-1}/m$) at the top level of the mesh protection i.e. at the junction of the top surface and the downstream slope (Figure 6a).

At Googong Dam,(32) 10km upstream from the city of Queanbeyan, with Canberra a further 10km downstream, the construction program required the mesh protection (Figure 7a) to be installed before the end of the dry season, the mesh being terminated at the 10,000 year dry season flood level. The flood which overtopped Googong in October 1976 when it was 19m high, and mesh-protected, had a recurrence interval of 1,000 years in the dry season and 20 years in the wet season. [The large disparity in recurrence interval for the same magnitude flood is indicative of the tremendeus contrast between the two season that is so characteristic of Australian conditions, and emphasises the reeson for avoiding construction of the criticel meshed section in the wet season (and the need for protection against overtopping of partly completed dams in the general sense)]. In the case of the Googong flood, a peak discharge depth of 2.5m was followed 23 hours later with a secondary peak of 1.5m. The damage to the mesh-protected dam was negligible - all necessary repair work was completed within a day, after which construction work on the embankment to its final height of 61m was recommended.

In the case of the Ord River Dam,(33) the Ord River flows in the 1970/1 wet season were passed over the partly completed rockfill embankment which was protected with a 1.8m layer of 0.9m individually placed rocks as armouring over the complete top surface and downstream slope together with a light mesh at 150mm centres both ways covered with a 25mm steel mesh of 1200mm by 450mm spacing anchored to the fill and the foundation. Overtopping was continuous for some six months of the wet season, with a maximum depth of flow of 10.7m over the embankment which was at this stage completely drowned. The flow was estimated to be $5660m^3/sec^{-1}$ or $14.2m^3/sec^{-1}/m$, and the peak velocity to be $4.6m/sec^{-1}$. The bank was designed to allow the passage of $28300m^3/sec^{-1}$ floods, with surface velocities estimated from hydraulic model studies to be as high as $8.5m/sec^{-1}$. In this case, with the structure drowned, the critical feature, so far as stability of the rockfill is concerned, is the high velocity regime causing high drag and lift forces across the top surface of the embankment. In the event, the actual peak flow caused no damage to the structure apart from a build-up of silt on the surface and a settlement of about 300mm in some places of the rockfill under the mesh. The rockfill mass retained its integrity, as clearly indicated by photographs(33) taken during and after the long duration flood event.

CASE STUDIES

The ANCOLD Report(31) identifies, tabulates and describes 50 known applications of mesh protection to dams and cofferdams over a period of 40 years to 1982 starting with San Ildefonso Dam in Mexico in 1942. 41 of these applications have been in Australia in the 20 years since 1963. Of the 50 applications, 19 have been upstream cofferdams and 31 main dams. 38 of the 50 structures were completed as at 1982, 18 have been overtopped by floods, and 5 of these have had some degree of failure.

This paper concludes with case studies of three of the 18 dams that have been overtopped, one of which failed.

Borumba Dam(34) in Queensland, was one of the early mesh protected dams built in Australia (Figure 7b). Following hydraulic model tests, a mesh-protected bank 13.4m high and 45.7m wide was construoted to accept overtopping floods in the first wet season. Floods with peaks of $340m^3/sec^{-1}$ and $425m^3/sec^{-1}$ with depths 2.1m and 2.7m resp. occurred, with only minor damage after the first flood caused by debris and stockpiled rockfill breaking several of the mesh bars and junction welds with resultant bulging and small loss of surface rocks. Flow measurements indicated that with upstream water level at top-of-bank level i.e. no overtopping, the discharge of $99m^3/sec^{-1}$ was divided into $48m^3/sec^{-1}$ flowing through the rockfill and $51m^3/sec^{-1}$ flowing through a dry-season-flow diversion conduit. With the upstream water level 2.1m above the crest and a total flow of $340m^3/sec^{-1}$, it was estimated that $71m^3/sec^{-1}$ was going through the conduit, $71m^3/sec^{-1}$ through the rockfill, and $198m^3/sec^{-1}$ over the rockfill.

Cethana Dam(21,20) a 110m high concrete faced rockfill structure in Tasmania, suffered partial failure by flood overtopping when the rockfill at the downstream edge was 15.2m above river bed level. An upstream 10m high cofferdam with 90mm dia. by 2440mm long cylindrical gabion protection was constructed to divert the one year recurrence interval summer flood through a 6.55m dia. diversion tunnel, whilst the main dam was being built and mesh-protected to a height of 34m before the winter flood season. This level corresponded to a 10 year winter flood level. Further, as the dam height increased, it was estimated that with flow taking place only through the rockfill, the upstream level would need to be 23.5m higher than the top of the mesh protection (Figure 6b) before the phreatic line emerged on the downstream face above the mesh. This upstream level corresponded to a flood of $1557m^3/sec^{-1}$ with a recurrence interval greater than 1000 years. The chances of throughflow occurring above the mesh protection were therefore very slight indeed. In fact, when the dam was well above the top of the mesh protection level in height, a flood ponded to a depth of 55m and flow through the rockfill took place without damage or deformation.

The mesh for Cethana Dam (Figures 7c, 8) consisted of 3m wide rolls of chain-link fencing mesh which was tensioned over each 1.4m layer of rock to anchor bars set into the rock, then the chain-link mesh was rolled back to the face awaiting the next layer of rock - thus forming 3-sided gabion-type cells. Each 3m strip of mesh was joined to adjacent strips across the dam width by wire ties.

At the time when the main bank was 15.2m high, a flood of $453m^3/sec^{-1}$ overtopped the rockfill. The diversion tunnel accepted $283m^3/ sec^{-1}$, and the remaining $170m^3/sec^{-1}$ passed through and over the rockfill. The progress of the flow through and over the rockfill with the resultant failure after 5 hours of overtopping are well depicted in ANCOLD Bulletin 28(21) (Figure 8).

When the flood occurred, only two-thirds of the last-placed layer of rockfill had been mesh-protected. During the 5 hours of overtopping there was a continuous loss of rock from the face of the top layer in the unprotected area and this unravelling progressed upstream, with rocks bouncing down the face. Suddenly, within ten minutes, the protection of the left side of the dam failed and a Vee-shaped notch 9m deep formed in the face, accompanied by a loss of about $15290m^3$ of rockfill. The scoured channel in the top surface of the rockfill extended 183m upstream from the downstream face and had a general slope of about 1 in 20.

The mesh over the protected two-thirds of the face showed no signs of distress or excessive movement. There is some conjecture on the reasons for the failure, though it would seem clear that the unprotected rocks bounoing down the slope damaged the mesh and created leakage holes through it. Likewise, since the major part of the bank remained stable during 5 hours of overtopping, it is evidence that the design dimensions of the anchor bars and mesh were adequate for the hydraulic forces imposed. The observation that anchor bars were still in position late in the failure confirms that it was the mesh that failed. It should be noted, however, that 30 per cent of the rockfill in the bank was material smaller than 25mm size which could be eroded through the mesh. It should be further noted that the technique used in many other dams of using cranked anchor bars to provide a narrow one layer thick mesh protected bank at the downstream face was not used at Cethana Dam.

An alternative system of mesh protection was used after the failure (Figure 7d). A continuous trapezoidal gabion of "gridmesh", a heavy duty type of expanded metal, was anchored back to the previous layer surface as depicted in ANCOLD Bulletin 28(21). The anchor lengths, size and horizontal spacing were the same as for the previous chain-link mesh system. The continuous gabion was completed and filled before any rockfill was placed in that layer, thus protecting all fill at all times. This technique was used up to the original mesh-protected height without further incidents or overtopping. The grid-mesh, incidentally, was found to be difficult material to work with, had sharp edges to cause injury to workers, and was about three times more expensive than chain-link mesh.

Moochalabra Dam(22) is a 15m high water supply dam for Wyndham in the remote tropical north west region of Western Australia. It differs from most other mesh-protected rockfill dams in that the embankment itself serves as a overflow spillway in a long-term fashion, rather than only during the 1-3 year construction period of a conventional dam. All steel anchors and mesh (Figure 7e) were galvanised to provide long-term protection against corrosion.

The dam was completed in 1972 as the first stage of a larger structure to be built when the demand for water increased. This demand has not yet eventuated so that, for the past 15 years, floods have overtopped the original dam each wet season for periods of 10 to 20 weeks with flow depths at the crest up to 1.0m in 1974 and 1.25m in 1984.

Stability against flow through and over the dam was provided by mesh and anchors covering the downstream face, the crest, and extending for a short distanca down the upstream face. The design flood for the dam was $900m^3/sec^{-1}$ causing a design depth at the 120m long crest of 2.7m.

Regular inspections of the dam(22) have revealed that the performance of the galvanising has been considerably better than expected i.e. life greater than 10 years. The main areas of corrosion attack have been on the field welds which were kept to a minimum in the original construction. Settlement of the dam, monitored since 1973, has shown that surface points have moved a maximum 220mm downstream and settled up to 90mm. Remedial work to the steel bars, loss of rock and bulging of the mesh, have been minimal for this long-term severe test of overtopping.

CONCLUSIONS

Mesh protection of rockfill banks has gained widespread use in the past 30 years. Innovative, rather than stereotype, design and construction techniques have been adopted in practice with satisfactory result, with few exceptions. Analytical solutions for both flow and stability characteristics of rockfill have been developed, and these have subsequently been utilized in other contexts, including analysis of natural slopes as in the 1000m deep open-cut copper mine at Bougainville, Papua New Guinea.

ACKNOWLEDGEMENT

Thanks are due to Mr. R. J. Wark, Secretary of the Australian National Committee of the International Commission on Large Dams, and to Mr. M. D. Fitzpatrick, Chief Civil Engineer, The Hydro-Electric Commission, Tasmania, for providing information and illustrations useful in the preparation of this paper.

REFERENCES

(1) Wilkins, J. K., "The Flow of Water Through Rockfill and its Application to the design of Dams", Proc., 2nd Australia-New Zealand Conference on Soil Mechanics and Foundations Engrg., 1956, p. 141-149.

(2) Isbash, S.V., "Construction of Dams by Dumping Stone in Running Water" Moscow - Leningrad 1932, and 2nd Congress on Large Dams, Washington D.C. 5, 1936, p. 123.

(3) Sandie, R.B., "A Laboratory Investigation of Self-spillway Rockfill Dams", M.Eng.Sc. Thesis, University of Melbourne, 1961.

(4) Sharp, B.B., "Flow in a Rockfill Dam with Inbuilt Spillway", Civil Engineering, 56, No. 665 (1961).

(5) Parkin, A.K., Report DR2 - "Rockfill Dams with Inbuilt Spillways - Part 1, Hydraulic Characteristics" - Department of Civil Eng., Uni. of Melbourne, August 1962. (issued as Water Research Fcundation Bulletin No. 6, March 1963).

(6) James, J.P., "The Stability of a Simple Rockfill Slope Subject to Seepage", M.Eng. Sc. Thesis, Uni. of Melbourne, 1962.

(7) Lawson, J.D., Trollope, D.H. and Parkin, A.K., "Some Hydraulic Aspects of Unconventional Rockfill Dams", Proc. 1st Aust. Conf. on Hydraulics and Fluid Mechanics, Uni. of W.A. Perth, December 1962.

(8) Sharp, B.B. and James, J.P., "Spatially Varied Flow at the toe of a Rockfill Slope", Proc. 1st Aust. Conf. on Hydraulics and Fluid Mechanics, Uni. of W.A. Perth, December 1962.

(9) Parkin, A.K., Report DR3 - "Rockfill Dams with Inbuilt Spillways, Part 2, Stability Characteristics" Department of Civil Engineering, Uni. of Melbourne, Feb. 1963. (issued as Water Research Foundation Bulletin No. 7, Dec. 1963).

(10) Parkin, A.K., "The Hydraulic and Stability Characteristics of Self Spillway Rockfill Dams". M.Eng.Sc. Thesis, Uni. of Melbourne, March 1963.

(11) Parkin, A.K., Report DR5 - "Freefall Dispersion in Ideal Granular Media" Dept. of Civil Engineering, Uni. of Melbourne, July 1964.

(12) Parkin, A.K., Trollope, D.H., and Lawson, J.D., "Rockfill Structures Subject to Water Flow", ASCE, Proc. Paper 4973, Vol. 92, No. SM6, Nov. 1966, pp 135-151.

(13) Curtis, R.P., and Lawson, J.D., "Flow over and through Rockfill Banks", ASCE, Proc. Paper 5412, Vol. 93, No. HY5, Sept. 1967, pp 1-21.

(14) Olivier, H., "Through and Overflow Rockfill Dams - New Design Techniques", Proc. Instn. Civ. Engrs., Mar. 1967, 36 (Paper No. 7012), pp 433-471, Discussion: Proc. Inst. Civ. Engrs., Aug. 1967, 37 pp 855-888.

(15) Linford, A. and Saunders, D.H., "A Hydraulic Investigation of Through and Overflow Rockfill Dams", Brit. Hydromechanics Research Association, RR 888, March, 1967.

(16) Fenton, J.D., "Hydraulic and Stability Analyses of Rockfill Dams", Departmental Report DR15, Dpt. of Civil Engineering, Univ. of Melbourne, Parkville. July 1968.

(17) Wright, D.E., "Nonlinear Flow through Granular Materials", ASCE, Proc. Paper 6018, Vol. 94, No. HY4, July 1968, pp 851-872.

(18) Volker, R.E., "Nonlinear Flow in Porous Media by Finite Elements", ASCE, Proc. Paper 6927, Vol. 95, No. HY6, Nov. 1969, pp 2093-2114.

(19) Parkin, A.K., "Field Solutions for Turbulent Seepage Flow", ASCE, Proc. Paper 7794, Vol. 97, No. SM1, Jan. 1971, pp 209-218.

(20) Thomas, Henry H., "The Engineering of Large Dams", Part II, Chapter 15, Wiley, 1976.

(21) H.E.C. (Tasmania) "Chethana Dam. Flood Breach of Partly Completed Rockfill Dam", Aust. Nat. Committee on Large Dams, Bulletin 28, July 1969, pp 23-36.

(22) Wark, R.J., and Szymakowski, J., "Moochalabra Dam - 10 years Experience with an Overtopped Rockfill", Australian Nat. Committee on Large Dams, Bulletin 63, December 1982, pp 40-43.

(23) Linford, A., "A Review of Literature on the Construction of Rockfill Dams by Dumping Stones in Running Water", Brit. Hydromechanics Research Association, TN873, March 1967.

(24) Western Construction, "The Failure of Hell Hole Dam". Vol. 40, No. 4, April 1965.

(25) Scheidegger, A.E., "The Physics of Flow Through Porous Media", 3rd Edition Univ. of Toronto Press, 1974.

(26) Bear, Jacob, "Dynamics of Fluids in Porous Media", Elsevier, 1972.

(27) Lee, I.K. et al, "Soil Mechanics: Selected Topics", Chapter 3, Butterworths, 1968.

(28) Ward, J.C., "Turbulent Flow in Porous Media", ASCE, Proc. Paper 4019, Vol. 90, No. HY5, Sept. 1964, pp 1-12.

(29) Knauss, J., "Computation of Maximum Discharge at Overflow Rockfill Dams (a comparison of different model test results)", Proc. XIII ICOLD Congress, New Delhi, 1979, pp 143-160.

(30) Bishop, A.W., "The Use of the Slip Circle in the Stability Analysis of Slopes", Geotechnique, Vol. 5, 1955, pp 7-17.

(31) ANCOLD, "Report on Mesh Protection of Rockfill Dams and Cofferdams", March 1982. Report by Australian National Committee on Large Dams to ICOLD for Cofferdams section of Technology of Dam Construction report.

(32) ANCOLD Bulletin 47, April 1977, Fitzgerald B.J. "Flood Forecasting during Construction of the Googong Dam", pp 13 - 15.

(33) ANCOLD Bulletin 34, Sept. 1971, PWD of WA "Overtopping of Stage 1 of the Ord River Dam", pp 3-6.

(34) ANCOLD Bulletin 17, Oct. 1965, Robson W.M., "The Passage of Floods during the Construction of Borumba Dam", pp 14-16.

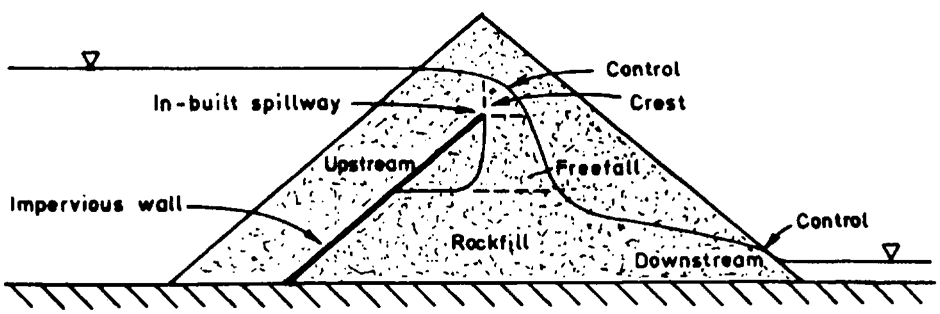


Figure 1. Regions of Flow in an In-Built Spillway Dam

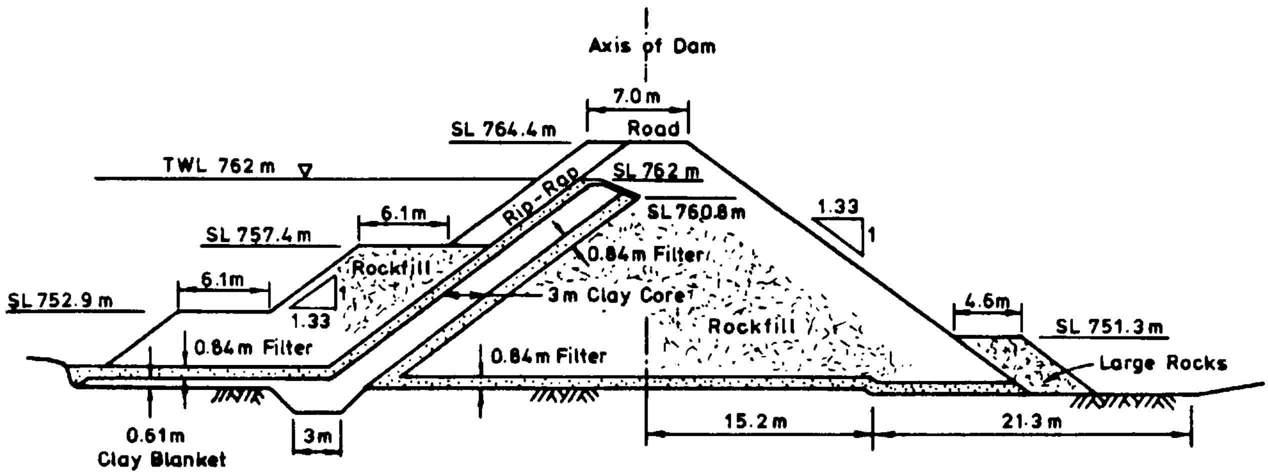


Figure 2. Laughing Jack Marsh Dam (after Wilkins[1])

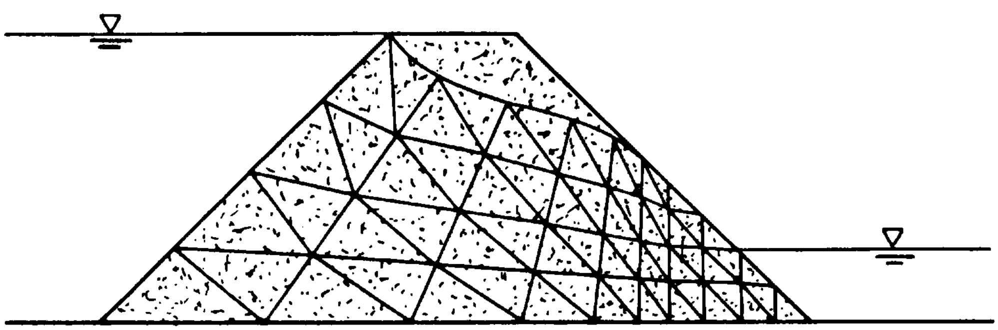
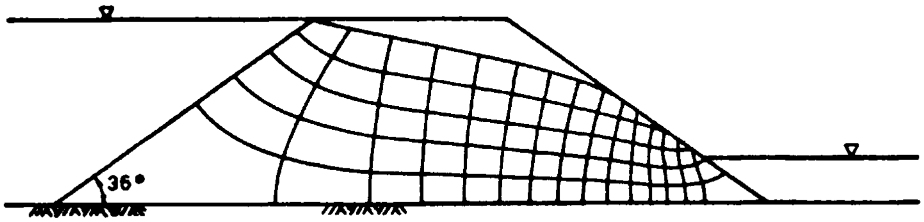
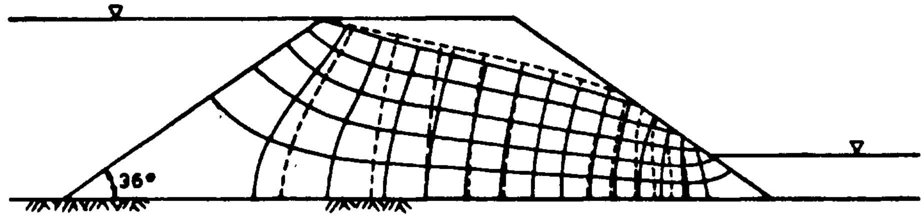


Figure 3. Finite Elements for Dam Analysis



(a) Turbulent Flow Net, n=1.85



(b) Laminar Flow Net, n=1.00, with Turbulent Equipotentials superimposed in broken lines

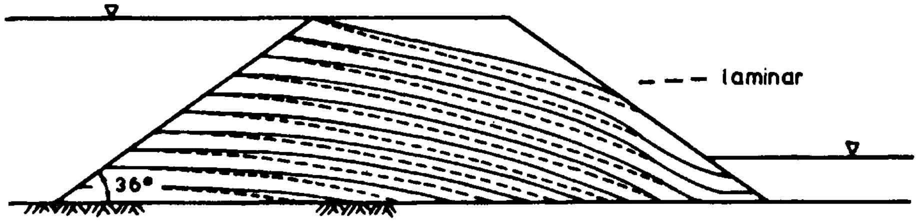


Figure 4. (c) Laminar and Turbulent Isobars (after Parkin[19])

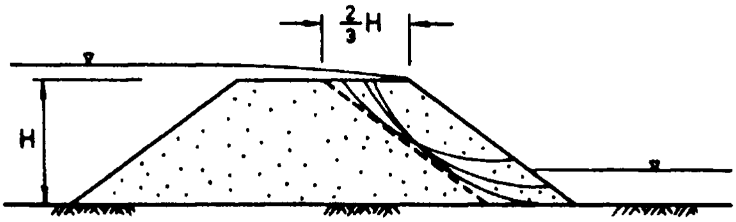
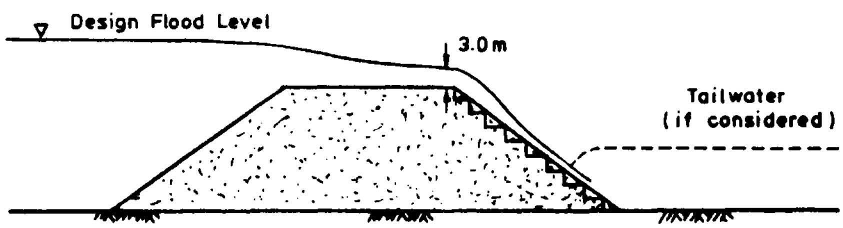
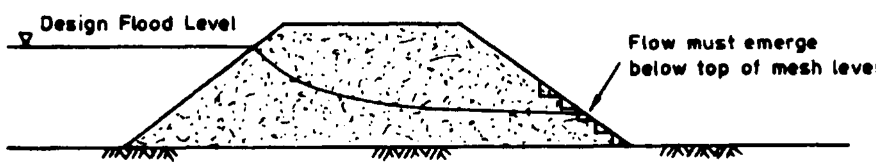


Figure 5. Envelope to F=1 Slip Circles for Overtopped Rockfill Dam



(a) Overflow Case



(b) Seepage Case

Figure 6. Design Flow Definitions

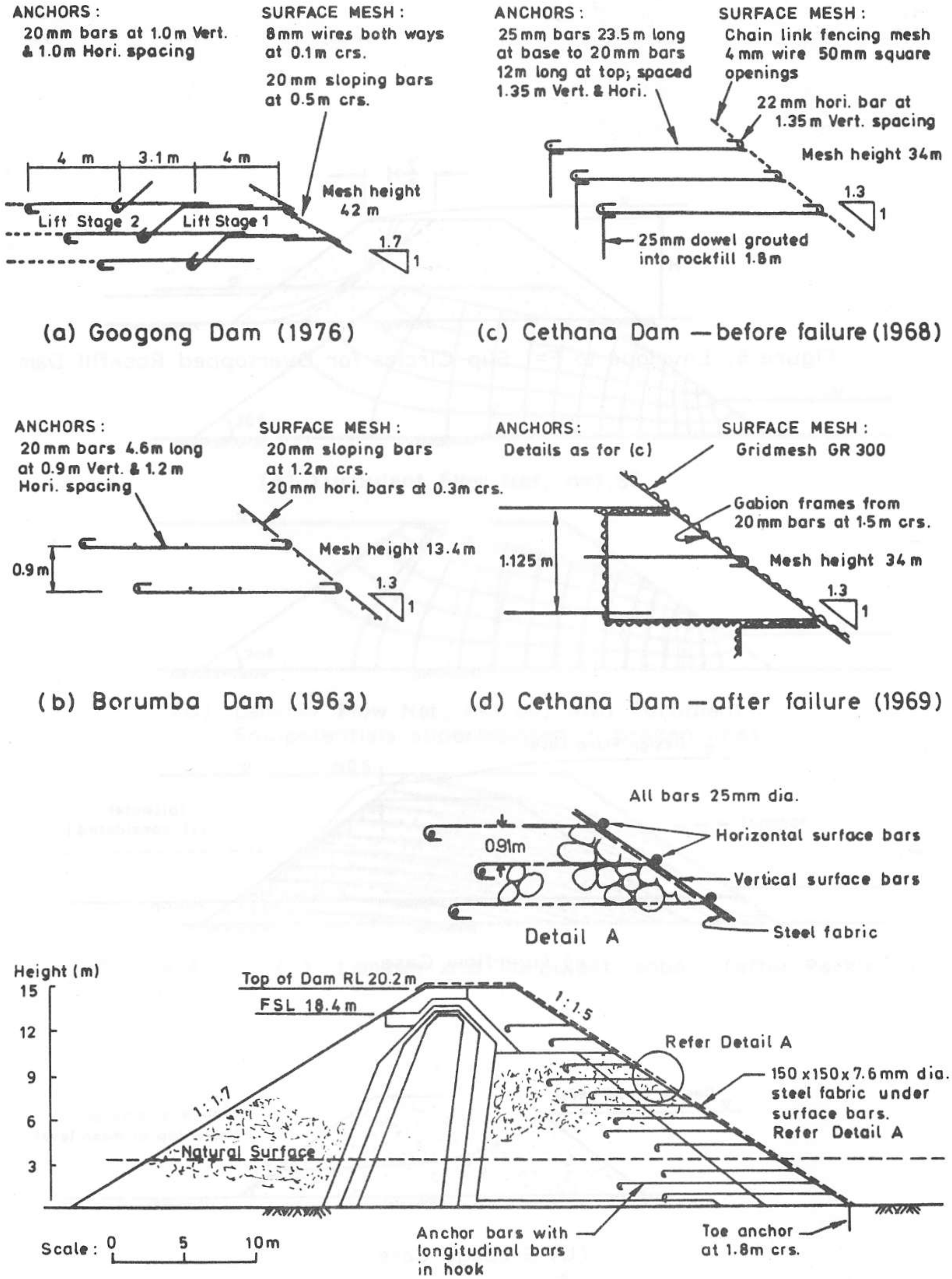
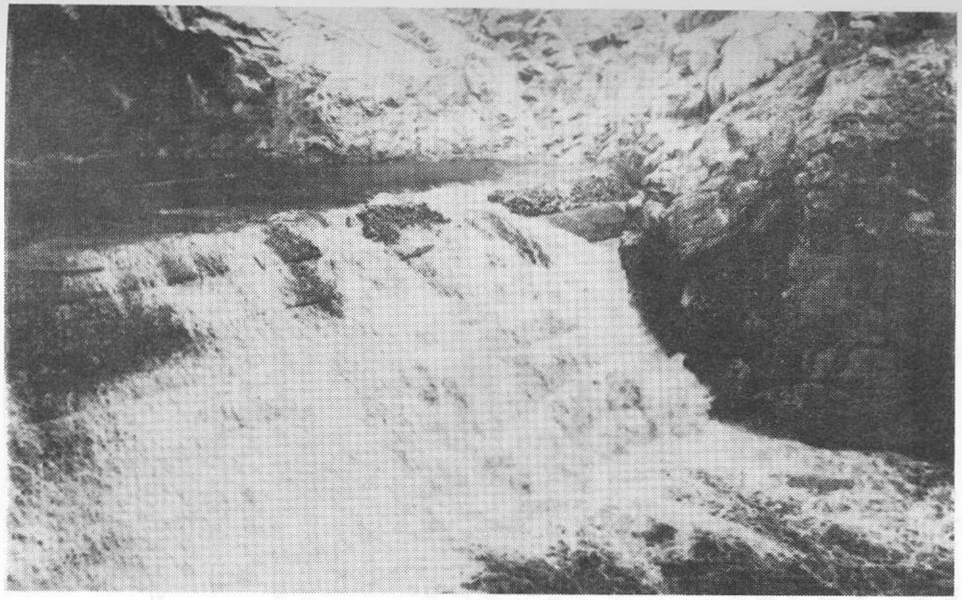
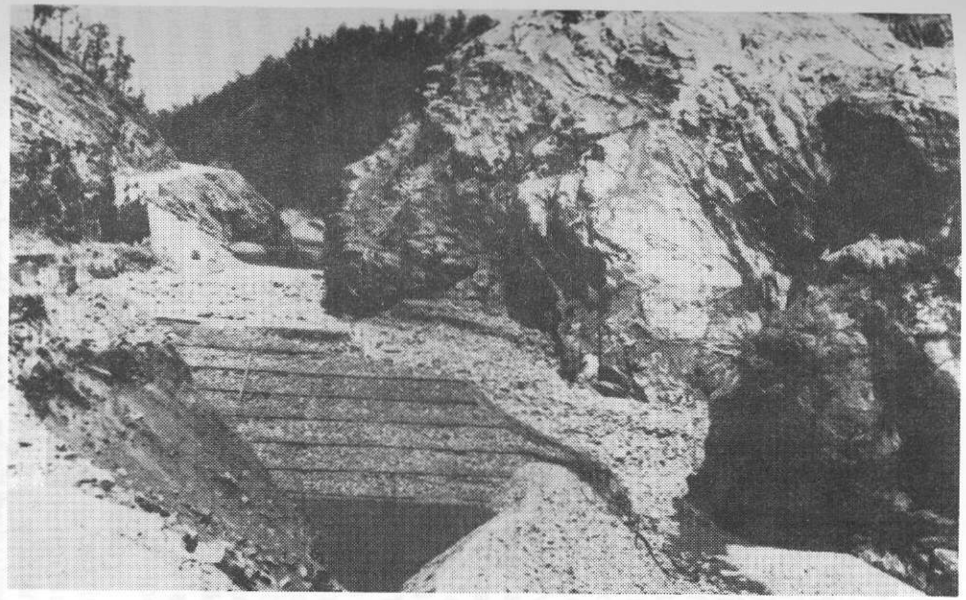


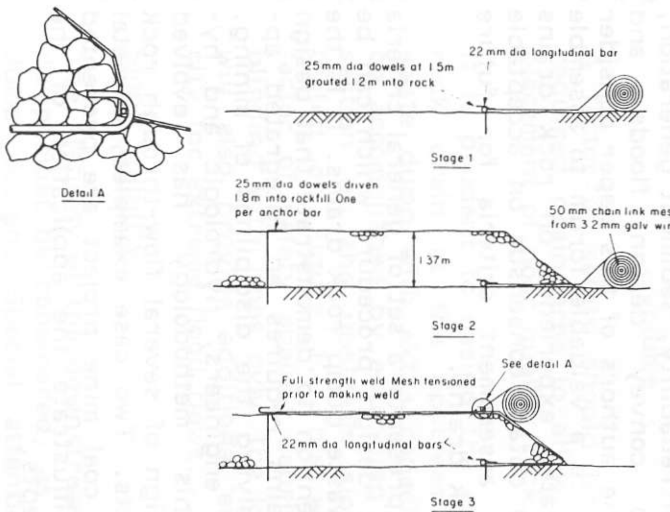
Figure 7. Anchor and Surface Mesh Details (after Ancold[31])



soon after overtopping



Vee-shaped breach after flood



Construction sequence for installing mesh

initial mesh protection

Figure 8. Cethana Dam

AN INTEGRATED APPROACH TO DESIGN OF ROCK DRAINS

by
Peter C. Lighthall
Manager, Mining Services Division
C. David Sellars
Manager, Water Resources Division
of
Klohn Leonoff Ltd., Richmond, British Columbia

INTRODUCTION

Rock drains for mine waste dumps have become accepted solutions for conveying streamflow through mine waste dumps, particularly in mountainous regions of British Columbia. However, reviewing agencies often express considerable doubt about the performance of rock drains, particularly in regard to long term reliability, sediment generation, capacity to convey design floods, and stability. The authors of this paper consider this conference a valuable forum to assemble the accumulated experience on rock drains and also to obtain consensus on acceptable design and assessment criteria for future proposed rock drains.

This paper presents a set of general criteria and a basic design procedure which can be applied to waste dump rock drains. It is the authors' intention to demonstrate that design of rock drains requires an integrated approach, applying the disciplines of mining, geotechnical engineers, hydrologic and hydraulic. This methodology has evolved through design of several flow-through rock drain projects. Two case examples, a metal mine and a coal mine project are presented which will illustrate the application of the design concepts.

The design approval described herein assumes that the site selection and foundation design for the waste dump have been completed. This paper focuses on the design of the rock drain.

DESIGN CONSIDERATIONS

Rock Quality

An assessment must be made of the quality and quantity of waste rock available for drain construction. Ideally, rock for drains should consist of large fragments of hard, durable material.

The basic element of rock quality assessment is core logging to inventory the rock types and structural characteristics of the waste. Structural logging is important, since even hard rock types may not be suitable if they are highly fractured and will form small fragments when blasted.

When potentially suitable material is identified by core logging, tests should be carried out to evaluate the compressive strength of the rock. This may be done on samples of core, either by unconfined compressive testing or by empirical correlation of point load tests. Samples of potential rock types for drain construction should be submitted to a qualified laboratory for evaluation of acid production potential. Any potentially acid-producing materials should be excluded from use in the drain.

Coal measures generally consist of a varying sequence of sedimentary rocks, ranging from hard sandstones and conglomerates through to soft, easily weathered siltstones, mudstones and shales. In metal mines, mineral deposits often occur in highly altered rock types, so waste rock quality may vary considerably.

Waste Rock Selection and Placement

Waste rock selection for drain construction must be integrated into the overall mine plan to ensure that suitable rock will be available when required. The constraints of selecting rock could have considerable impact on the mining plan. Ideally, where a bedding layer, seam or deposit of suitable rock is available in sufficient quantities, drain rock can be obtained by normal mining practice, with no special procedures. It may be necessary to widen drill hole spacings to reduce blast fragmentation. In cases where rock is less available, special procedures such as grizzlying waste material or individual selection of rock fragments may be necessary.

There are two basic methods of constructing rock drains for valley fills, depending on the availability of sufficient quantities of suitable rock to convey the design flow. Ideally, good quality waste can be end-dumped from the top of the valley slopes and the drain be formed by segregation of coarse particles at the toe. A minimum height should be specified for end-dumping to ensure that sufficient segregation occurs. Nichols and Rutledge (1982) noted that better segregation is achieved by dumping directly over the slope rather than pushing over the edge with dozers.

The second method of constructing rock drains is to place the rock directly in the drain. This method is required when limited quantities of suitable rock are available and/or where a high quality drain is desired to limit sediment production. Obviously, the direct placement method is considerably more expensive than construction by end-dumping, as select rock must be hauled to the valley bottom.

Hydrologic Analysis

Design of flow-through waste dump drains requires an assessment of the peak flows which the drain must carry. Normally, basic hydrologic parameters are developed as part of the water management plan for a mining development, so that they need only be applied to the specific catchment upstream of the waste dump. The recommended design flow for flow-through waste dumps is the 200-year return period peak flow. For abandonment, the effect of the Probable Maximum Flood should be considered.

It has been found that a rock drain has considerable attenuation effect on flood peaks. Detailed hydrologic analysis and flood routing through the rock drain will show that the downstream flows are significantly reduced. This can have cost savings for downstream water management structures.

Hydraulic Design

Design principles for flow through rockfill were developed for civil engineering applications for flow through rockfill dams. The principles were derived from the general equation of flow in a porous medium:

$$v = ki^n$$

where v = the velocity

i = the hydraulic gradient

and n = 1 for Darcy laminar flow

For non-Darcy turbulent flow in rockfill, Wilkins (1956) derived the following empirical formula:

$$v = 5.24\, m^{0.5}\, i^{0.54}$$

where v = the void velocity in m/s

i = the hydraulic gradient

and m = the hydraulic mean radius

$$= \frac{\text{volume of voids}}{\text{surface area of particles}}$$

$$= \frac{\text{void ratio}}{\text{surface area per unit volume}}$$

A factor of safety should be added to the required drain area to allow for accidental placement of poor quality rock.

It is prudent to design the entrance of the rock drain at the upstream side of the waste dump with extra capacity. This may be accomplished by extending the full drain section up the upstream slope, as illustrated on Figure 1. This will provide a factor of safety against debris blockage or blinding with fines over the long-term, and will also provide additional entrance capacity for extreme flood flows.

Slope Stability

The stability of the downstream slope of the dump must be considered, as for any waste dump design. Special attention, however, mut be paid to the stability of the rock drain up to the highest point of exit of the design flows. The force of the emerging flow will reduce the stability of the slope. Sliding at the toe could lead to progressive sliding on the downstream slope of the dump. Such sliding could cause large volumes of debris and sediment to be carried downstream.

Stability analyses of the waste dump toe should be carried out to determine a suitable design. The common methods of improving toe stability are: (1) flattening the slope and (2) providing large, angular rock fragments.

FIGURE 1

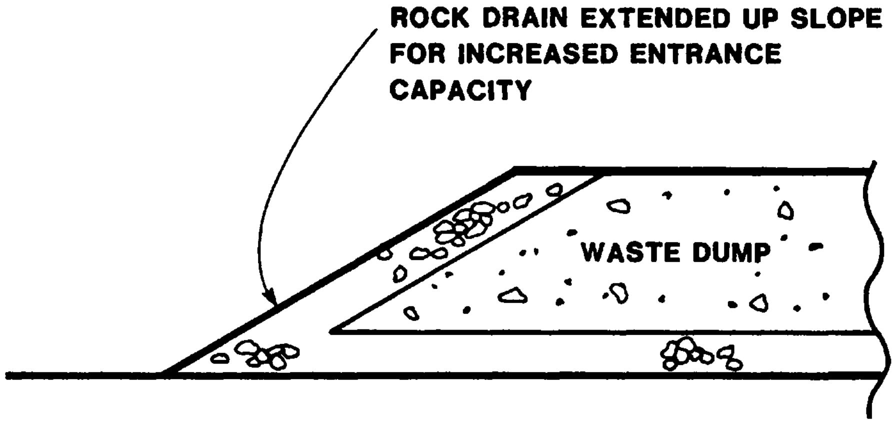


FIGURE 1
UPSTREAM SLOPE
OF WASTE DUMP

These materials will initially be flushed through the drain, causing increased levels of suspended sediment downstream. This will normally require that a sedimentation pond be constructed downstream to allow settling of suspended solids. Observations of operating flow-through drains by Nichols and Rutledge (1982) have shown that over the long-term, the sediment concentration downstream will be very low.

In some cases, sediment production may need to be reduced, for example where construction of a sedimentation pond downstream would not be practical. This may require direct placement of clean, select rock. Other water quality parameters that should be considered in design are acid generation potential of the waste rock and residual nitrates from blasting.

Reclamation and Abandonment

Valley fills, with flow-through drains, lend themselves readily to reclamation. By placing fill in a valley, the surface area of the dump slopes is considerably reduced. The British Columbia Mine Reclamation Guidelines recommend re-sloping dump faces to 27° or flatter.

For abandonment, the rock drain is considered a permanent structure. Design for the Probable Maximum Flood (PMF) may be appropriate, depending on the anticipated downstream hazards which would result from a failure. The effects of sediment accumulation at the inlet should also be considered. If the drain does not have sufficient capacity to convey the PMF at abandonment, overtopping of the valley fill should be allowed for. This may require flattening downstream slopes.

CASE HISTORY - BULLMOOSE COAL MINE

To illustrate how the above design criteria can be applied, the development of a large rock drain at the Bullmoose Coal Project is described. The Bullmoose Coal Mine is located in the northeast coal development of British Columbia. Waste dump plans were submitted to government as part of the development plan and these were granted approval with delay. The mine commenced production on schedule and under budget in November, 1983 and has operated profitably since that time. Figure 2 shows a plan of the development.

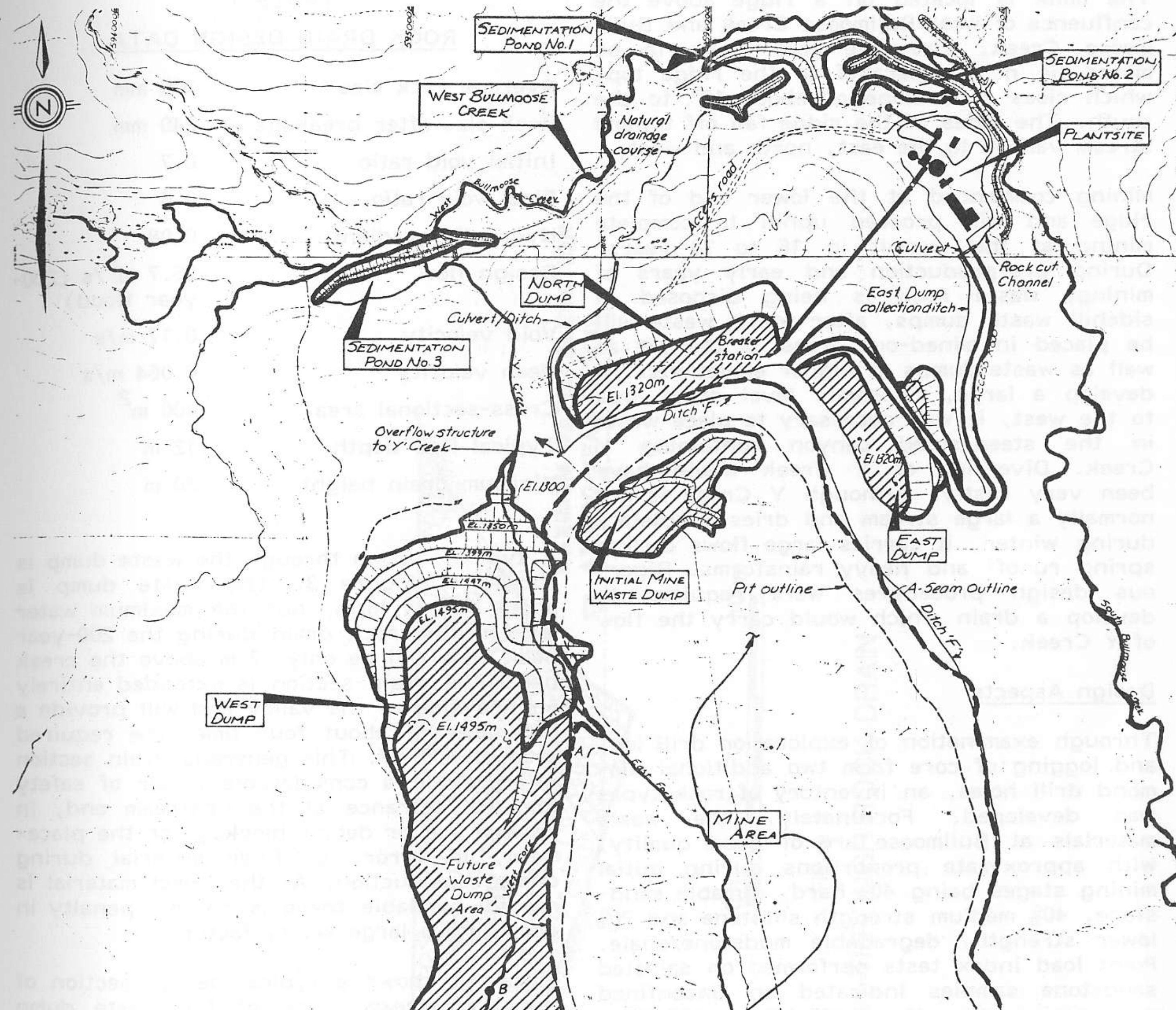


FIGURE 2

PLAN OF BULLMOOSE COAL MINE

The mine is located on a ridge above the confluence of West Bullmoose Creek and Bullmoose Creek. The five coal seams to be mined lie nearly parallel to the ridge top, which rises at a slope of about 10° to the south. The sides of the ridge fall off toward stream valleys to the east, north and west.

Mining commenced at the lower end of the ridge and will proceed uphill to complete mining at the summit in 16 to 18 years. During pre-production and early years of mining, waste rock is being disposed in sidehill waste dumps, after which waste will be placed in mined-out areas of the pit as well as waste dumps at higher elevations. To develop a large, relatively level dump area to the west, it was necessary to place waste in the steep-sided canyon containing Y Creek. Diversion of Y Creek would have been very costly. Although Y Creek is not normally a large stream and dries completely during winter, it carries large flows during spring runoff and heavy rainstorms. Rigorous design procedures were required to develop a drain which would carry the flow of Y Creek.

Design Aspects

Through examination of exploration drill logs and logging of core from two additional diamond drill holes, an inventory of rock types was developed. Fortunately, the waste materials at Bullmoose are of good quality, with approximate proportions during initial mining stages being 40% hard, durable sandstone, 40% medium strength siltstone and 20% lower strength, degradable mudstone/shale. Point load index tests performed on selected sandstone samples indicated an unconfined compressive strength of 125 MPa to 165 MPa. Both slake durability tests and magnesium sulphate soundness tests were carried out on sandstone samples, both of which indicated the material was resistant to weathering and chemical breakdown.

As ample quantities of waste rock were determined to be available, it was possible to design the drain conservatively.

The design data for the Y Creek rock drain are listed in Table 1.

Water Quality

Rock drains constructed by end dumping will normally contain some fine materials.

TABLE 1

ROCK DRAIN DESIGN DATA

Average rock size	300 mm
Rock size after breakage	240 mm
Initial void ratio	0.7
Final void ratio	0.6
Hydraulic gradient	0.08
Design flow	25.7 m^3/s (200-year flood)
Void velocity	0.17 m/s
Mean velocity	0.064 m/s
Cross-sectional area	400 m^2
Typical flow depth	12 m
Minimum drain height	20 m

A typical section through the waste dump is shown in Figure 3. The waste dump is about 100 m high, but the maximum water level in the rock drain during the 200-year peak flow will be only 12 m above the creek bed. The drain section is extended entirely on one side of the valley and will provide a flow area of about four times the required area of 400 m^2. This generous drain section will provide a considereble factor of safety for the entrance at the upstream end, in case of ice or debris blockage or the placement, in error, of finer material during dump construction. As the select material is readily available there is no cost penalty in providing a large safety factor.

Figure 4 shows a typical design section of the downstream slope of the waste dump along the alignment of Y Creek. The creek section was designed with a toe berm to accommodate the high water levels at the outlet during peak flow. The slope was designed to be flattened to 3.5 horizontal to 1 vertical, with a riprap toe. The overall waste dump slopes are laid out to allow resloping for reclamation. Berms are left at 50 m intervals to allow the slope to be flattened to 27° overall.

In addition to the advantages of convenience, flexibility in dump construction and lower costs, the flow-through rock drain on Y Creek was found to have a significant flood routing effect which reduced the construction costs of the spillway on Settling

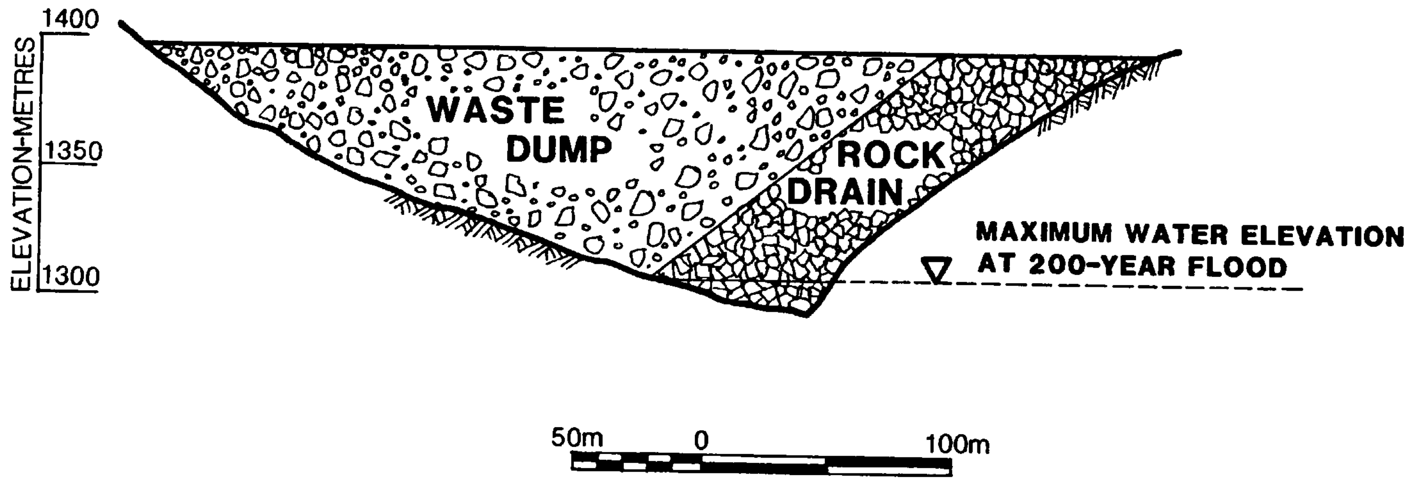


Figure 3

FLOW-THROUGH ROCK DRAIN

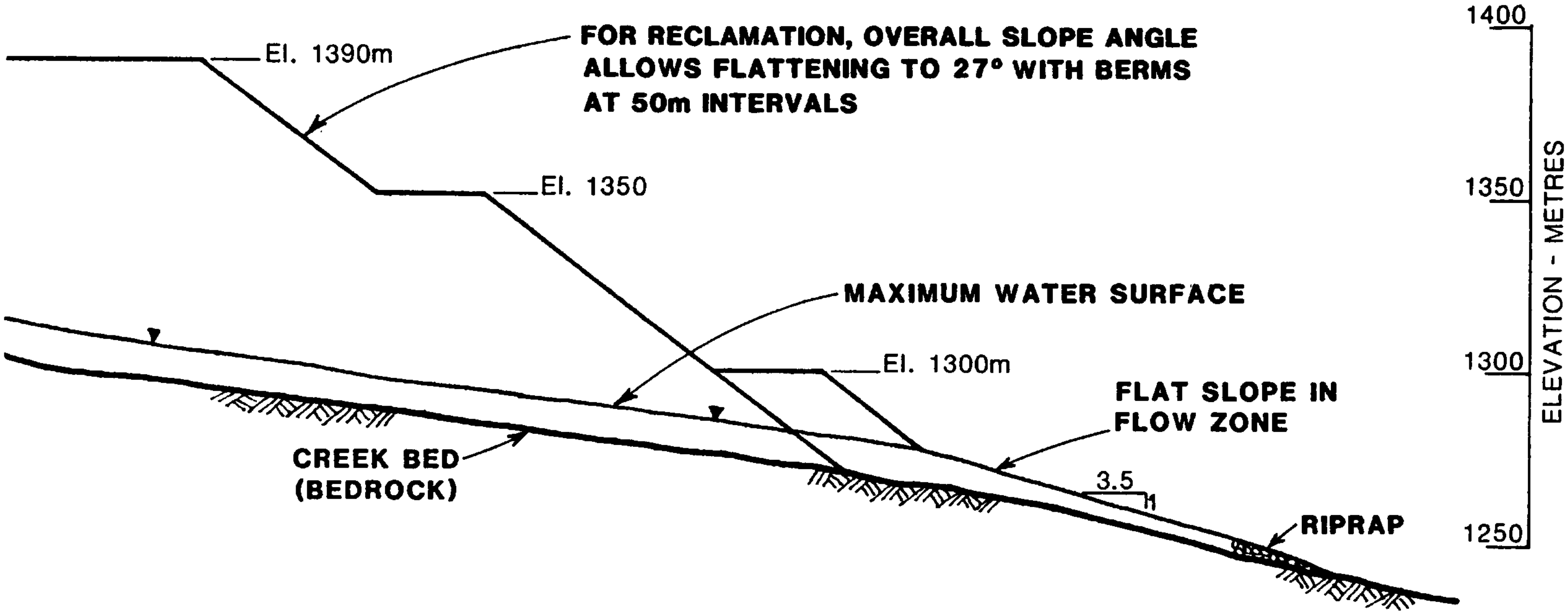


FIGURE 4

DOWNSTREAM SLOPE OF WASTE DUMP

Pond No. 3 downstream of the dump. (Sellars, Lighthall and Robertson, 1984.) Figure 5 is a diagram of the catchment areas contributing to Settling Pond No. 3. The 200-year inflow to the rock drain was derived from Catchment 301 was routed through the rock drain, and then added to the hydrograph from Catchment 302 (2.64 km^2). The combined hydrograph was the inflow to the settling pond.

The inflow design hydrographs to Settling Pond No. 3 are shown in Figure 6. The routing effect of the rock drain not only reduces the peak flow from Catchment 301 but also delays the timing of the peak. When the outflow hydrograph from the rock drain is added to the hydrograph from Catchment 302, the total peak inflow to the pond is reduced by a factor of about two. This hydrologic analysis allowed a reduction in the size of the spillway design for Settling Pond No. 3, which is located downstream of the waste dump, just above the confluence of Y Creek with West Bullmoose Creek. As the dumps are extended to cover more of the Y Creek catchment area, the flood peaks will be further attenuated as runoff infiltrates into the dumps.

Construction and Performance

Construction of the waste dump drain being in early 1984, with placement of a high lift at elevation 1390 m near the downstream end of the Y Creek drain. Subsequently, in late 1984 and early 1985, berms were added to elevations 1350m and 1300 m. Partial drain sections were also placed as creek crossings at several locations upstream.

Prior to the spring freshet in 1984, very little flow was observed in Y Creek and Bullmoose personnel were concerned whether the drain was functioning properly. However, as snowmelt began, the rock drain carried all of the flow with no blockages observed even at the highest flows. As expected, a considerable amount of sediment was flushed from the drain initially. By summer, however, flows appeared free of sediment.

One design modification has been agreed upon since construction of the downstream slope. The flattening berm at the toe of the dump has been eliminated. The toe area is so well confined by the canyon walls and by a bend in the stream alignment that no instability would be expected during high flows, other than minor ravelling at the toe.

CASE HISTORY - KITSAULT MINE

Kitsault Mine is located on a steep mountain side of the Coast Range, overlooking Alice Arm to the northwest. The planned ultimate open pit area straddles Patsy Creek just upstream of its confluence with Lime Creek, which flows on a steep gradient approximately 6 km northeast to discharge into Alice Arm. Both Patsy and Lime Creeks flow in V-shaped stream valleys which are deeply incised into bedrock.

AMAX of Canada Ltd., operators of the Kitsault molybdenum mine, requires disposel sites for approximately 230 million tonnes of waste rock during the planned 25-year life of the mine. The economic feasibility of the mining operation depends to a great extent on being ablo to utilize the Patsy and Lime Creek valleys for waste disposal. The alternative waste disposal location on sidehill dumps to the north of the mine area, would involve a long uphill haul. The use of this alternative (Clary Dump) would render the mine uneconomic beyond an estimated 15-year operating life.

In April 1981, AMAX applied to the Chief Inspector of Mines for a 3-year mining permit for Kitsault Mine. This permit application laid out AMAX's proposed strategy for determining the feasibility of dumping in the stream valleys. Waste rock for the 3-year period was to be placed in the Clary Dump area. To accomplish that plan, the stripping ratio was reduced in years 2 and 3 from the original figure of 2.5:1 to 1.8:1. This allowed AMAX to sustain the planned mill throughput of 10,900 tonnes per day without increasing their haul truck fleet. During the 3-year period, AMAX proposed to construct a test dump in Patsy Creek to demonstrate the feasibility of valley fill waste disposal.

AMAX was granted approvel for their mining plan, with the exception that the proposed test dump was not approved because of environmental concerns, particularly related to downstream suspended sediment levels. The mine began operation in April 1981, placing waste in the Clary Dump haul road and in Clary Dump.

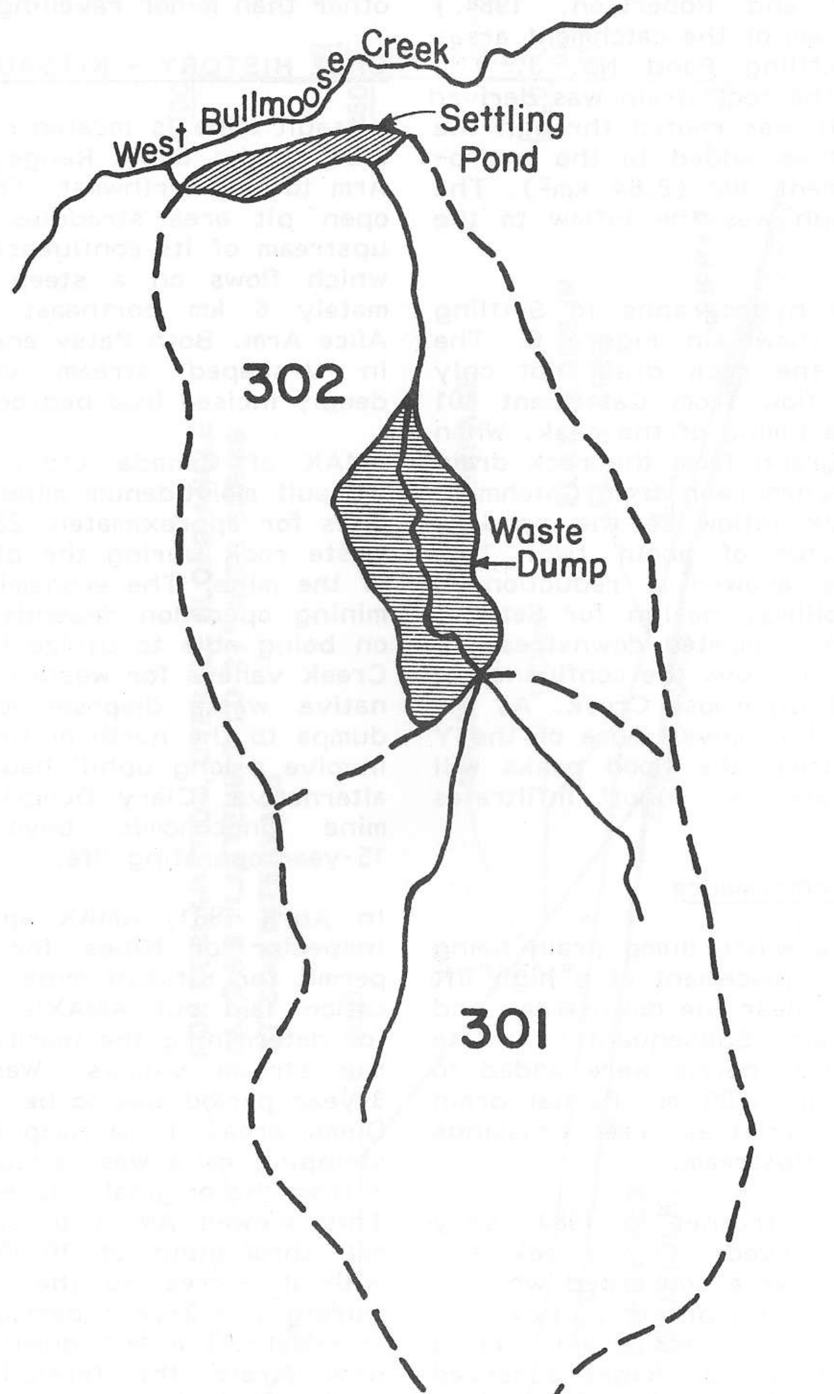


Figure 2

Y-CREEK CATCHMENT AREAS

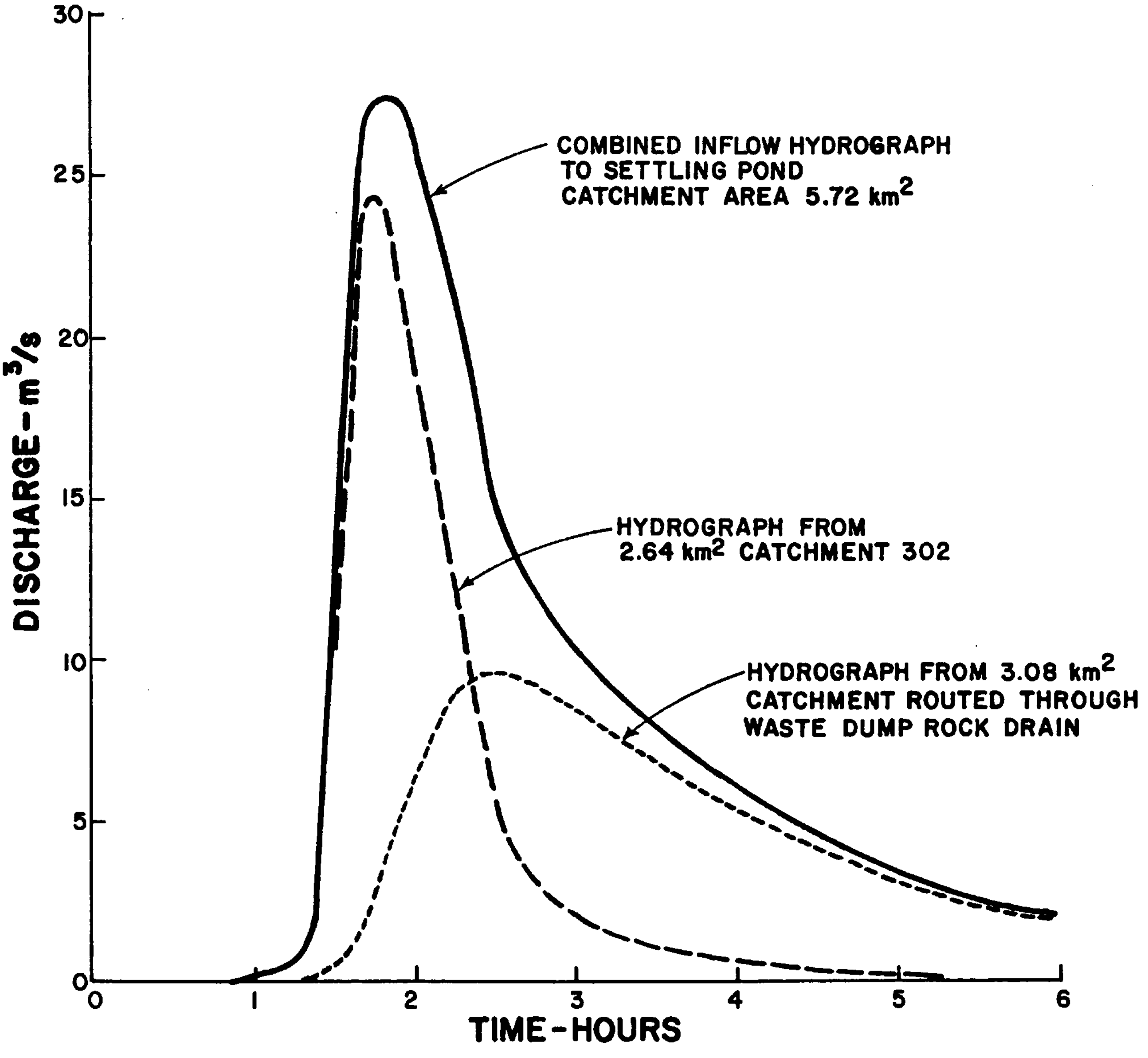


Figure 3

SETTLING POND NO. 3

200–YEAR INFLOW DESIGN HYDROGRAPHS

AMAX planned that, following successful completion of the test dump monitoring phase, placement of waste would proceed in both Patsy and Lime valleys. The ultimate pit will extend across the lower reaches of Patsy Creek, which will be diverted along a bench in the south wall of the pit in about year 5. Normal flows of Lime Creek below the confluence with Patsy Creek were expected to be conveyed through the waste dump, while a high level diversion was to be excavated in rock to carry flood flows around the dump.

The mine was closed indefinitely in October, 1982 due to low metal prices. During the operating period, negotiations continued with the federal government (Department of Fisheries and Oceans and the Environmental Protection Service (EPS)) and with the provincial govornment (Waste Management Branch and Mines Branch). While the concerns of government reviewers were near to being resolved, no agreement was in place at the time of shutdown to allow the test fill to proceed. As AMAX were occupied with shutdown activities in 1983, discussions and further studies of the waste dumps were suspended. In early 1985, AMAX requested Kohn Leonoff Ltd. to review the status of the proposed valley fill dumps and to propose technical measures which would allow permits to be issued for construction of the test fill and also to determine a feasible scheme for the ultimate valley fill dumps. AMAX's objective is to obtain approval for the Patsy Creek test fill and to develop a sound scheme for design of the ultimate dumps. AMAX wish to obtain final approval to build the test dump so that they may proceed immediately on re-opening the mine. The present mine plan allows waste to be placed in the Clary Dump for two years. Beyond that time, the waste-to-ore ratio must be increased to maintain the rate of mill feed. Capital expenditures for additional haul trucks would be required for the long uphill haul to Clay Dump if Patsy and Lime Dumps could not be developed.

At the outset of the assignment, Klohn Leonoff reviewed AMAX's files of previous reports and correspondence on the waste dumps. Some of the significant conclusions of this initial review were as follows:

a) The waste rock produced at Kitsault has a finer gradation than is found at most British Columbia metal mines. End-dumped waste rock in Patsy and Lime Creeks may not have sufficient flow-through capacity to convey peak flood flows. Saturation of a significant proportion of the dumps would occur during flood flows.

b) Sediment transport from the dumps is a major concern of the Environmental Protection Service (EPS). It is difficult to demonstrate theoretically or by model studies that levels of suspended solids can be satisfactorily controlled if part of the stream-flow is conveyed through end-dumped rockfill. It would, therefore, be advantageous to construct drains so that the design flows are conveyed entirely by clean rockfill and more certain predictions of levels of suspended sediments could be made.

c) The previous scheme for the waste dump included a sidehill diversion excavated in the steep southern slope of Lime Creek valley to bypass flood flows around Lime Dump. In Klohn Leonoff's opinion, such a diversion would not be stable for abandonment and would require considerable ongoing maintenance.

d) The peak flood flows derived from previous hydrologic studies appeared high. A re-analysis of the hydrology should result in considerable reduction in design flows.

e) The waste rock contains enough large, durable fragments which may be selected and utilized for drain construction.

Hydrology

Estimation of Flood Flows

As the first stage of review of the Kitsault waste dumps, Klohn Leonoff re-examined the hydrologic analysis which had been used to estimate design streamflow at the Patsy Creek test dump site. Previous hydrologic analyses had been based on a single very high flow estimate of 300 m^3/s at the mouth of Lime Creek (catchment area 39.4 km^2) during a high rainfall event and Water Survey of Canada (WSC) agreed that the estimate was incorrect. The 300 m^3/s flow estimate for November 1, 1978 has been deleted from WSC records.

Klohn Leonoff used three different methods of analysis to estimate peak flows in the Lime

Creek catchment: (1) a regional flood frequency analysis using records from both British Columbia and Alaska; (2) multiple regression equations developed for southeast Alaska by the U.S. Geological Survey; and (3) hydrograph modelling. A good agreement was obtained between the three methods of analysis. The recommended 100-year peak flows are much lower than those previously adopted (Table 2).

TABLE 2

COMPARISON OF RECOMMENDED 100-YEAR DESIGN FLOWS WITH PREVIOUS ESTIMATES

Location	100-YEAR PEAK FLOW (m^3/s) Recommended by KLL	Previous Estimate
Lime Creek @ the mouth (39.4 km^2)	121	402
Patsy Creek @ Test Dump Site (6.1 km^2)	26	60

For purposes of design of rock drains for the waste dumps, a 200-year return period storm runoff is considered appropriate. The 200-year peak streamflow was determined using a factor of 1.2 to convert the estimated 100-year flow to a 200-year flow. The factor of 1.2 was estimated based on the ratios of 200-year to 100-year peak flows for other gauging stations in the region.

Hydrologic Model

In order to develop design hydrographs for flood routing through the waste dumps, a hydrograph model was developed for the Lime and Patsy Creek catchments. This model was calibrated so that peak flows derived from the other hydrologic analyses coincided with peak flows estimated using the model. The model selected for determining the rainfall excess and transformation of the rainfall excess to streamflow was a microcomputer version of OTTHYMO, originally developed by the U.S. Department of Agriculture and subsequently modified by the University of Ottawa. This model allows the flexibility of dividing a catchment into smaller sub-catchments, generating hydrographs from the smaller areas and then progressively adding the hydrographs in a downstream direction. The model also has the capability of routing hydrographs through channels or reservoirs. It is thus well suited to evaluating the effect of flow-through waste dumps on reduction of hydrograph peaks.

The streamflow was modelled by subdividing the Lime Creek catchment into seven sub-drainage areas, as shown on Figure 7. The model was calibrated so that a peak flow was generated at the mouth of Lime Creek equal to the 200-year flood flow of 141 m^3/s estimated from the hydrologic analysis.

The routing procedure was carried out as follows:

a) The hydrographs generated from catchments number 301 and 302 were combined and routed through the rock drain in the Patsy Creek Dump. The drain was represented by a stage-discharge curve which was derived from a typical Patsy Creek cross-section and a velocity determined from the flow-through rockfill formula.

b) The routed flow hydrograph from the Patsy Creek Dump was combined with the hydrographs from catchments 303, 304 and 305 to determine the inflow hydrograph to the Lime Creek Dump. This combined hydrograph was in turn routed through the Lime Dump rock drain.

c) The outflow hydrograph from the Lime Dump was added to catchments 306 and 307 to determine the flows at the mouth of Lime Creek following routing through the waste dump rock drains.

Table 3 shows the design flows determined by the OTTHYMO program for the rock drains with the 200-year flood routed through the system. The effect of the rock drains is to decrease the 200-year peak flow at the mouth of Lime Creek from 141 m^3/s under natural conditions to about 82 m^3/s with both drains in place.

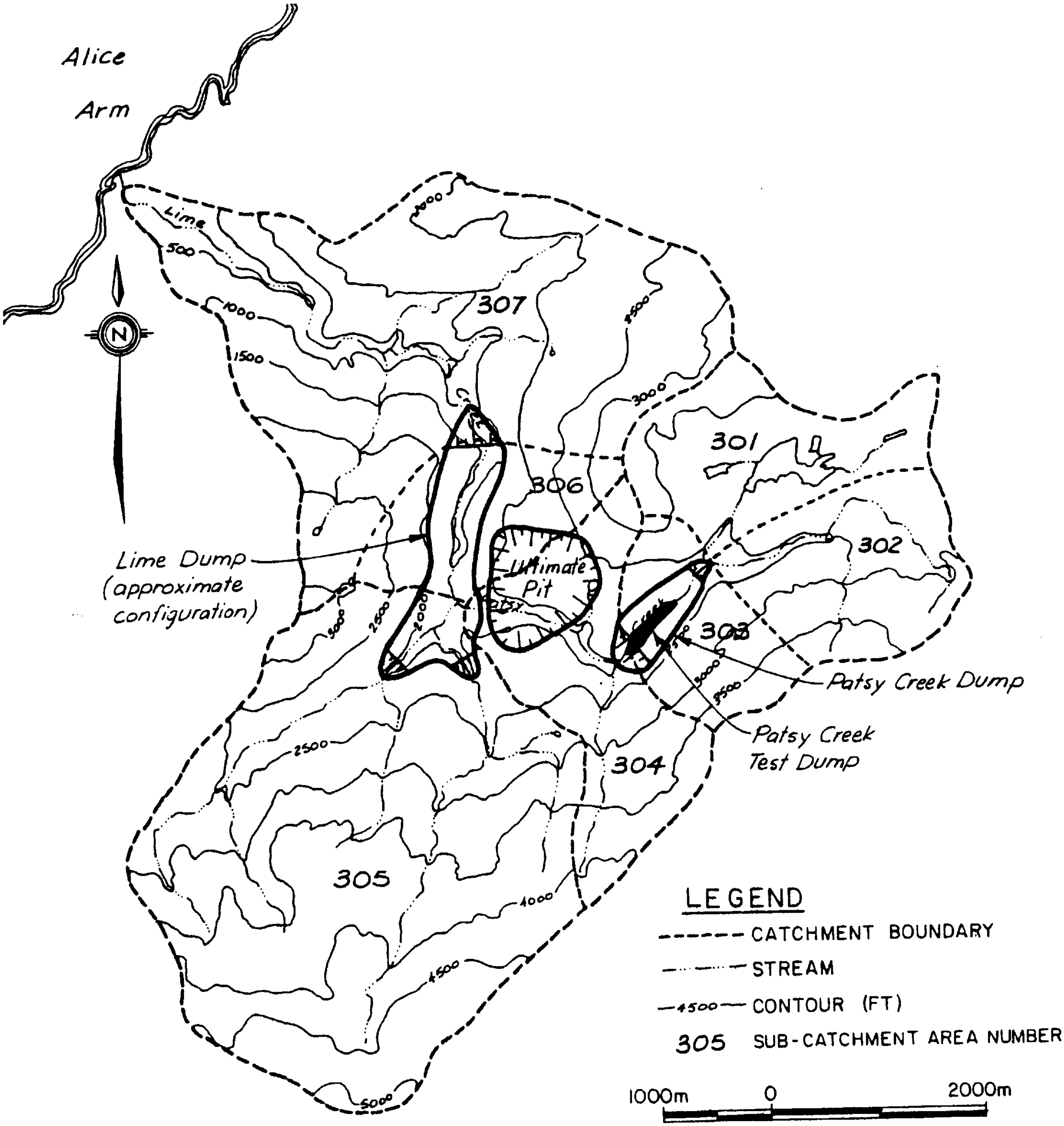


FIGURE 7

AMAX KITSAULT MINE

TABLE 3

SUMMARY OF DESIGN FLOWS

Location	Total Catchments Area (ha)	200-Year Peak Flows (m^3/s)
Patsy Cr. Dump		
-inflow	620	27.7*
-outflow	740	21.5
Lime Cr. Dump		
-inflow	2500	82.7
-outflow	2720	70.1
Patsy Cr. @ the mouth	3940	81.7

* Inflow to the Patsy Creek dump is a natural flow. All other flows are less than those that would occur under natural conditions due to the routing effects of the rock drains.

Design of Flow-Through Rock Drains

The proposed concept for design of the Kitsault waste dumps is to construct rock drains capable of passing the design flood flows. The drains will be constructed of select waste rock placed directly in the stream bottoms and waste rock will be end-dumped over the drains. The advantages of flow-through rock drains are as follows:

1. Sediment transport will be reduced to acceptable levels because the entire streamflow will be conveyed through clean, coarse rock.
2. The flow-through drain will be suitable for abandonment, by provision of a large entrance capacity to allow for some siltation upstream.
3. There will be no requirement for expensive and maintenance-intensive sidehill diversions.
4. Kitsault will be able to utilize the economical waste disposal locations in Patsy and Lime Creeks.
5. Rock drains are a proven concept which have been successfully implemented at other British Columbia mines.

The proposed waste dump drains are designed with two layers of select rock. The lower layer, placed directly on the stream bottom, will be constructed of 1 m average diameter quartz diorite boulders. These boulders will be individually selected from the waste rock. This coarse lower zone will be designed to carry one-third of the 200-year return period design flow. This will ensure that all normal flows will be carried by the coarse rock drain.

The remaining flow capacity for peak floods will be carried by a zone of clean rockfill obtained by passing waste rock over a grizzly. For design purposes, a mean rock size (D_{50}) of 400 mm has been assumed for the grizzlied zone.

The required flow capacities for the rock drains were determined by the OTTHYMO computer program. Required areas were determined from the above formulae, using the following assumptions:

Void ratio		
Boulders	$e =$	0.8
Grizzlied waste	$e =$	0.65
Hydraulic mean radius		
Boulders	$m =$	131 mm
Grizzlied waste	$m =$	49 mm
Hydraulic gradient	$i =$	0.06

The required drain areas and fill volumes are summarized in Table 4. The drain areas shown are the average for each drain. The actual areas will vary, as the drain will be designed to convey the appropriate routed peak flows at the upstream and downstream ends of the dump.

During the final six months of production in 1982, Kitsault selected and stockpiled approximately 50 000 tonnes of quartz diorite boulders. Based on this experience, the rate of boulder recovery for the rock drains is set for design purposes at 100 000 tonnes per year.

The rock drains must carry not only the flow entering at the upstream ends of the dumps but also flow from small streams discharging onto the dump from the valley walls. These smaller streams will be handled by constructing appropriately sized tributary drains.

TABLE 4

SUMMARY OF ROCK DRAIN AREAS AND VOLUMES

	BOULDER DRAIN			GRIZZLIED DRAIN		
	Area (m^2)	m^3	Volume t	Area (m^2)	m^3	Volume t
Patsy						
Test Dump	50	25 000	38 000	185	90 000	160 000
Final Dump	50	110 000	165 000	185	380 000	680 000
Lime		260 000	390 000		1 000 000	1 620 000

Patsy Creek Test Dump Construction

The purpose of the proposed Patsy Creek test dump will be to demonstrate that the flow-through concept can be successfully applied and that the rock drain can be constructed and operated without excessive sediment production. The test dump will be located about 2000 m upstream from the confluence of Lime and Patsy Creeks, to the southeast of the open pit. The present Patsy waste dump haul roads will provide access to the top of the valley slopes at the test dump site. At the test dump location, the Patsy Creek valley slopes are as steep as 45°. The height of the valley wall on the dump side (northeast) is about 35 m.

Access to the bottom of the valley to construct rock drains will require excavation of a sidehill access ramp. The ramp will be put in rock, so that careful drilling, blasting and hauling the excavated material out of the valley will be necessary to avoid introducing sediment into Patsy Creek.

The large boulder base drain layer and the upper grizzlied layer will be advanced simultaneously, beginning at the base of the access ramp and then proceeding downstream. The boulder layer will be end-dumped from the surface of the grizzlied layer, which will be advanced just behind the boulder layer. This will allow the upper surface of the grizzlied layer to be smoothed and used for a travelling surface. The two-layer rock drain will be edvanced downstream about 200 m beyond the design toe of the test dump. This will allow an access ramp to be pushed down onto the drain so that construction can continue downstream when approval is granted for the full-scale Patsy Dump.

Following placement of the rock drain, general waste rock will be end-dumped from elevation 660 m to complete the test dump.

Two small tributary streams enter Patsy Creek on the southwest side of the valley. Drain rock material will be extended up these streams to allow the tributary flows to enter the main rock drain.

Construction of the rock drain in the test dump will be scheduled for lower flow periods, either in summer or winter. The drain construction period is expected to be about three months. Placement of the general waste will not effect the stream, as all flow will be within the rock drain, so this work need not be scheduled for any particular season. The general waste will have a volume of about 900 000 m^3 (1 500 000 t) and will require about three months to complete the placement.

The test dump will not require an overflow spillway. The drain will be designed to accommodate peak 200-year return period flood flows.

For stability purposes, the downstream toe of the rock drain will be constructed at a slope angle of 4 horizontal:1 vertical. The general fill above the rock drain will be stable at angle of repose as no saturation of the general fill will occur.

Test Dump Monitoring

The main objectives of monitoring the test fill will be to confirm the following aspects:

a) That excessive levels of sediment will not be generated during construction of the rock drains.

b) That the rock drains have sufficient hydraulic capacity to convey design streamflows beneath the general waste rock.

c) That downstream sediment loadings after completion of the drain and test dump satisfy environmental criteria.

Sediment Generation

The flow-through rock drain concept is expected to provide satisfactory performance with regard to downstream sediment loading. The streamflow will be carried entirely by the coarse rock drains. Some sediment loading will result initially as the coarse particles are washed by the stream following construction, but this is not expected to produce significant sediment quantities. The rock drain will be placed directly on the stream bottom, not end-dumped from a large height, so that disturbance of soils on the valley slopes or stream bottom will be kept to a minimum.

Downward movement of fines through the waste rock is not expected to occur to any significant extent. End-dumping will create a generally graded mass which will act as a graded filter to prevent fines percolating downward. The general waste will not be saturated by streamflow. Rainfall infiltration will create some downward percolation, but will not cause the dump to be saturated.

Some sediment generation will result from rainfall onto and runoff over the dump surfaces and the dump slopes. Sedimentation basins will be created on the dump surfaces and settled water will be discharged from these shallow basins to tributary stream drains. Runoff on dump slopes will be collected in ditches on individual benches. Essentially, sediment control from runoff and precipitation on dump surfaces will be no different than any other large open pit mine.

Long-Term Stability

The flow-through rock drain concept is expected to provide a more permament solution than the previous concept of sidehill diversions.

The rock drains will be constructed with large entrance capacities, by extending coarse rock up the upstream slopes of the final dumps. This will allow sediment to settle upstream of the dumps without affecting the performance of the drains. At any rate, upstream sediment is not expected to be a significant problem. Both Patsy and Lime Creeks flow in canyons cut in rock so that they carry little natural sediment. The waste dumps with flow-through rock drains should be suitable for abandonment without requiring significant ongoing maintenance. The sidehill diversions, on the other hand, would have required frequent inspection and maintenance to prevent overtopping caused by debris blockage from the numerous tributary streams. This would have been a risky and possibly unacceptable solution for final abandonment.

CONCLUSIONS

In summary, the advantages of constructing a flow-through rock drain as part of a mine waste dump, are as follows:

- allows a convenient waste disposal location in valley fill;

- avoids costly stream diversion and erosion control measures;

- allows flexibility in dump construction;

- constructed at no cost to the mine other than requirement for selective disposal of waste rock;

- has significant flood routing effect which reduces downstream spillway construction costs.

By the application of sound engineering principles, waste dumps can be designed to safely carry significant streamflow, both during mine operation and following abandonment. The rock drain at Bullmoose has operated successfully to date and is an excellent example of the use of these principles to develop a safe and economical design.

REFERENCES

1. Nichols, R.S. and Rutledge, A.L. (1982) "Water Drainage Through Waste Dumps at Fording Coal Ltd." CIM Bulletin Vol. 75, No. 844.

2. Sellars, C.D., Lighthall, P.C. and Robertson, J. (1984) "Cost-Saving Water Management Strategies for Control of Run-off from Mine Developments", Canadian Water Resources Association 37th Annual Conference, May 16 to 18, 1984.

3. Wilkins, J.K. (1965) "Flow of Water Through Rockfill and its Application to the Design of Dams", Proceedings of the Second Australian Soil Mechanics Conference.

DESIGN AND CONSTRUCTION PRACTICES IN ROCK DRAINS

by

Jonathan D. Ventura, Civil Engineer
U.S. Department of the Interior
Office of Surface Mining
Reclamation and Enforcement
Pittsburgh, Pennsylvania

ABSTRACT

The underdrain system in an excess spoil fill structure is provided to collect underground water flows and water flows from infiltration from surface water. The national performance standards for surface mine activities includes requirements to provide environmental protection and protection of property. These standards also include requirements for underdrain systems. This paper provides a review of these performance standards and general design and construction of current practices.

INTRODUCTION

Major earth structures are usually created on most surface coal mining operations in the Appalachian coalfields in the States of West Virginia, Virginia, and Kentucky from the disposal of excess spoil materials. Excess spoil materials are generally defined as the overburden materials that are not required to regrade the disturbed surface to the approximate original contour (AOC). The excess materials result from both the swell of the overburden during excavation and placement, and from variances that can be approved from the AOC requirements.

The size and number of these earth structures vary significantly depending on different site conditions and site economics. Structures ranging in size up to ten million yards of materials end vertical heights up to 500 feet have been constructed.

The topography in these areas can be generally characterized as narrow valleys with steep side slopes (plus 20°). The relief can be as much as 1,000 feet in some areas. The geology of these areas can be described as a stratigraphic interval of cyclic deposits of sandstone, sandy shales, silty shales, and coal. The spoil usually consists of a mixture of sandstone and shales.

The excess spoil structures are usually located across a small valley and/or near the head of the drainage areas. When located at or near the top of the drainage ridge and constructed with a center core drainage system, these structures are called head-of-hollow fills; otherwise, they are labeled valley fills (see Figure No. 1). When the materials consist of 80 percent hard rock (durable rock), all materials can be end-dumped in place. These structures are further labeled or called end-dump valley fills (see Figure No. 2). A third type is the zone fill (see Figure No. 2). In these fills, the front face area is built in accordance with stability requirements and the back zone is placed without any engineering controls. The back zone can be end-dumped without regard to spoil type; that is, durable versus non-durable.

In response to Public Law 95-87, national rules and regulations (30 CFR 816.71-816.74) were developed to regulate all surface mining activities. The basic purpose of the legislation was for the protection of people and property, land, water, and other resources and aesthetic values and for the restoration and reclamation of surface areas affected by mining. The legislation also require each State regulatory agency to develop regulatory programs which were as effective as the national rules.

These rules established performance standards for the various surface mine activities, including excess spoil structures. The performance standards for excess spoil structures specify requirements for the location of areas used for disposal sites and the design, construction, and inspection of these structures.

This paper presents a general summary of these performance standards and relates how the performance standards for the underdrain system are met by the current design and construction practices under the various types of disposal methods described above.

PERFORMANCE STANDARDS FOR EXCESS SPOIL DISPOSAL

This section consists of an overview of performance standards to be followed during mining and reclamation activities. These standards describe the general location, design, construction, and inspection requirements. Added performance standards governing head-of-hollow (rock core drains) fills, end-dump fills, and zone fills design and construction are discussed separately.

a. General Requirements

The general requirements are that the excess spoil must be placed in designated disposal areas, within a permit area, in a controlled manner to minimize the adverse effects of leachate and surface water runoff from the fill on surface and ground water. The placement manner must ensure mass stability and prevent mass movement during and after construction. The final fill must be suitable for reclamation and revegetation and compatible with the natural surroundings and the approved postmining land use.

b. Qualification of Designer

To meet design requirements, the fill and appurtenant structures have to be designed using current, prudent engineering practices by a qualified, registered professional engineer experienced in the design of earth and rock fills who must certify the design of the fill and appurtenant structures.

c. Stability

The fill has to be designed and constructed to attain a minimum long-term static safety factor of 1.5. The foundation and abutments of the fill and all other features have to be sufficient to ensure stability of the fill and appurtenant structures under all stages and conditions of construction. The disposal area has to be located on the most moderately sloping and naturally stable area available.

Stability analyses have to be performed by a qualified, registered professional engineer. Parameters used in the stability analyses have to be based upon adequate investigations of foundation and fill materials, including field reconnaissance; subsurface investigations; and data obtained from laboratory analyses of such materials. The analyses of foundation conditions have to take into consideration the effect of any underground mine workings upon the stability of the fill and appurtenant structures. If the toe of the fill rests on an area which has a natural land slope in excess of 2.8h:lv (36 percent) keyway cuts and/or rock toe buttresses have to be constructed to ensure stability of the fill.

d. Clearing and Grubbing

Vegetative and organic materials have to be removed, either progressively or in a single set of operations, from the disposal area prior to placement of the excess spoil. Topsoil has to be removed, segregated, and stored and/or redistributed.

e. Placement

The excess spoil has to be transported and placed in a controlled manner in horizontal lifts not exceeding four feet in thickness (or less if required to achieve the density necessary to ensure mass stability and to prevent mass movement, to avoid adverse impacts on the rock underdrain or rock core, or to minimize the formation of voids); concurrently compacted as necessary to ensure mass stability and to prevent mass movement during and after construction; graded so that surface and subsurface drainage is compatible with the natural approve designs which incorporate placement of excess spoil in lifts greater than four feet in thickness, if it is demonstrated in the permit application and certified by a qualified, registered professional engineer that the design ensures the stability of the fill and the design complies with all other requirements.

f. Erosion Control Measures

The general grading rquirements are also to minimize erosion. The top of the fill has to be graded no steeper than 20h:lv (five percent) toward properly designed drainage channels in natural ground along the periphery of the fill. Surface runoff from the top surface of the fill must not be allowed to flow over the outslope of the fill. The outslope of the fill cannot exceed 2h:lv (50 percent).

Terraces are usually constructed on the outslope of the fill for stability, for control of erosion, to conserve soil moisture, and to facilitate the approved postmining land use. Terrace benches have to be graded with a three to ten percent slope toward the fill. The outslope between terrace benches cannot exceed 2h:lv (50 percent) or such a lesser slope as may be required to ensure stability or minimize erosion. Runoff has to be collected by a ditch along the intersection of each terrace bench and the outslope. This ditch has to route runoff to stabilized diversion channels and cannot have a maximum slope greater than 20h:lv (five percent) unless a steeper slope is necessary for permanent roads in conjunction with an approved postmining land use and a steeper slope will not adversely affect the stability of the fill or result in excessive erosion.

g. Underdrains

Underdrains must be constructed of durable, nonacid-forming, and nontoxic-forming rock; free of coal, clay, and nondurable material; and must be designed and constructed using current, prudent engineering practices. The underdrain system must be designed and constructed to carry, away from the fill, the maximum anticipated seepage of water due to precipitation and the maximum anticipated seepage and discharge from seeps and springs in the foundation of the disposal area and has to be protected from piping and contamination by a filter system designed and constructed to ensure proper long-term functioning of the underdrain using current, prudent engineering practices.

The minimum cross-sectional dimensions of the underdrain must be as specified below unless the applicant demonstrates in the application, through detailed analyses, that alternative cross-sectional dimensions will provide adequate long-term capacity for drainage at the site. In constructing the underdrain, no more than ten percent of the rock can be less than 12 inches in size and no single rock can be larger than 25 percent of the width of the segment of the underdrain in which the rock is located.

Perforated pipe underdrains can be substituted for the rock underdrain. Perforated pipe underdrains have to be corrosion resistant; have characteristics consistent with the long-term life of the fill; be designed and constructed using current, prudent engineering practices; be designed and constructed to carry, away from the fill, the maximum anticipated seepage of water due to precipitation and the maximum anticipated discharge from seeps and springs in the foundation of the disposal area; and be protected from clogging and contamination functioning of the perforated pipe underdrain using current, prudent engineering practices.

For fill construction in a valley, an underdrain system has to be installed along the natural drainageways; extend from the toe to the head of the fill; and contain lateral drains to each area of potential drainage or seepage.

For situations in which excess durable rock spoil is placed in single or multiple lifts such that the underdrain system is constructed simultaneously with excess spoil placement by the natural segregation of dumped materials, color photographs are taken of the underdrain as the underdrain system is being formed.

h. Inspections

A qualified, registered professional engineer, or other qualified professional specialist under the direction of a professional engineer, has to periodically inspect the fill during construction. The professional engineer or specialist hes to be experienced in the construction of earth and rock fills. Inspections of the fill site have to be made during critical construction periods as necessary to ensure compliance with this regulation. Critical constructuion periods have to include at a minimum: foundation preparation including the removal of all organic material and topsoil; placement of underdrains and protective filter systems; installation of final surface drainage systems; completion of the final grading; and completion of the initial revegetating of the completed fill. In addition to the above, inspections of the fill site have to be made, beginning at the initial site-preparation phase of construction, at least once every three months throughout

MINIMUM DRAIN SEGMENT CROSS-SECTIONAL DIMENSIONS

*Total cumulative volume of fill material to be drained by segment	Predominant type of fill material	Minimum size of drain segment (in feet)	
		Width	Height
Less than 1,000,000 yd^3	Sandstone	10	4
	Shale	16	8
More than 1,000,000 yd^3	Sandstone	16	8
	Shale	16	16

*The underdrain may be divided into segments for purposes of determining required dimensions of the individual drain segments. Each segment will drain the volume of fill overlying the segment plus carry the water drained to the segment from areas of the fill located upstream of the segment. Where the cumulative volume of the fill material to be drained by a segment is less than 1,000,000 yd^3, the small dimension may be used.

construction of the fill, including during placement and compaction of fill materials.

The qualified, registered professional engineer must provide a certified report to the department's appropriate regional office within two weeks after each inspection stating that the fill has been or is being constructed and maintained as designed and is in accordance with the approved plan and all appropriate regulations. The report has to address appearances of instability, structural weakness, and other hazardous conditions. The certified reports on the drainage system and protective filter have to include color photographs taken during and after construction of the underdrain and protective filter but before the drainage system is covered with excess spoil. If the underdrain system is constructed in phases, each phase has to be certified separately.

i. Surface Drainage

Impoundments are not allowed on the completed fills. Drainage requirements are established for surface runoff and ground water as necessary to control erosion, minimize water infiltration into the fill, and to ensure stability.

Surface water runoff from the area above the fill must be diverted away from the fill and into stabilized diversion channels designed to pass safely the runoff from a 100-year, 24-hour precipitation event. Surface runoff from the fill surface has to be diverted to stabilized channels off the fill.

PERFORMANCE STANDARDS FOR ROCK-CORE CHIMNEY DRAINS

Rock-core chimney drains can be employed in a head-of-hollow fill instead of the subdrain and surface runoff diversion system normally required, as long as the fill is not located in an area containing an intermittent or perennial stream.

a. Design and Construction

The fill must have, along the vertical projection of the main buried stream channel or fill, a vertical core of durable rock at least 16 feet thick which extends from the toe of the fill to the head of the fill and from the base of the fill to the surface of the fill. A system of lateral rock underdrains has to connect this rock core to each area of potential drainage or seepage in the disposal area. The underdrain system and the rock core must be designed and constructed to carry, away from the fill, the maximum anticipated seepage of water due to precipitation and the maximum anticipated discharge from seeps and springs in the foundation of the disposal area.

b. Filter System

A filter system to ensure the proper long-term functioning of the rock core has to be designed and constructed using current, prudent engineering practices.

c. Drainage Control

The drainage control system has to be capable of safely passing the runoff from a 100-year, 24-hour precipitation event or a larger event, if specified.

PERFORMANCE STANDARDS FOR END-DUMPED FILLS

This alternative method of disposal can be employed when at least 80 percent of the spoil consists of durable rock. The alternative method allows the spoil to be placed by gravity placement in single or multiple lifts.

All noncemented and poorly cemented shale, clay, soil, and nondurable excess spoil materials disposed of in the fill have to be distributed within the fill by selective dumping or other adequate methods of placement to avoid localized concentrations of nondurable materials which would adversely affect the stability or internal drainage of the fill. The fill has to be designed and constructed to attain a minimum long-term static safety factor of 1.5 and a minimum earthquake safety factor of 1.1.

The underdrain system can be constructed simultaneously with excess spoil placement by the natural segregation of the dumped materials provided that the resulting underdrain system is capable of carrying, away from the fill, the maximum anticipated seepage of water due to precipitation and the maximum anticipated discharge from seeps and springs in the foundation of the disposal area and provided that the other requirements for drainage control are met. For situations (such as the dumping of fill material from an insufficient height or on an insufficient slope) in which, in the judgment of the regulatory authority, the natural segregation of dumped materials will not form an adequate underdrain system, the underdrain system has to be separately constructed.

PERFORMANCE STANDARDS FOR ZONE FILL

Additional performance stendards were included in some State Regulatory Programs to allow for placing the spoil in zones consistent with the other performance standards. The additional standards cover the placement of spoil in the different zones.

In the structural zone, spoil has to be placed in approximately horizontal lifts and compacted to densities and strengths required to ensure mass stability and to prevent mass movement. The life thickness and grading requirements of the spoil has to be consistent with the design parameters. The extent of this zone has to be based on accepted engineering analyses.

The temporary outside slope of the nonstructural zone cannot exceed the angle of repose and the height of the nonstructural zone has to be limited to a height determined not to pose an actual or probable hazard to property, public health and safety, or the environment in the event of failure during construction. The structural zone and the nonstructural zone have to be constructed as concurrently as practicable and the distance between the structural zone and the nonstructural zone has to be minimized to assure proper stability and control of the temporary fill slope.

ROCK DRAIN SYSTEMS

The types of rock drain system designed and constructed depends on the type of excess spoil structure to be utilized by the operator for placement of the materials; that is, a valley fill placed in four-foot lifts, a valley fill placed by the end-dump method, a valley fill built by the zone concept, or a head-of-hollow fill.

On most operations, the rock drain is constructed of a good durable sandstone which usually comes from a strata of overburden to be removed in exposing the coalbed or beds. Therefore, a separate material cost is not usually necessary. The perforated pipe system is rarely used. With the exception of the end-dump fills, the filter system is usually a synthetic fabric type.

When a valley fill is constructed with four-foot lifts, the underdrain system is installed as a separate operation prior to the placement of spoil materials. A design procedure is presented in OSMRE's Engineering and Design Manual - Disposal of Excess Spoil (Holmquist, et. al. 1983, p. 139). However, the usual procedure is to use the cross-section area mentioned in the performance standards. The usual construction specifications also repeats the performance standards.

When the head-of-hollow fill structure is constructed, the rock-core series of particle sizes from the four to five-foot boulders along the original ground line to sand and silt-size particles at the top. The rolling of materials down the slope naturally segregates the particle sizes. As the durability of the material decreases so that a higher percentage of smaller size particles is generated in the mining process, the natural segregation process is adversely effected and closer inspection is required to assure that an underdrain system develops.

Experience with the zone fill has been limited. The underdrain system for this type of fill is usually constructed similar to the valley-fill method.

PERFORMANCE

It is estimated that at least 200 excess spoil structures are completed each year in the Appalachian coal fields. Since the implementation of the national standards, only three excess spoil structures are known to have failed and required remedial reclamation. These structures were constructed by the end-dump process and had narrow bases with a steep natural ground profile, resembling a side hill-type fill geometry. Due to a lack of detailed inspection documentation, it could not be determined whether or not an effective underdrain system was developed during construction in these fills. The standard for a plctorial record of the underdrain system is a recent addition now being implemented.

Limited instrumentation monitering has been done on the effectiveness of underdrain systems. The head-of-hollow type structures have been built under West Virginia program standards since the early 1960's. No failures have been reported in any of these structures. Concerns have been expressed with eventually clogging these center core drains with sediments carried by surface water. This has not happened on any structure to date. Usually, the location of structures is such that only small a drainage area contributes flows into the drains. Strict revegetation requirement enforcement has also limited sediment loads.

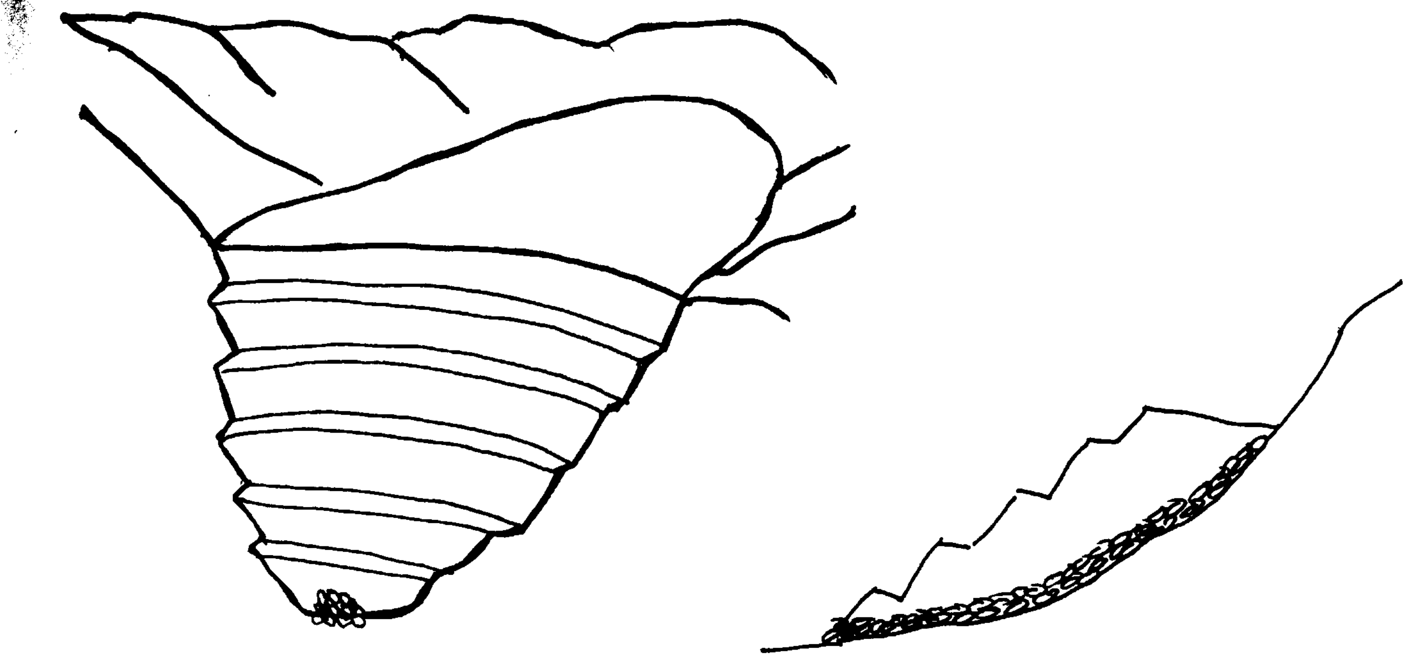
CONCLUSIONS

The minimum standards established by regulatory agencies have been the main source of guidance for coal operators for design and construction of underdrain systems. While the standards may be established more from practical experience than theory, these standards have been effective in keeping failures within reasonable tolerances. This leads to the further conclusion that these standard underdrains have adequate capacity to keep a water level from developing within these fills.

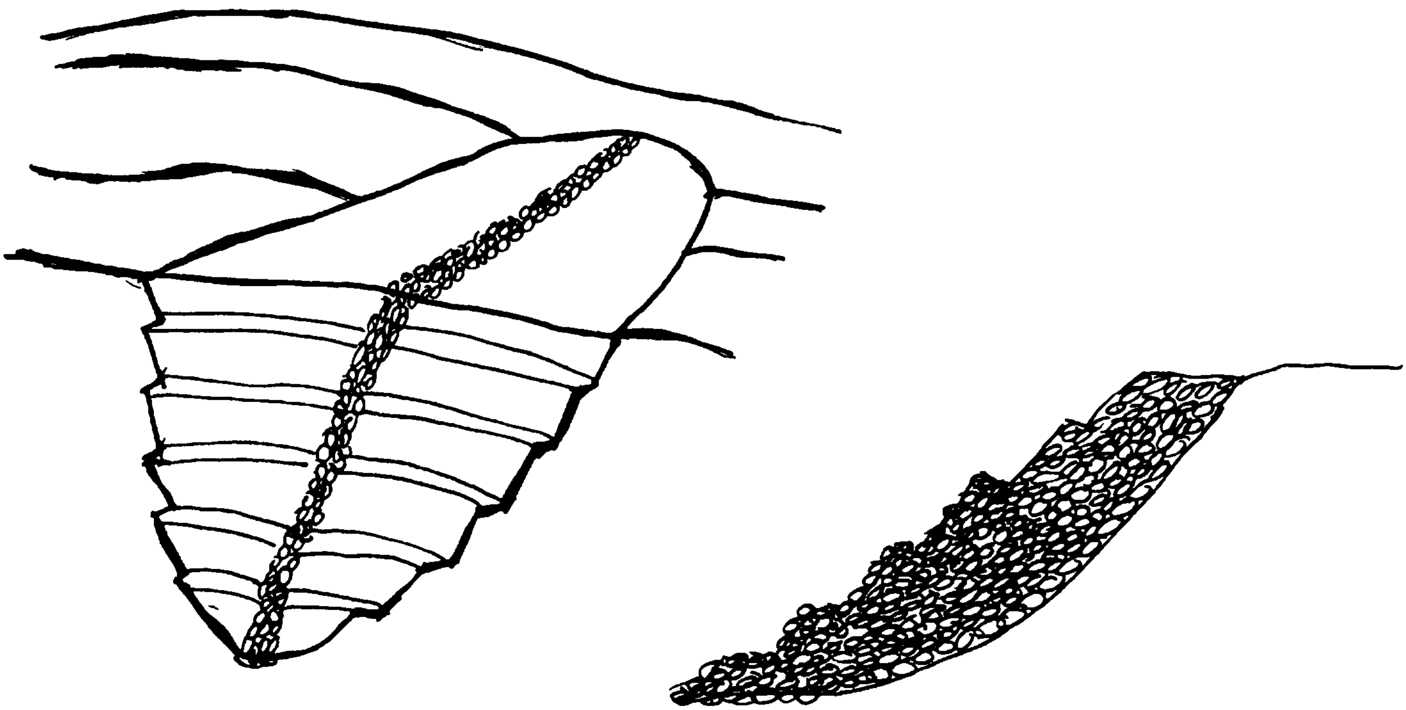
REFERENCES

Holmquist, D.V., McOmber, R.M., and Roberts, J.M., "Engineering and Design Manual for Disposal of Excess Spoil." Office of Surface Mining, OSM/TM-2/83.

United States Code of Federal Regulations, Tital 30, Mineral Resources, Chapter VII, Enforcement, Department of Interior, Sections 816.71-816.74.

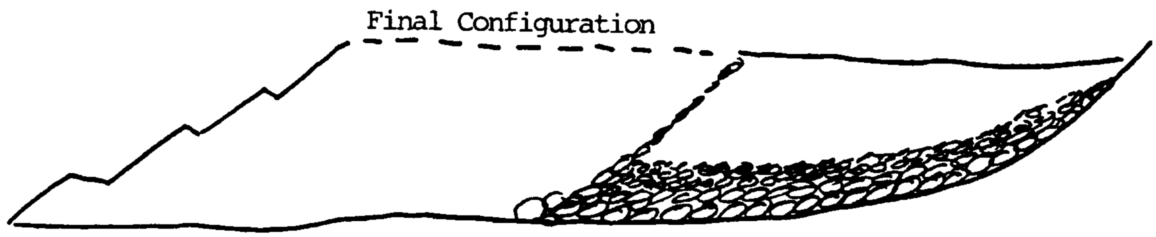


VALLEY FILL

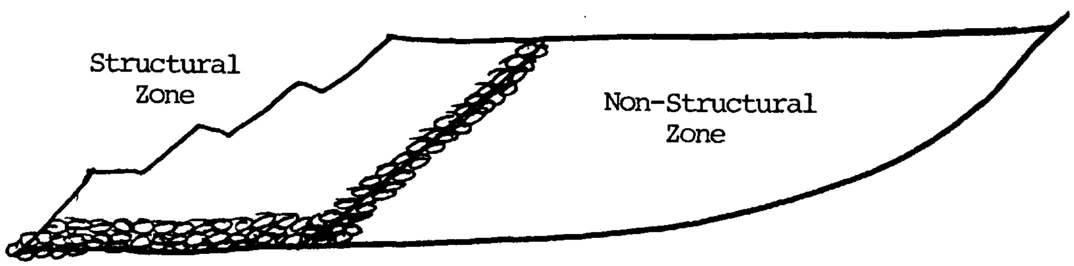


HEAD-OF-HOLLOW

FIGURE NO. 1



END DUMP FILLS



ZONE FILLS

FIGURE NO. 2

GEOTECHNICAL CONSIDERATIONS FOR ROCK DRAINS

by

C.O. Brawner, P.Eng.
Professor of Geomechanics, Dept. of Mining
Engineering, University of British Columbia
and
Consulting Geotechnical Engineer
Vancouver

INTRODUCTION

The use of rock drains to pass water under waste piles has substantial practical and economic benefits to handle mine drainage. Many waste piles are required on mountain areas. They are constructed on moderate to steep slopes, frequently extend for considerable length and wrap around the mountain. Heights in excess of 200-300 meters are becoming common.

Large volumes of waste are often involved, hence locations having large storage capacity are required. This tends to result in extensive use of gullies and creek channels. It is essential that facilities be developed in the gulley and creek bottoms to catch and carry underseepage and creek flow. The design of these underdrains from a geotechnical standpoint is very important.

Design considerations must include foundation stability and erosion resistance, resistance of drain rock to abrasion and wear, flow capacity, drain siltation protection and stability and spreading of the spoil pile.

The life expectancy of the rock drain must be addressed and reclamation design plans must be formulated at an early date.

FOUNDATION STABILITY

Foundation stability of any gully dump must be adequate so that rock drain will not be pulled apart by foundation movement, either differential settlement or spreading.

This requires a geotechnical evaluation of foundation strata. It is important to determine the existence and depth of overburden organics, the depth of overburden soil and its shear strength, drainage and consolidation characteristics.

If bedrock is shallow, the bedrock type, orientation, and dip angle of discontinuities must be determined. If weak discontinuities dip parallel to or out of the slope the shear strength along these weaknesses must be determined. The soil-bedrock interface must be located.

If the soil or rock conditions are unfavorable it is necessary to perform a stability analysis to assess stability. Where a spoil pile failure could result in serious problems or damage a minimum safety factor of 1.5 is recommended. If failure would result in negligible problems or damage a safety factor of 1.2 should be acceptable. If earthquake stresses may develop they must be included in the analysis.

The geotechnical profile may be determined with backhoe test pits or boreholes. If continuous exposed bedrock exists, surface geological mapping may be adequate. Under high spoil piles test pits should be the minimum investigation requirement.

The foundation stability for the rock drain itself must be adequate. In gullies that flow water intermittently or where glacial lake sediments occur weak soils may exist under the proposed rock drain. The weak soils must be removed otherwise the rock drain may become disrupted.

FOUNDATION EROSION RESISTANCE

Where moderate to high flows are to be expected in the rock drain, erosion at the base of the drain may occur in the foundation materials. This erosion will likely be differential. It is obvious that if the natural base of the gully or creek channel will scour or erode that an erosion resistant base must be provided.

The design of this erosion resistant base will depend on flow volumes and soil or rock conditions. Each location will be slte specific.

A graded filter (Figure 1) (Terzaghi & Peck, 1967) with or without reinforced geotextiles, involving a sandy gravel base over medium to coarse crushed rock under the rock drain is one option. Where gradients are steep, fibre reinforced shotcrete with silicon fume admixture may be considered (Morgan, 1986). Site access, materials availabilify and cost will be considerations in the design selected.

DRAIN ROCK REQUIREMENTS

The drain rock may be subject to very high loads. Therefore the rock must have high point to point contact strength. If the contact points crush the rock drain will move internally, disrupt the top filter and reduce the flow capacity. Rounded strong glacial deposited rock or river rock is better than angular blasted rock if it is available.

Blasted rock may be damaged by the blasting and be subject to gradual disintegration.

The rock must be resistant to dirty water abrasion.

The water chemistry should be determined to assess chemical deteoriation of the rock.

The drain rock should meet the standard requlrements for rip rap to be placed elong a fast flowing river.

The size and gradation of the drain rock will be a function of the flow volume. The more uniform the gradation the better the flow characteristics. Substantial overdesign must be built into flow volume determination.

DRAIN TOP FILTER

Some form of filter is required over the drain rock to prevent fine particles from entering and eventually plugging the drain.

The waste rock always contains some fines developed by blasting, loading, hauling, dumping and slaking. Sedimentary shales, slates, claystones, mudstones and siltstone and weathered sandstones are usually the source of the fines, particularly on coal projects. As drainage, snow melt and precipitation seep down through the spoil pile these fines are carried down to the base layer of the soil. They must not be allowed to contaminate or plug the rock drain. This would also inhibit side hill seepage under the spoil pile and build up pore water pressures to reduce stability. This can be more serious during the low flow period in the drain.

Several designs are available for this cover zone. A standard granular filter may be used. A cover of medium to coarse rock is placed on top of the drain. This is followed by one or more finer granular layers. Each layer must meet filter design requirements. These requirements are shown in Figure 1. Reinforced geotextile may be placed between the filter layers. Alternatively reinforced geotextile may be placed directly on top of the rock drain with a cover of spoil about 2 meters thick to protect it from damage. Another alternative is to cover the drain with compacted clayey till to act as a seal. The drain cover must be placed as an impact layer to protect the drain from disruption by high speed bouncing and rolling waste rock.

An alternative design can be used if the majority of the waste rock is very competent with low fines. The drain base is required as usual. The drain can be developed by placing drain rock at least 4 meters wide at the base and dumping the good quality waste rock directly above the base drain for the full height of the dump.

CONSTRUCTION

The development of rock drains in the field should be done during dry weather conditions to minimize base contamination.

If shotcrete is used some durable drain boulders should be placed in the base first and shotcreted around. These will provide a key for stability.

The granular under filter should receive moderate but not heavy compaction to ensure moderate permeability. In this way the drain will allow underseepage pressures to reduce to drain elevation.

The rock drain material can be placed by back dumping and leveling or by crane.

The dimensions of the drain are a function of design volume, grade, porosity safety factor etc. They are beyond the scope of this paper.

The filter drain cover or impervious drain cover require careful construction.

The filter cover must function long term as a filter or the sealed cover must remain essentially impervious on a long term basis. With material likely to be dumped from substantial height up slope these filter or seal cover materials must be protected with several meters of waste rock as a protective cover.

The rock drain must be constructed in advance of the waste pile toe. The distance in advance will be a function of the height of the slope and rate of placement.

It is emphasized that a rock drain is a drainage structure and must be located, designed and constructed as such.

FILL STABILITY INFLUENCE ON ROCK DRAINS

The integrity of the rock drain must be retained during the operational life of the waste pile to carry drain water and continue to control pore water pressure buildup after abandonment. This latter requirement will be a function of the final slope angle. If the overall spoil pile angle is flattened, underdrainage pore water control is not so critical.

It is essential that the spoil pile over the rock drain retain its integrity. The spoil pile must not slide, spread or flow. If it does the rock drain will be disrupted and will not function as designed. In fact it may become a source of high water pressure and cause instability.

The allowable movements in waste piles over rock drains must be less than for spoil piles in general in order to retain the drain integrity. It is obvious that the spoil pile must be developed more carefully than usual.

Any organic or week soils must be removed from the spoil pile foundation within 25 to 100 m from the drain depending on the pile height.

In addition, clays, clay tills and clay type rock must not be dumped on the slope in the vicinity of the drain or on top of snow on the slope. The presence of such layers will likely lead to excessive movement and disruption of the drain.

Long term fill stability must also be considered. The clayey type rocks may gradually distintegrate in the spoil pile with time. As a result the effective angle of friction and safe slope angle will reduce. Hence a pile which is table today can become unstable due to strength reduction. Such materials must not be placed in the vicinity of the drain.

The use of rock drains are not recommended in areas of permafrost since they will disrupt the permafrost regime and likely lead to settlement.

DRAINAGE CONTROL AFTER MINING

The life of rock drains will likely not be permanent. They are expected to gradually deteoriate with time.

Accordingly, alternative long term surface drainage control will likely be required. The design and location of spoil piles and long term drainage should keep this ultimate reclamation requirement in mind.

A CASE EXAMPLE

In November 1968 one half of a spoil pile at Kaiser Resources failed. (Figure 2). Waste rock had been pushed over the upper steep slope onto an area including a gully with a small creek. The waste rock was generally moderate to hard limestone with shale layers. No slope or foundation preparation work was done. The limestone rock in the creek channel acted as a rock drain, at least initially.

The pile had reached about 800 feet high when the north half failed. An eye witness reported that the lower area of the slope started to move first followed by retrogressive movement up the slope.

Subsequent inspection of the failure, air photographs and test pits revealed that the lower portion of the spoil pile had encroached upon and over stressed the weak sediments of old glacial lake Fernie.

A close inspection of the failure face down the spoil pile revealed that the lower 1 to 1.5 meters of very coarse waste rock had the voids filled with fines. These had been washed down through the spoil pile by precipitation and snow melt. This confirms the need for an upper filter or seal layer.

This layer with fines creates a seepage barrier so that water pressures develop under the spoil pile. This is the reason an under filter is also recommended.

While there is no proof, it is strongly suspected that rock in the creek channel only acted as a rock drain for a limited period of time. The creek location was central in the failure area. This would result in the development of pore water pressures to reduce stability.

Several layers of compacted snow were also observed in the exposed failure face. During winter periods it is recommended that waste dumping be perpendicular to the mountain slope to minimize the development of potential failure zones.

CONCLUSIONS

The rock drain is a geotechnical structure and must be located, designed and constructed accordingly.

Adequate foundation stability is required for the rock drain and also for the spoil pile in the vicinity of the drain.

Erosion below the base of the drain must not be allowed to occur. In soil a filter underdrain is recommended.

The drain rock must be durable and meet comparable requirements for rip rap placed along rapid flowing rivers.

A rock drain cover designed as a filter or seal is recommended on top of the drain to prevent long term plugging by fines from above.

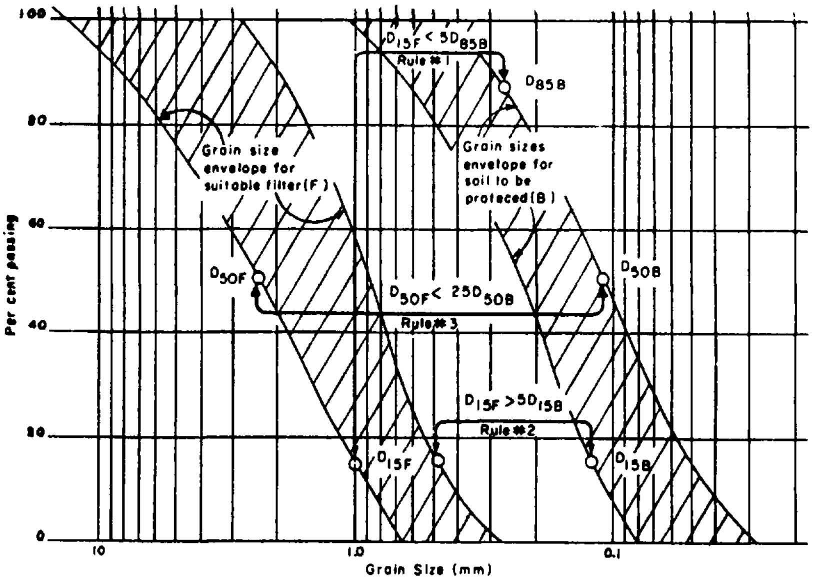
The spoil pile must remain stable in the area of the drain or it will become disrupted.

REFERENCES

Terzaghi, K. and Peck, R.B., 1967, "Soil Mechanics in Practice". John Wiley and Sons.

Morgan, R.D. and Gunnyon, G.K., 1985, "Rehabilitation of the Pier B.C. Structure at Canada Place Trade and Convention Center. Vancouver, B.C." American Concrete Institute Seminar on Concrete Rehabilitation with Shotcrete, Chicago.

Figure 1. Filter Design Criteria



Rule 1: $\dfrac{D_{15\ F}}{D_{85\ B}} < 5$

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

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

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

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

The filter material should be graded smoothly: gap graded materals 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:

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

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. (10mm) sieve.

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

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

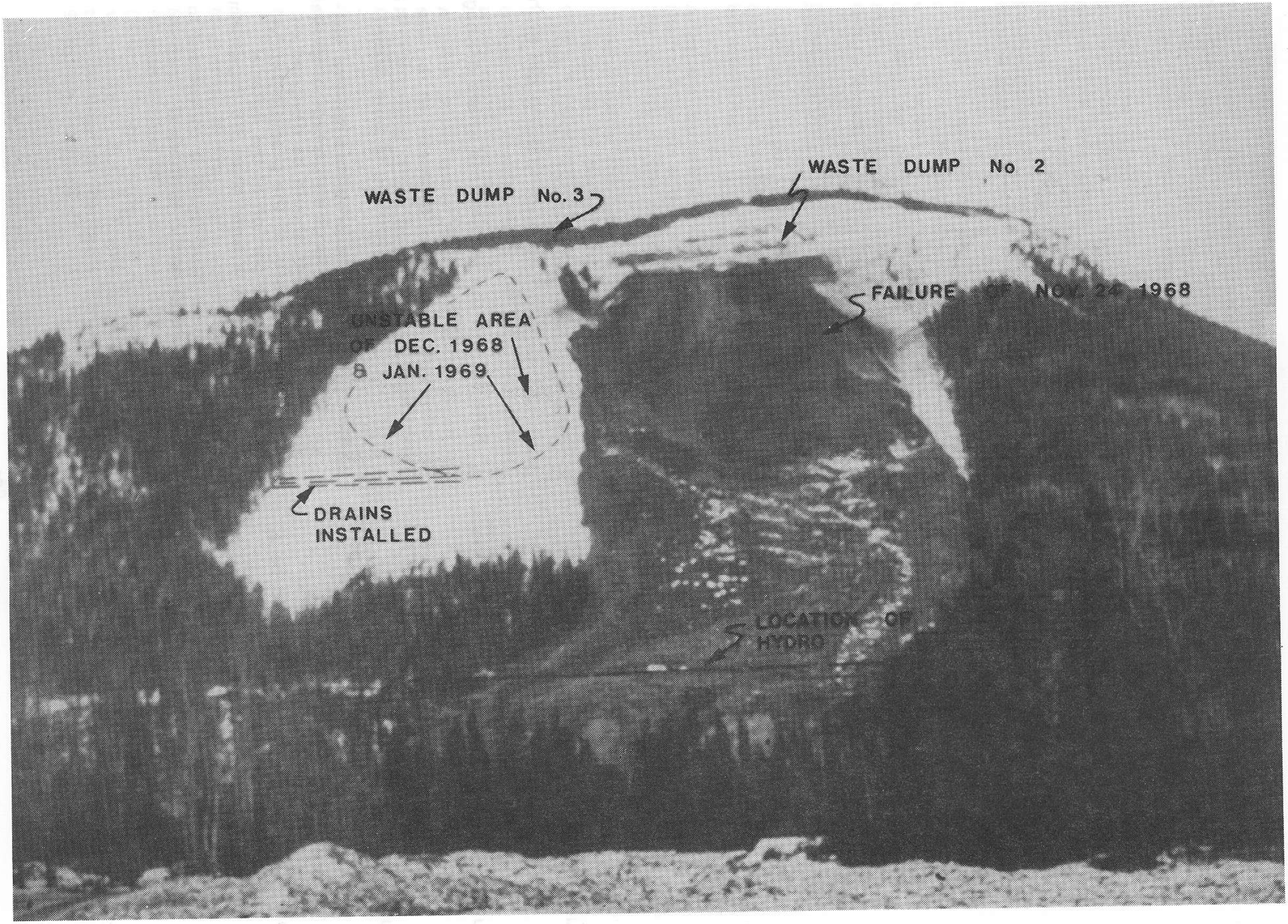


Figure 2. Waste pile that failed at Natal, B.C. in 1968. The pile was about 250 m high and was constructed over a small creek channel. No special channel drainage was developed.

EVALUATION OF DURABILITY TESTING TECHNIQUES FOR ROCK UNDERDRAIN MATERIAL USED IN APPALACHIAN SURFACE COAL MINING VALLEY FILLS

by

Robert A. Welsh, Jr.
Luis E. Vallejo
Michael K. Robinson

Office of Surface Mining Reclamation and Enforcement
Pittsburgh, Pennsylvania, 15220, U.S.A.

ABSTRACT

The integrity and efficiency of excess spoil fills in the Appalachian region of the United States is partially dependent upon placement of durable rock material in the underdrains. Currently used durability tests have limitations when determining the competence of marginally durable materials. Review of recent research on rock durability has suggested new classification systems that have not as yet been applied to surface mining excess spoil fills. A geotechnical testing program is being designed by the U.S. Office of Surface Mining Reclamation and Enforcement (OSMRE) to allow better prediction of rock durability. An array of rapid, inexpensive rock competency tests are being compared to determine which tests or combination of tests give correlative results to allow prediction of the behavior of underdrain rock in fills. The testing program is described and preliminary results of the research are discussed.

INTRODUCTION

Excess spoil consists of overburden (soil and rock excavated during the mining operation) not needed to reclaim the disturbed area to the approximate original contour. In the past, little emphasis has been placed on engineering of excess spoil structures. Often these structures were placed at locations selected strictly to optimize mining operations. Little thought was given to potential environmental consequences or safety hazards. Since the passage of the 1977 Surface Mining control and Reclamation Act, there has been an increase in the engineering effort directed toward design and construction of excess spoil disposal areas.

In general, methods of placement for excess spoil Include: (a) the lift type construction method; (b) the durable rock (gravity) fill method; (c) the zoned or barrier fill method; and, (d) the head-of-hollow fill method.

In the lift method, excess spoil is usually desposited in uniform, horizontal lifts of four feet or less and compacted to achieve the desired density. Prior to placement of the spoil in this type of fill, the foundation must be prepared and underdrains installed. According to U.S. regulations, the rock placed in the underdrain must be durable (rock that will not slake in water, nor degrade to soil material); non-acid or toxic forming; and, comprise rock which is free of coal, clay or other non-durable material.

The durable rock fill method consists of dumping spoil to its angle of repose into valleys in a single high lift or several smaller lifts. The lifts can range between 50 to over 400 feet in thickness. These lifts are then graded to develop the final fill configuration. The material forming the rock fill is generally made up of angular blast rock. The durable rock fill method can only be used if durable rock overburden is present and comprises at least 80 percent by unit volume of the fill. Durable rock has been defined as rock which does not slake in water, but the intent of the durability standard is to selectively obtain rock that can withstand blasting, handling, compaction and weathering without significant degradation. That is, a durable rock fill should behave more as a rock mass than as a soil mass. Durable rock fill material has high intergranular friction as the primary source of strength. No designed underdrain is required for this type of fill, in as much as the gravity segregation which occurs upon dumping forms a highly permeable zone of large-sized durable rock in the lower one-third of the fill.

The zoned or barrier fill method uses principles similar to embankment and dam engineering. A structural zone is constructed at the embankment face using controlled placement and compaction of spoil materials. Behind this zone, spoil is placed without the strict compaction requirements of the structural zone. Underdrains of durable rock are an integral part of this design. The zoned or barrier fill also requires a "chimney" drain between the compacted and gravity placed zones to insure free drainage. This chimney drain is similar to that placed in dam construction and requires diligent quality assurance to insure that the zoned filters are not contaminated. This practice is not widely used because of the logistics and added costs necessitated by the controls of prudent engineering practice.

A final alternative for excess spoil disposal involves the placement of spoil in lifts at the upper reaches of a watershed. This, "head-of-hollow fill" method originated in West Virginia in the early 1970s, and combines the lift-replacement technique and a chimney drain in the center of the fill. The "rock core chimney drain" results from physical segregation of larger rock during spreading of spoil material and lift compaction. All surface and subsurface drainage is to be controlled by this rock core, to prevent elevation of the phreatic surface within the fill mass. This type of fill <u>must</u> be placed where the surface drainage entering the core is minimized to prevent a decrease in permeability due to filter contamination by fine particles introduced by surface water erosion.

The successful performance of excess spoil structures is directly related to the durability of the rock forming the underdrain. If this rock degrades into soil-sized particles as a result of overburden pressures and moisture absorption, the drainage system provided by the void space between the rocks may become clogged. The clogging may cause excess pore water pressures to develop that will cause a decrease in the shear strength of the rock. This decrease in shear strength can cause the failure of the excess spoil structure. Therefore, the correct assessment of the durability of the rock is a critical underdrain design factor, on a par with proper filter design and proper sizing for anticipated flows.

OBJECTIVE

The OSMRE recognizes the need for a suitable rock durability standard. The goal is to select a rapid, inexpensive durability testing standard which will clearly differentiate between durable and non-durable materials, model surface mining conditions, and allow assurance of the long-term stability of properly designed fill structures.

Durability classification systems that involve more than two tests may be uneconomical and are subject to the accumulative effect of mechanical and human errors during testing. In addition, Franklin (1970) and Bieniawski (1974) consider the following as necessary prerequisites for any rock classification system employed on a routine basis:

1. Should be based on measureable parameters determinable by relevant tests performed quickly and inexpensively in the field;

2. Should involve only rapid testing techniques due to the potential for large numbers of routine samples;

3. Testing techniques should be simple enough to be carried out by semi-skilled field and laboratory staff; and,

4. The range of test result values should allow for a sufficient power of discrimination when applied to the various test samples.

METHODS

Geologic materials removed from their in situ environment during the surface mining of coal exhibit changes in physical integrity. Such changes are caused by physical and chemical mechanisms induced by variations in moisture and stress regimes. The rock in fills has been subjected to blasting, handling, compaction and weathering. Generally speaking, a sedimentary rock that can withstand these processes without significant changes in its original structure can be classified as a durable rock.

When selecting durable rock for fills, one should choose a single test or a combination of tests that best simulate surface mining conditions (Robinson and Ventura, 1983). For this study, recent research on rock durability has been reviewed, and testing techniques were selected for application to surface mining rock fills. In order to establish which test or combination of tests best serve as an indicator of sedimentary rock durability, a laboratory testing program was designed to simulate the moisture chenges and stress regimes that a sedimentary rock goes through during the processes of excavation and placement in excess spoil fills.

The program includes the following tests:

a. Slake Durability Testing as proposed by Chandra (1970) and Franklin end Chandra (1972) tests oven-dried samples of rock that are placed in a wire mesh drum partially immersed in water. The drum is rotated at 20 revolutions per minute for approximately ten minutes; the sample is then removed, dried, and run through a second cycle. This test includes minor abrasion effects and saturation-desiccation stresses.

b. Uniaxial Unconfined Compressive Strength Tests were run on rock cores loaded in the direction normal to bedding. Loading is applied to the point of breakage, defined as the maximum stress. A constant loading rate of 8,000 lb/min wes used. This test simulates loading stresses in a fill.

c. Atterberg Limits Testing involves measuring the liquid limit, plastic limit, and plasticity index of fine (minus #4 sieve size) materials derived from the Los Angeles abrasion test. This test is an indicator of the plasticity and the type of clay minerals in the rock.

d. Swell Testing measures the volume expansion of the cored rock normal to bedding upon immersion in water for a period of 24 hours after oven drying to 105 degrees Centigrade. Swelling strains are probably the result of expansion due to air breakage along interconnected voids such as microcracks in the rock (Olivier, 1979). Swell tests indicate the slaking stress that affects rock when it is removed from its in situ environment in the stratigraphic column.

e. Modified Los Angeles Abrasion Tests simulate the process of rock degradation during haulage and disposal of excess spoil in surfece coal mining operations. Gravity placement of excess spoil in rock fill structures subjects the rock to processes such as abrasion, impact and grinding. The Los Angeles abrasion test simulates these processes when a sample of rock is enclosed in a rotating cylindrical steel drum with steel balls. For the abrasion testing program in the present study, the steel balls were not added to the rock sample in the drum, comprising a modification of the ASTM C-131-69 (ASTM, 1978a) standard Los Angeles abrasion test. Addition of steel balls to the rock sample may make the abrasion process too rigorous to represent field conditions on a minesite. The changes in rock samples during the abrasion testing were measured after 150, 300 and 500 revolutions. The percent change in particle size distribution after the test represents the effects of abrasion, impact and grinding on the rock sample.

f. Army Corps of Engineers Accelerated Weathering Tests (cyclic wet-dry and freeze-thaw tests) model conditions that may exist at or near the surface of excess spoil structures. The behavior of spoil within this zone will most likely be controlled by alternate wetting and drying from rainfall and infiltration, followed by evaporation, and be influenced by freeze-thaw cycles resulting from diurnal changes in winter air temperatures. During these tests, sample splits of the sedimentary rocks are concurrently subjected to 80 cycles of wetting and drying and 35 cycles of freezing and thawing. The duration of individual cycles is 24 hours. The net loss of test material is measured after completion of the tests.

The above tests are being carried out in the geotechnical engineering laboratory of the U.S. Army Corps of Engineers, Ohio River Division in Mariemont, Ohio, USA. In the present discussion, results of the tests carried out on single shale and sandstone samples are reported. Thus, the conclusions derived from the testing program should be considered preliminary in nature until further analyses on varied samples are completed.

The initial samples tested in this study were collected from the recently-blasted highwalls of a surface mine in Wise County, Virginia. Grab samples of freshly-blasted rock weighing between 80-100 pounds were collected at each site. Detailed descriptions of geologic properties and mine site conditions relating to rock durability were made on-site, and photographs were taken of the sampled highwall. A simple acid test for calcareous cementing agents were performed.

One shale and one sandstone unit of the Pennsylvanian-age Wise Formation have been tested. The shale sample (VARR-1) contains silty interbeds and has mica flakes disseminated throughout the rock. This shale appears to be well-cemented and competent. Sample VARR-2 is a massive to crossbedded sandstone with randomly-disseminated coalified wood fragments and mica flakes. Although the rock is generally competent, Friable zones occur where cementation is irregular.

DURABILITY CLASSIFICATION

Based on a review of the available literature, several classification systems utilizing these tests were selected as potentially useful for the purposes of this study. The slake durability index is a single-index classification system that has been accepted by the OSMRE and thus is widely used among the coal industry for the selection of durable rock. The durability of the rock is assessed by an index, I_d, defined as the percentage retention measured by dry weight after two cycles of testing (Figure 1).

However, the use of the slake durability index as the determinant for durable rock has had several shortcomings:

1. The test fails to subject rock samples to the types of physical stresses common to surface mining conditions (impact, heavy abrasion, saturation and desiccation, compaction, etc.).

2. The index does not assess properties of rock samples indicative of rock-like or soil-like behavior.

3. Samples classified as durable in the laboratory exhibit soil-like behavior in the mining process.

Olivier (1979) relates an example of the third shortcoming of the slake durability test, encountered during testing for tunneling projects in shaley sedimentary rocks in the Republic of South Africa. After the standard two cycles of wetting and drying, silty mudstones disintegrated into small fragments, but were retained in the drums. The percentage of rock retained amounted to 95 percent of the original sample weight, thereby ranking the rock as highly durable (see Figure 1). However, the test did not take into account the high level of disintegration that took place. According to Olivier (1979), this disintegration indicates that this ample is not as durable as the slake durability test results indicate. This problem is attributed to the relatively small diameter (2mm) of the openings in the hardware cloth forming the testing drums. Such breakdown of rock in fills, particularly in the crucial underdrain areas of durable rock fills, may lead to drain blockage and consequent long-term fill failure.

Dual-index classification systems developed by Deere and Gamble (1971), Olivier (1979), and Morgenstern and Eigenbrod (1974) were also utilized to provide a basis for correlation of durability measures.

The Deere and Gamble (1971) durability classification system is based on the two-cycle slaking durability and the plasticity index of sedimentary rocks. This matrix classification system is shown as Figure 2. Gamble (1971) and Deere and Gamble (1971) found from laboratory testing that the plasticity of sedimentary rocks was directly related to their slake durability. They also found evidence of correlation between the plasticity index and the swelling potential of sedimentary rocks. Swelling is one of the important factors associated with the disintegration of sedimentary rocks. When Olivier (1979) subjected clayey sedimentary rocks to swelling tests (the samples were first oven dried and then placed under water for 24 hours), total disintegration of the samples occurred. The swelling strains recorded measured up to 5 percent swell, a high swelling value. A rock with this swelling potential and disintegration will not perform well as a rock fill material (Gamble, 1971).

Olivier (1979) recorded sedimentary rock behavior when it is removed from the in situ environment. Rocks swell and disintegrate as a result of stress relief. This swelling increases as rock absorbs moisture. Olivier (1979) also found that as swelling increases, the uniaxial compressive strength of the rock decreases. The resulting rock durability classification system involves the measurement of two rock properties. The first parameter is the magnitude of rock swelling after a dried sample is immersed in water. The second is the uniaxial compressive strength of the rock after swelling. Using these two parameters, sedimentary rock durability is assessed, using the matrix classification system shown in Figure 3.

The Morgenstern and Eigenbrod (1974) system is based on a qualitative description of the sample at its liquid limit. This classification system is used mainly to corroborate results from the other systems. Materials with a liquid limit of over 20% will "disintegrate into a granular discontinuous mass", while rock with liquid limits of under 20% will show only "slight disintegration" and opening of fissures.

RESULTS

Results from the laboratory testing program are shown in Table 1. The values obtained in the laboratory for parameters such as the swelling strain, slake durability index, unconfined compression strength, and liquid limit of the rocks, wherever applicable, where used to classify their durability. Durability classification is shown as Figures 1 through 3, using single and multiple index classification systems.

The results from the accelerated weathering tests and from the modified Los Angeles abrasion tests were used to compare durability classifications. The accelerated weathering tests (cyclic wet-dry and freeze-thaw tests) model conditions that may exist at or near the surface of excess spoil structures. The behavior of spoil within this zone will most likely be controlled by alternate wetting and drying from rainfall and infiltration, followed by evaporation, and including freeze-thaw cycles resulting from changes in air temperature.

DISCUSSION

Deere and Gamble's (1971) durability classification system makes use of the slake durability test. Thus, from our previous discussion, its validity to select durable rocks remains questionable.

All the classification systems correlated well in classifying the sandstone as a rock with high durability. For the shale durability, however, the different classification systems did not agree. The Franklin and Chandra (1972) slake durability index and the Deere and Gamble (1971) classification system classed the rock as of very high durability. The Olivier (1979) and Morgenstern and Eigenbrod (1974) classification systems graded the shale as either moderately poor or poor with respect to durability. These durability rankings are substantiated by the accelerated weathering test result (Table 2). The accelerated weathering tests show that the shale had very high levels of disintegration after 35 cycles of wetting and drying. However, the samples exhibited high levels of resistance against abrasion in the modified LA abrasion test. Results from these modified abrasion tests show very low percentages of loss.

The Franklin and Chandra (1972) and the Deere and Gamble (1971) classification systems use slake durability test results as the basis for a rock durability classification. Olivier (1979) as well as Robinson (1983) have expressed concern about the validity of the slake durability test as a measure of durability of sedimentary rocks. Primary reasons for this concern are the small size of the openings (2mm) in the sieve (Hardware cloth) that forms the drums, the small number of cycles in the test (2 cycles), and the failure of the test to subject rock samples to stresses common to surface mining conditions.

The possibility exists that additional cycles of slake durability tests would cause the material to further disintegrate and pass the sieves. The result would be a downgrading of the rock with respect to durability. Thus, the sieve openings and number of the test cycles in the slake durability test seem to have marked influence on durability classification systems for sedimentary rocks (Olivier, 1979; Robinson, 1983). These two factors are reflected in the lack of agreement with respect to durability of the shale sample when comparing the slake durability and other tests.

CONCLUSIONS

A preliminary evaluation of the classification systems that use one or two indices for selection of durable rock for use in excess spoil fill underdrains has been presented. A silty shale and sandstone were tested; from the laboratory and evaluation program the following was found:

TABLE I.

LABORATORY RESULTS

	Silty Shale	Sandstone
Unit Weight	138.21 pcf (21.7 kN/m^3)	160.6 pcf (25.23 kN/m^3)
In situ water content	2.0%	2.1%
Longitudinal Swelling Strain E_D	0.0085	0
Slake Durability Index, I_D	98.5%	99.4%
Unconfined Compressive Strength	11,880 lb/in^2 (81.9 MPa)	14,280 lb/in^2 (98.5 MPa)
Atterberg Limits:		
Liquid Limit	30%	N/A
Plastic Limit	21%	N/A
Plasticity Index	9%	N/A
Modified Los Angeles Abrasion Percentage lost after:		
150 cycles	6.3%	5.7%
300 cycles	9.5%	7.3%
500 cycles	11.7%	7.7%
Accelerated Weathering*		
Percentage lost after 80 cycles of wetting and drying	62.50%	0.51%
Percentage lost after 35 cycles of freezing and thawing	54.0%	13.8%

*One cycle has a duration of 24 hours

TABLE II.

ROCK DURABILITY CLASSIFICATION

	Silty Shale	Sandstone
Single Index Classification Systems		
The Franklin and Chandra (1972) Slake Durability Classification System (Fig. 1)	Very highly durable	Very highly durable
The Morgenstern and Eigenbrod (1974) Classification System (based on liquid limit of 20%)	*Material Disintegrates into granular discontinuous mass	N/A
Multiple Index Classification Systems		
The Deere and Gamble (1971) Classification (Fig. 1)	Very highly durable	Very highly durable
The Olivier (1979) Classification (Fig. 3)	*Moderately to poorly durable	Excellent durability

1. For the sandstone, single index and two indices classification systems correlated well in the rock durability evaluation. The classification systems used were those developed by Franklin and Chandra (1972), Deere and Gamble (1971) and Olivier (1979). These classification systems were supported by accelerated weathering tests (cyclic freeze-thaw and wet-dry).

2. For the shale, discrepancies in durability determinations between the different classification systems were encountered. Classification systems using the slake durability test as a basis [Franklin and Chandra (1972); Deere and Gamble (1971)], ranked shale as very highly durable. Classification systems using parameters other than the slake durability test for the durability test [Morgenstern and Eigenbrod (1974); Olivier, (1979)], ranked the shale as a rock with poor durability. The poor durability of the shale was substantiated from the accelerated weathering test results.

3. If the slake durability test is used as a basis for the selection of durable rock, the effect of parameters such as the size of the openings in the sieves (hardware cloth) and the limited number of cycles used are valid areas for further study. From the preliminary results presented, these two parameters have a marked influence in classification systems using the slake durability test as the cornerstone for durable rock classifications.

FURTHER STUDY

Additional testing will be performed to assess the reproductivity of results utilizing a two-indices testing protocol (swell and compressive strength). Modifications of the slake durability test will also be assessed.

REFERENCES

American Society for Testing and Materials (1978). "Part 14, Concrete and Mineral Aggregates," 1978 Annual Book of ASTM Standards, ASTM, Philadelphia, 814 pp.

Andrews, D.E., Withiam, J.L., Perry, E.F., and Crouse, H.L. (1980). "Environmental Effects of Slaking of Surface Mine Spoils: Eastern and Central United States." Bureau of Mines, U.S. Department of the Interior, Denver, Colorado, Final Report, 247 pp.

Bieniawski, Z.T. (1974). "Geomechanics Classification of Rock Masses and Its Application in Tunneling." Proceedings of the 3rd International Congress on Rock Mechanics, Denver, Colorado, Vol. II, PP. 27-32.

Chandra, R. (1970). "Slake Durability Tests for Rocks." Master's Thesis, Imperial College, London University, London, 55 pp.

Deere, D.U. and Gamble, J.C. (1971). "Durability-Plasticity Classification of Shales and Indurated Clay." Proceedings for the 22nd Annual Highway Geological Symposium, Norman, Oklahoma, pp. 37-52.

Deo, P. (1972). "Shales as Embankment Materials." Final Report, JHRP-72-45, Indiana State Highway Commission, 202 pp.

Eingenbrod, K.D. (1972). "Progressive Failure in Overconsolidated Clays and Mudstones." Ph.D. Thesis, University of Alberta, Canada, 245 pp.

Franklin, J.A. (1970). "Observations and Tests for Engineering Description and Mapping of Rocks." Proceedings of 2nd International Congress on Rock Mechanics, Belgrade, Vol. 1, pp. 1-3.

Franklin, J.A. and Chandra, R. (1972). "The Slake Durability Test." International Journal of Rock Mechanics and Mining Science, Vol. 9, pp. 325-341.

Gamble, J.C. (1971). "Durability-Plasticity Classification of Shales and other Argillaceous Rocks." Ph.D. Thesis, University of Illinois, 161 pp.

Morgenstern, N.R. and Eigenbrod, K.D. (1974). "Classification of Argillaceous Soils and Rocks." Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, pp. 1137-1155.

Olivier, H.J. (1979). "A New Engineering-Geological Rock Durability Classification." Engineering Geology, Vol. 14, pp. 255-279.

Reidenouer, D.R., Geiger, E.G. and Howe, R.H. (1976). "Suitability of Shale as a Construction Material." American Society for Testing and Materials, STP 597, pp. 59-75.

Robinson, M.K. and Ventura, J.D. (1983). "Disposal of Excess Spoil: Durable Rock Fills." Symposium on Surface Mining, Hydrology, Sedimentology and Reclamation, Lexington, Kentucky, pp. 179-187.

Strohm, W.E., Bragg, G.H. and Ziegler, T.W. (1978). "Design and Construction of Compacted Shale Embankments," Vol. 5, Federal Highway Administration, Office of Research and Development, 207 pp.

United States Code of Federal Regulations, Title 30, Mineral Resources, Chapter VII, Enforcement, Department of the Interior. Section 817.73, p. 378, 1983.

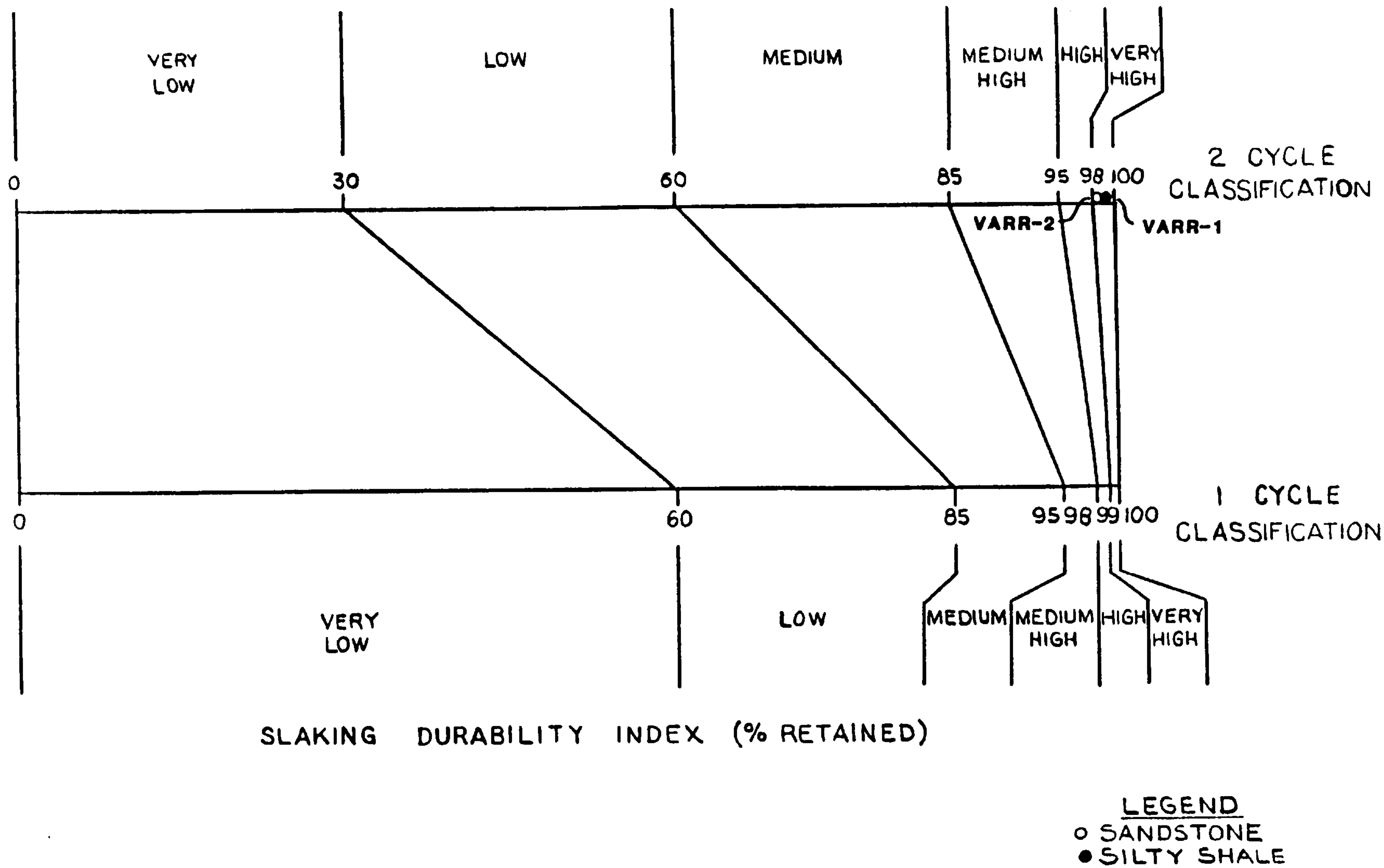


Figure 1. Franklin and Chandra (1972) system classification of preliminary samples.

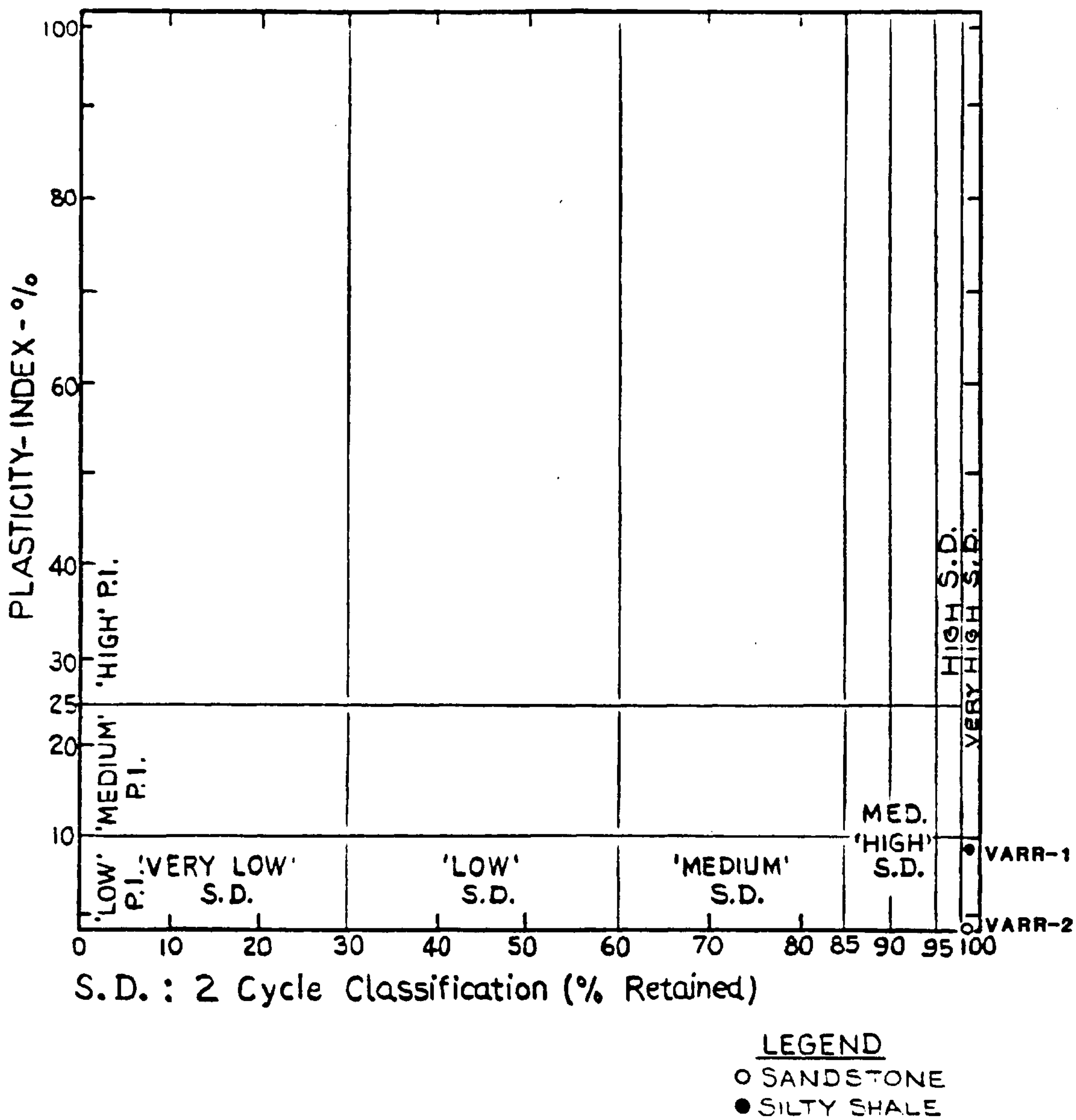


Figure 2. Deere and Gamble (1971) system classification of samples.

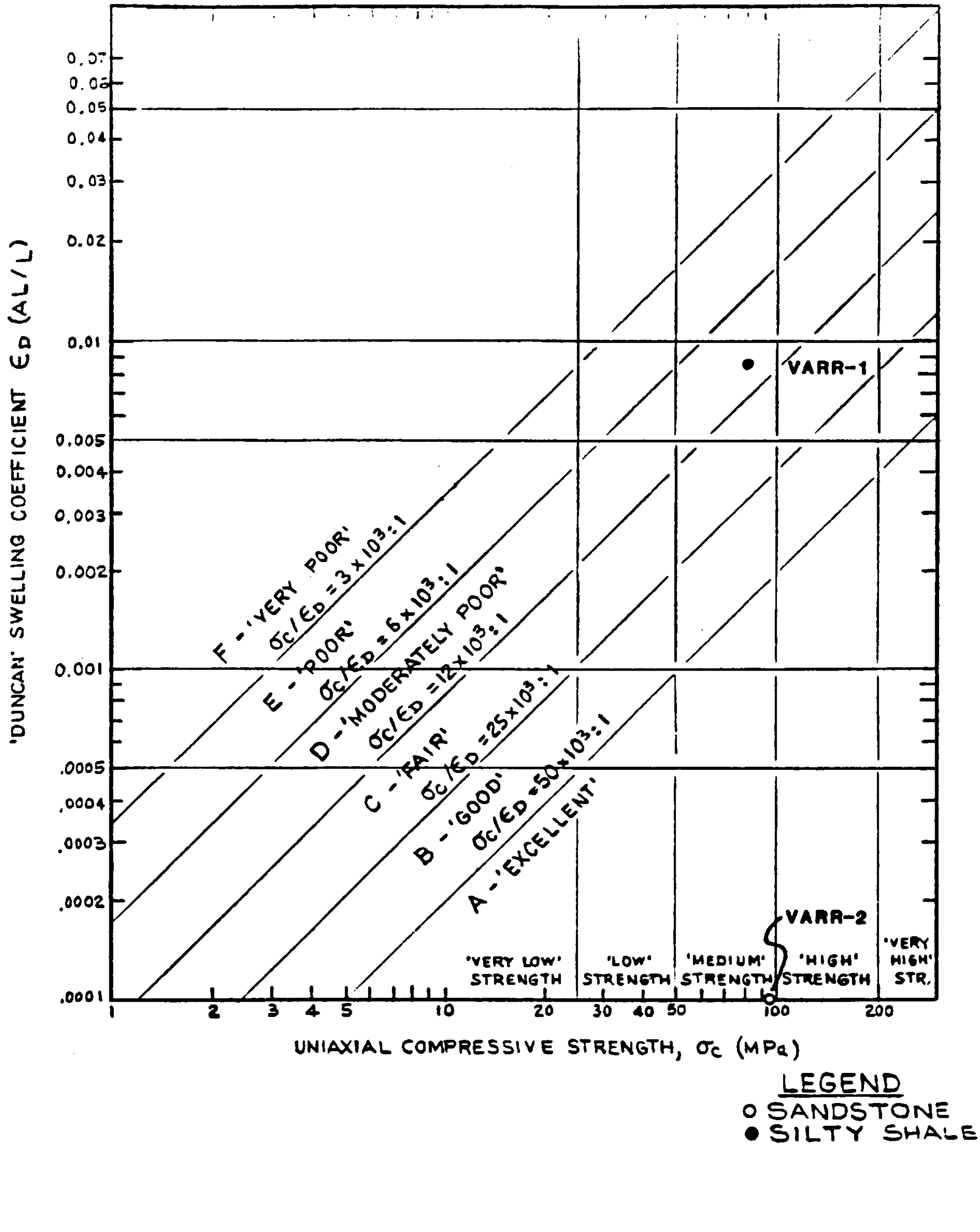


Figure 3. Olivier (1979) system classification of samples.

DISCUSSIONS OF CONCERNS REGARDING THE LONG-TERM PERFORMANCE OF ROCK DRAINS

by

David B. Campbell, P.Eng.
Principal, Golder Associates
Consulting Geotechnical Engineers
Vancouver, British Columbia

ABSTRACT

Employees of regulatory agencies responsible for assessing permit applications for development or rock drain projects have postulated a number of scenarios that they thought could be responsible for impairment of the long term performance of rock drains. These scenarios include deposition of bedload and suspended sediment within the drain, downward migration of fines from the upper to the lower region of the dump, and potential scour of the natural stream bed at the base of the rock drain. These scenarios are discussed, and are shown not to be significant relative to the expected long-term performance of a rock drain.

INTRODUCTION

Rock drains are currently being used in British Columbia to conduct creek flows beneath waste rock dumps. The function of these rock drains is similar to that of culverts. The rock drain is a relatively new and innovative concept, the first installation having been developed at Fording Coal's Fording River open pit coal mine during the winter of 1981-82. This rock drain conducts the surface flows in Swift Creek beneath a rockfill corridor that provides truck access from the southern Greenhills mining area to a waste dump located on the opposite side of the drainage course.

The rock drain concept has wide application at operating mines in mountain regions where terrain is steep, and suitabie locations for dump development are limited. Rock drains are now in use or are being proposed at all of the operation open pit coal mines in the eastern cordillera region of British Columbia.

Since the rock drain concept is new, and no precedent operating data have previously been available, members of regulatory agencies responsible for the assessment of permit applications have held reservations regarding the long-term performance of rock drains, and have postulated scenarios which they considered might result in gradual reduction of the through-flow capacity of a rock drain over time. The scenarios most commonly raised include:

1. Deposition of both bedload and suspended sediment within the rock drain with attendant reduction in through-flow capacity,

2. Downward migration of particles within the dump, and deposition of these fine particles within the rock drain resulting in reduction of through-flow capacity over time,

3. Degradation of the constituent fragments comprising the drain with attendant reduction in through-flow capacity,

4. Erosion and scour at the base of the drain.

This paper discusses the major concerns that have been raised, and presents data which show that provided reasonable precautions are exercized during development of a rock drain, the scenarios that have been postulated are not a problem, and should not be cause for concern regarding long term performance. This conclusion is based on the results of field observations, the interpretation of data collected at the Swift Creek rock drain which has now been in operation for a period of five years, the results of both field and laboratory modelling studies, and simple numerical analyses.

DEPOSITION OF SEDIMENT

Suspended Sediment

The potential for deposition of suspended sediment within a rock drain can be assessed by examining the field data that have been recorded at Fording Coal's Swift Creek rock drain. Before the rockfill was advanced across the Swift Creek drainage course, six piezometers were installed along the bottom of the stream channel to permit measurement of the elevation of the freewater surface within the rock drain. Detailed surveys were made to establish the configuration of the channel cross-section at each of the piezometers. The piezometer data, together with the detailed survey data establish the gross area of the wetted cross-section corresponding to measured rates of discharge. Plots of discharge versus the corresponding areas of the wetted cross section within the rock drain can be represented by a straight line, the slope of which is a measure of the through-flow capacity of the drain per square meter of wetted cross-section at the location of a specific piezometer. A typical plot is shown on Figure 1.

At the Swift Creek rock drain, the lowest measured rate of through-flow is approximately 0.04 $m^3/sec./m^2$ of wetted cross-section, at a location where the rock drain is covered by approximately 55 m of fill. Although it is not possible to make direct measurements of void ratio within the rock drain beneath the 55 m of overlying rock fill, it is reasonable to assume that the void ratio within the rock drain is approximately 0.5, and that the corresponding average velocity of flow through the void spaces between the rock fragments comprising the drain is approximately 0.12 m/sec.

Figure 2 is a chart showing the relationship between velocity of flow and scour-size particle, i.e. the size of particle which if located at the flow boundary, is subject to incipient scour. For an average flow velocity of 0.12 m/sec., the effective diameter of the scour-size particle is 0.09 mm. This size of particle, and all smaller-sized particles which if already in suspension, could be expected to remain in suspension within the turbulent flow field within the rock drain, and pass through the drain. Thus, only those particles larger than the scour-size particle could be expected to settle, and to be deposited within the rock drain.

The resistance to flow through a rock drain is greater than the resistance to flow within the open channel upstream of the inlet to the rock drain. Consequently, during periods of high stream discharge, a pool can be expected to develop at the inlet to a rock drain. This pool acts as a sedimentation trap in which both bedload and suspended solids are deposited. The pool therefore provides a degree of protection against entry of sediment into the rock drain. Photograph No. 1 shows the pool upstream of the inlet end of the Swift Creek rock drain. At the entrance to the drain, this pool has a maximum depth of approximately 1.5 m. The scour-size particle, which has an effective diameter of 0.09 mm would settle to the bottom of this 1.5 m deep pool in approximately 3.4 minutes. The retention time in the pool is also approximately 3.4 minutes. At any time when the retention time in the pool at the inlet to a rock drain is equal to or greater than the settling time for the scour-size particle, only those particles smaller than the scour-size particle will remain in suspension to enter the drain. Under these circumstances, suspended sediment entering the drain can be expected to remain in suspension within the turbulent flow field, and to be swept through the drain.

The pool that develops at the inlet end of a rock drain serves as a sedimentation trap that provides protection against entry of suspended sediment that could settle, and result in clogging of the drain over the long term.

Bedload

Photograph No. 2 is a view looking in the downstream direction toward the inlet end of the Swift Creek rock drain. This photograph was taken in late fall, when the flow in Swift Creek had dropped to a low value. The gravel-size material on either side of the flow channel represents bedload that was deposited during periods of higher flow, when the pool was extant upstream of the inlet to the drain. It is evident in Photograph No. 2 that the low flows have reworked some of the bedload sediments deposited in the pool, and that some of these sediments have been transported toward the inlet end of the drain.

Inspection of Photograph No. 2 also shows that pervious rock extends up the upstream face of the causeway fill to a level at least as high as the upper limit of the photograph. With time, continued deposition of sediment may result in a gradual raising of the approach channel to the rock drain. As the base of the approach channel rises progressively, the width of the line of intersection between the surface of the sediment and the upstream face of the rockfill increases progressively. Consequently, deposition of sediment will not result in reduction of the area available for entry of water into the drain.

If progressive accumulation of sediment within the approach channel leading to the inlet end of the rock drain occurs, the resulting conditions would be analagous to the Laughing Jack Marsh Dam described by Wilkins (1956). With sediment deposited adjacent to the toe of the upstream fill slope, part of the stream discharge would enter the drain via seepage through these sediments, and the remainder of the flow would enter the rockfill at the level where the surface of the fluvial sediments intersect the upstream face of the causeway. From this point, the flow within the rockfill would be steeply downward toward the base region of the rock drain beneath the upstream fill slope as illustrated on Figure 3. The flow would then continue through the rock drain in the manner that has prevailed to date.

The data that have been obtained at the Swift Creek rock drain show that flow velocities increase exponentially in the downstream direction, and that the most significant increase in velocity occurs within the downstream half of the drain. This is illustrated in Figure 5. A similar increase in velocity proceeding in the downstream direction, particularly beneath the downstream face of the fill is to be expected for any rock drain. The distance that a given size of particle might be transported into the rock drain is not known. However, since the velocities increase exponentionally in the downstream direction, it is evident that any particle which might be transported as far as the midpoint of the drain would not remain permanently at that location, but would be transported onward through the drain.

The foregoing discussion leads to the conclusion that neither deposition of suspended solids, nor of bedload should be cause for concern regarding the long-term performance of a rock drain. This conclusion is supported by the data that have been collected at the Swift Creek rock drain since its initial operation during the Spring runoff of 1982. The data show no indication of a reduction in the through-flow capacity over time.

DOWNWARD MIGRATION OF PARTICLES

Inspection on the surface of a dump platform shows typically that material underfoot contains a large percentage of small-sized rock fragments. By comparing the size of these small particles on the surface of the dump platform with the size of the large rock fragments that separate on the face of the dump and accumulate at the dump toe, the casual observer is likely to conclude that downward migration of fine rock fragments from the upper region to the lower region of a dump could result in potential clogging of the void spaces within a rock drain. The evidence presented in the following paragraphs shows that this is not the case. This evidence includes detailed inspection of the face of a 200 m high dump, the results of grain size analyses from field dumping trials, and grain size analyses of material comprising a model dump developed in the laboratory.

Particle Size Distribution on Dump Face

Photo No. 3 comprises a series of photographs that illustrate the size of rock fragments on the face of a 200 m high waste rock dump at the Line Creek Mine operated by Crows Nest Resources Ltd. In each of the photos, the distance between the camera and the mean surface of the dump face was 3.0 m. The series of photographs shows relative rock sizes proceeding from the toe toward the crest of the dump. The actual rock sizes are indicated by the scale.

Examination of the rock sizes at each of the levels as identified on Photo No. 3 indicates that the particles are sufficiently large that they could not be expected to pass through the inter-particle void spaces at the next lower level. The series of photographs indicate that the particle gradation on the dump face, which is also indicative of the size gradation within a vertical column extending through the dump, forms a well-graded filter which precludes downward migration of particles.

Laboratory Model

Since recovery of the representative samples from the face of a 200-metre high dump is impracticable, a model dump was developed in the laboratory using procedures that simulate development of the full size dump. The laboratory model dump had a total height of 600 mm, and the material used in its construction consisted of well graded, 10 mm minus crushed rock. The ratio of maximum particle size to dump height for the laboratory model is similar to the particle-height ratio for a 60 to 70 m high waste rock dump.

The laboratory model was developed by depositing the material at the dump crest, and permitting the material to roll and slide down the dump face which remained at the angle of repose. The model dump was advanced through the process of gradual accretion of material on the face, similar to the manner in which a full size waste rock dump develops. Lateral confinement for the model dump was provided by vertical glass panels spaced 100 mm apart. The transparent sides permitted visual inspection of sorting and stratification within the body of the model dump, and the change in gradation between the dump platform and the base. Photo No. 4 shows a side view of the model, illustrating the stratification parallel to the dump faces during intermediate stages of development, and the general increase in particle size form the top to the base of the model dump. Inclined stratification similar to that illustrated in Photo 4 results from shallow-seated sheet ravelling on dump faces, and similar inclined stratification is clearly evident on the faces of cut slopes made in full scale waste rock dumps.

After the face of the model dump had been advanced a distance of approximately 800 mm, samples were recovered from a 100 m square vertical column extending from the top to the base of the model. The vertical column comprised six samples, each consisting of 1 cubic decimeter of material. Grain size analyses were carried out on each of these six segments of the vertical column and the resulting grain size curves are shown on Figure 4.

Comparison of the grain size curve for any segment of the vertical column with the grain size curve of the next lower segment shows that downward migration of particles within the column could not occur. Considering the well-established filter design criteria to preclude particle migration, the D_{15} size of any segment could be 4 or 5 times larger than the D_{85} size of the next higher segment in the column. The grain size curves on Figure 4 show that the D_{15} size of the adjacent lower material is smaller than the D_{85} size of the material in any particular segment. In fact, the D_{15} sizes are between 2 and 5 percent of the maximum size that would normally be acceptable in filter design.

The grain size curves on Figure 4 show that downward particle migration from the upper to the lower part of the model dump is precluded. Examination of the grain size curves for samples recovered from the field dumping trials reported by Nichols (1986) shows similar results. That is, Nichols' curves also show that downward particle migration within the dump is precluded.

The rate at which precipitation of melt water can enter a dump is governed by the permeability of the material on the dump surface. The series of photos, Photo 3, as well as the grain size curves on Figure 4 show that permeability increases progressively proceeding from the surface toward the base of a dump. Since the rate at which water can enter the dump is governed by the permeability of the dump surface, and permeability within the dump increases with depth, downward percolation of water within the body of the dump occurs under conditions of non-saturated flow. For non-saturated flow, the water is not subject to any pressure field, and downward percolation occurs under the influence of gravity alone. For these conditions of non-saturated flow, the downward percolating water is at sub-atmospheric pressure and subject to capillary tension. These tension forces impart compression forces across the contacts between adjacent fine-grained particles. These inter-particle compression forces, which can be significant relative to the gravitational forces that act on fine-grained particles further tend to preclude downward movement of fine soil particles within the dump.

Both field and laboratory data show that downward migration of particles from the upper to the lower regions of the dump is unlikely to occur, and that potential clogging of a rock drain as a result of downward migration of particles from the upper to the lower regions of a dump is not a factor that could be expected to impair the long term performance of a rock drain.

As part of the continued open pit mining operations at Fording Coal, the No. 1 waste rock dump that was developed in approximately 1972, and has been in place for a period of 14 years, is being excavated. The current pit wall exposes segments of the base of this dump, together with the underlying in situ foundation. Examination of the exposures that have developed in the course of mining through the No. 1 spoil shows that the base of the dump consists of large rock fragments, together with smaller angular fragments which were probably produced by inpact at the time of dump development, as well as by fracturing due to high point-to-point contact stresses resulting from the weight of the rock fill above the base. The waste rock exposed at the base of the No. 1 spoil is not choked by fines, and provides further evidence that downward migration of fines within the body of a dump does not occur.

BASE SCOUR

It has been suggested that during periods of high discharge, the flow velocities could result in serious scour at the base of a rock drain. Mathematical analyses as well as the field data from the Swift Creek rock drain show that this is not the case.

Equation 1 is Manning's formula which expresses the velocity of flow in an open channel.

$$V = \frac{1.486}{n} R^{0.67} S^{0.5} \qquad \text{(Equation 1)}$$

Where: V = Velocity
R = Hydraulic radius

The term 'n' in the denominator is a channel roughness factor, and 'S' is the slope of the channel. For a given segment of stream channel, the first and third terms in Equation 1 are both constants, leading to the conclusion that the velocity of flow in an open channel is governed by hydraulic radius, which is gross wetted cross-sections divided by wetted surface. As flow increases in an open channel, wetted area increases more rapidly than does wetted surface, with the result that hydraulic radius increases. Therefore, as flow increases, the velocity of flow in the open channel also increases. The fact that velocity of flow increases with increasing discharge is evident to even the most casual observer.

The metric version of Wilkin's equation (Wilkins 1956) which expresses the average flow velocity for turbulent discharge through coarse angular rock is given by Equation 2.

$$Vv = 5.28 \left(\frac{eD}{7.5} \right)^{0.5} S^{0.54} \qquad \text{(Equation 2)}$$

Where: V_v = Average velocity of flow through the voids (m/sec)
e = Void ratio
D = Stone size (m)

The term 'S' in the equation represents slope. Provided the depth of the pool at the inlet end of a rock drain is small compared to the drop in elevation along the drainage course between the inlet and the outlet of the drain, the value of 'S', i.e. the third term in Equation 2 is independent of the rate of discharge, and remains constant. The term inside the brackets represents hydraulic radius. Provided 'S' remains constant, Equatlon (2) shows that velocity is governed by hydraulic radius as is the case for open channel flow. However, it is clear that an increase in stream discharge does not change the void ratio 'e', nor the stone size 'D', the two variables that govern hydraulic radius within a rock drain. As discharge through a rock drain increases, hydraulic radius remains constant. This simple analysis shows that for turbulent flow, the average velocity of flow through the void spaces within a rock drain remains essentially constant, and independent of rate of discharge.

The foregoing conclusion is confirmed by the field data obtained at the Swift Creek rock drain. Figure 1 which is plot of measured stream discharge versus the area of the wetted cross-section at a position beneath the crest of the causeway fill, shows that over the range of discharge 0.0 to 0.77 cumecs, the velocity of flow through the rock drain remained constant. This is the result that should be expected for turbulent flow through a porous medium.

Referring to Figure 1, the average rate of discharge through the rock drain was 0.05 m^3/sec/meter^2 of gross wetted cross section. It is reasonable to assume that the void ratio within the rock drain is approximately 0.5. Consequently, the average velocity of flow through the void spaces within this segment of the rock drain was approximately 0.15 m/per sec. This average velocity of flow through the void spaces is approximately one tenth of the measured flow velocity in the open channel.

At the time the rock drain was constructed, the Swift Creek channel had developed a stable regime. Thet is, the channel was neither actively aggrading nor degrading, and the natural channel bottom consisted of a lining of cobbles and gravel. Since the mean velocity of flow through the rock drain is one tenth or less of the velocities of flow that prevailed in the open channel during periods of Spring runoff, it is unreasonable to conclude that base scour is a problem when considering the flow through the Swift Creek or similar rock drains.

PARTICLE DEGRADATION

If degradation of the constituent particles comprising the rock drain were to occur, the through-flow capacity of the drain could be impaired. For this reason, it is essential that the type of rock used to develop a rock drain be chemically inert.

Successive cycles of wetting and drying can result in degradation of some rock types, particularly shales, and to a lesser degree siltstones. However, it is unlikely that wetting and drying is a significant factor that results in degradation of rock fragment within a dump, or a rock drain. Indirect evidence is provided by Brownell (1984) in a description of the living environment of the sand scorpion.

Brownell notes that the temperature at the surface of sand dunes in the Mojave Desert often exceeds 70° C, but that at a depth of approximately 10 cm below surface, the temperature drops to 40° C and relative humidity is greater than 90 percent. The fact that the relative humidity at this shallow depth below the surface of hot desert sand is greater than 90 percent, suggests that within a waste rock dump, where surface temperature conditions are much less severe, the relative humidity of the air occupying the void spaces between rock fragments is likely to be greater than 90 percent, and may even approach 100 percent. For these conditions, it is reasonable to expect that wetting and drying is not a significant factor that would contribute to degradation of rock fragments over the length of a rock drain.

CONCLUSIONS

The temporary pool that develops at the inlet to a rock drain during periods of high stream discherge serves as a settling pond in which both bedload and suspended solids are deposited. This temporary pool provides protection against entry of sediment that could settle, and result in clogging of a rock drain over the long term.

During periods of low flow, sediments deposited in the pool may be reworked, and transported toward the inlet to the drain. However, the rate at which water could enter the drain can be expected to be virtually unimpaired.

The velocity of flow through a rock drain can ba expected to increase exponentially in the downstream direction within that segment of the drain located beneath the downstream fill slope. Thus a particle that might be transported within the drain to a position beneath the downstream crest could be expected to be transported on through the drain. Consequently, deposition of sediment and reduction of through-flow capacity within the downstream portion of a rock drain should not be expected to occur.

Particle size segregation on a dump face produces a well-graded filter that preludes downward migration of particles within a dump. Consequently, downward migration of particles from the upper to the lower regions of a dump is not a factor that would result in clogging, and reduction of the through-flow capacity of a rock drain over time.

The velocities of flow through the void spaces within a rock drain can be expected to range from 0.1 to 0.4 metres per second. For this range of flow velocity, base scour is not a potential problem provided the natural soils on which the rock drain is developed consist of cearse sand or larger sized particles. The natural beds of mountain streams normally consist of gravels and cobbles, so that base scour is not a problem.

REFERENCES

Brownell, Philip H., "Prey Detection of the Sand Scorpion," Scientific American, December, 1984.

Nichols, R.S., "Rock Segregation in Waste Dumps," International Symposium on Flow-Through Rock Drains, Cranbrook, British Columbia, Canada, September, 1986.

Wilkins, J.K., "Flow of Water Through Rockfill and its Application to the Design of Dams." Proceedings, Second Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, 1956.

PHOTO NO. 1

A view from the crest of the causeway across the Swift Creek drainage course, showing the pool upstream of the inlet to the rock drain. This pool serves as a sedimentation trap that collects bed-load as well as suspended sediment.

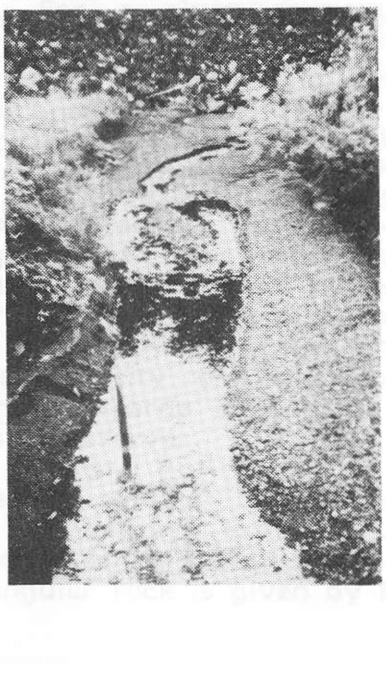
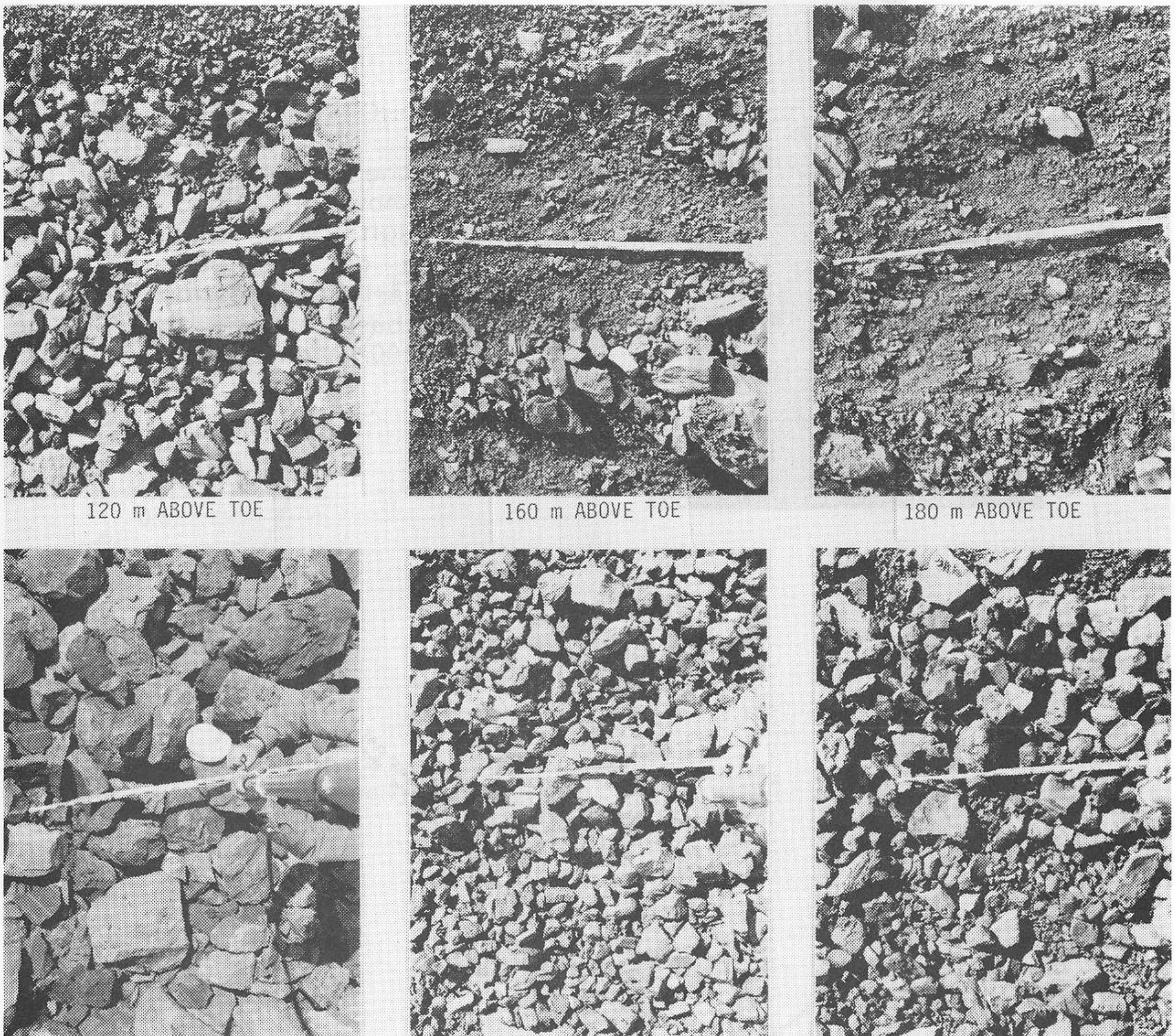


PHOTO NO. 2

Looking toward the inlet end of the Swift Creek rock drain, illustrating the sediment that was deposited in the pool during periods of higher discharge. During low flow, some of the trapped sediments have been reworked, and have been transported toward the inlet to the rock drain. The rock on the upstream face of the causeway fill between the surface of the sediments and the upper limit of the photo is pervious, and will conduct the flows to the rock drain if the level of the upper surface of the sediments continues to rise over time.



120 m ABOVE TOE

160 m ABOVE TOE

180 m ABOVE TOE

TOE OF DRAIN

CENTRE OF DRAIN

PHOTO NO. 3

A series of photographs at selected intervals of height above the toe of Crows Nest Resources' West Line Creek waste rock dump. The photos illustrate the reduction in the size of the rock fragments proceeding from the dump toe toward the dump crest. All photographs were taken looking normal to the surface of the waste rock, with the camera at a distance 3.0 meters from the surface. The sizes of individual rock fragments are indicated by the scale.

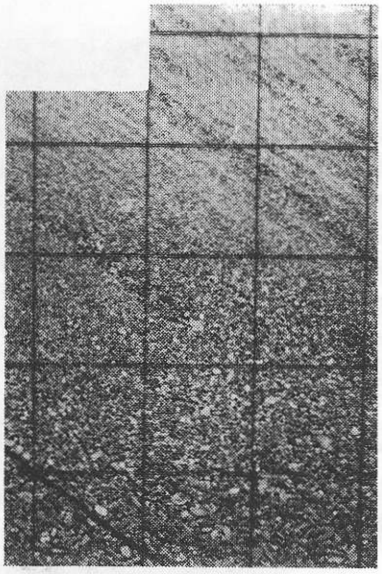


PHOTO NO. 4

A view of the cross section of the model dump showing inclined stratification that results from shallow seated sheet sliding on the face, and the increase in particle gradation from the top to the base of the dump. The squares are 100mm dimension.

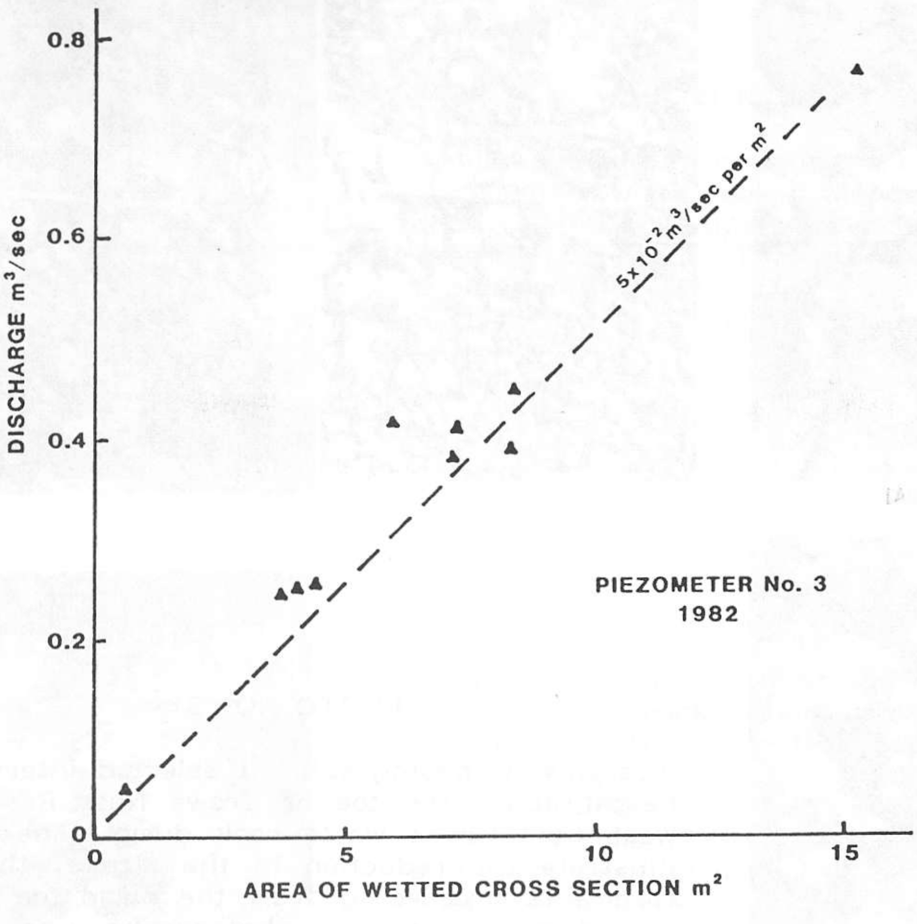


FIG 1

A typical plot of rate of discharge through the Swift Creek Rock drain vs the area of the wetted cross section. The slope of the best fit straight line through the data points is a measure of the discharge capacity per unit of cross sectional area.

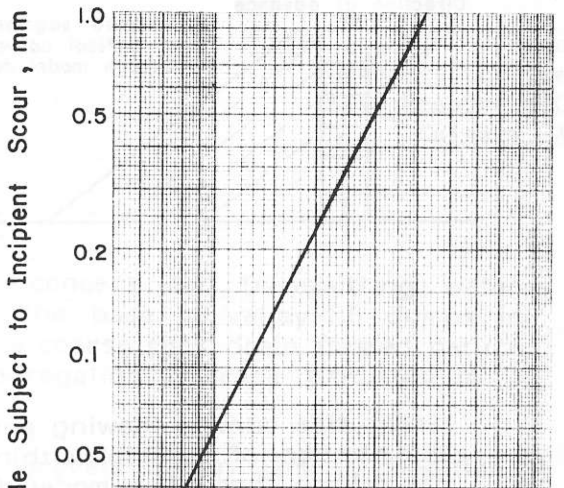


FIG 2

A graphical presentation showing the relationship between flow velocity and the size of particles subject to incipient scour.

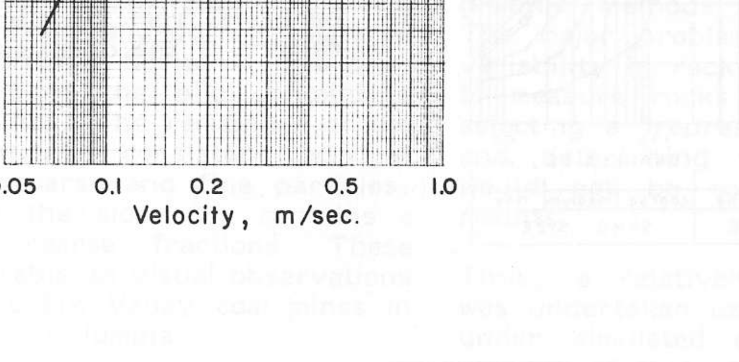
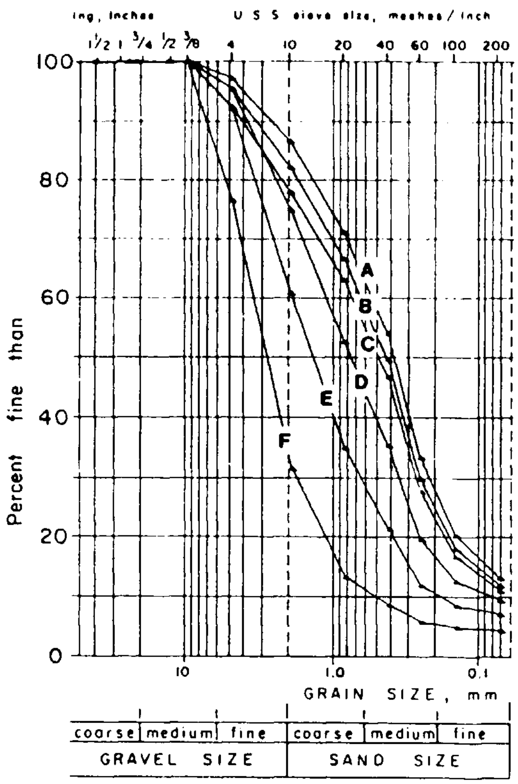


FIG 3

Illustrating the manner in which water can be expected to enter a rock drain after fluvial sediments have been deposited adjacent to the upstream toe of the rock fill.



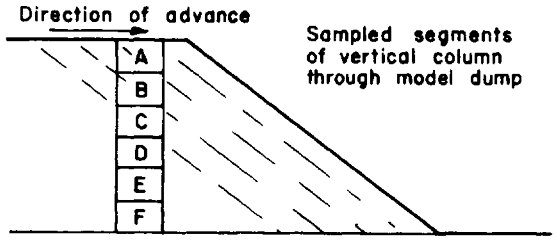


FIG 4

Grain size curves showing particle size distribution of material within a vertical column through a model dump, and illustrating that downward migration of particles from the upper to the lower regions of the dump cannot occur.

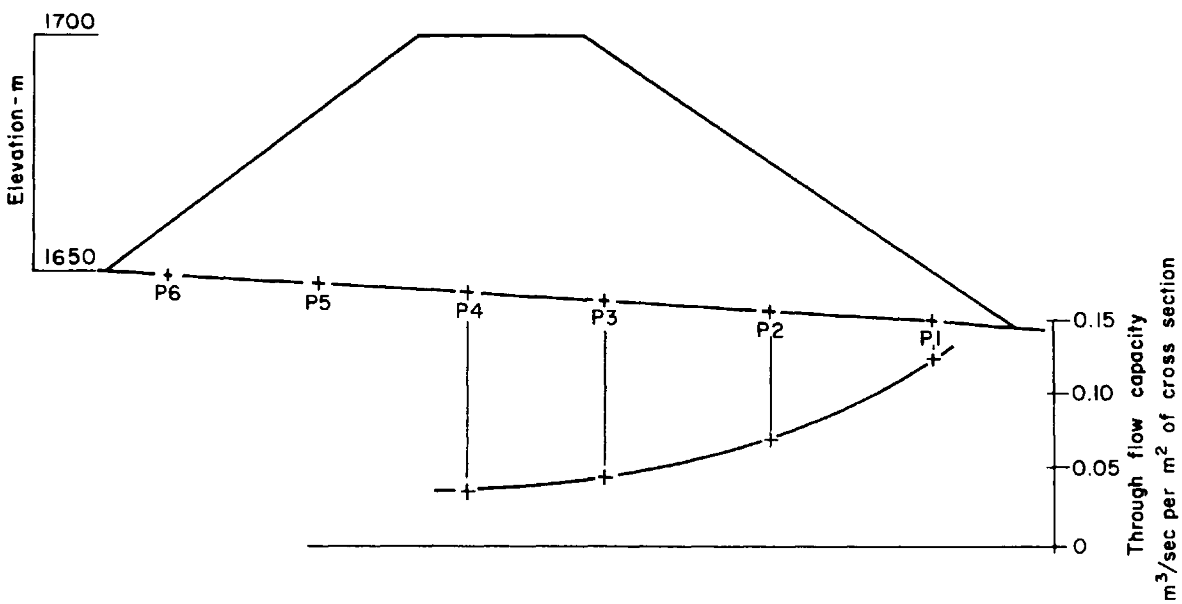


FIG 5

A plot showing the measured through-flow capacities at the Swift Creek rock drain, and illustrating the manner in which through-flow capacity increases in the downstream direction.

ROCK SEGREGATION IN WASTE DUMPS

by

R.S. Nichols
Project Services Supervisor
Esso Resources Canada Limited
Calgary, Alberta

ABSTRACT

A popular concept for transmitting water flow along the base of valley-fill dumps is the use of a coarse rock drain formed naturally by segregation of rock particles in a dump.

Testing of gravels under various conditions was undertaken to determine the amount and nature of segregation. The results indicate that segregation from end-dumped material occurs at three distinct horizons in a pile. The upper 10-15% of the pile contains a concentration of fines. The remainder of the pile to the toe contains a consistent, homogeneous mixture of coarse and fine particles. The zone beyond the slope toe contains a concentration of coarse fractions. These results are comparable to visual observations of waste dumps at Elk Valley coal mines in southeastern British Columbia.

The amount of segregation decreases somewhat with a shorter slope and finer original material. When material is push-dumped, the amount of coarse particles collecting beyond the toe is about half of that for end-dumped material. Also, push-dumping results in an oversteepened, unstable slope.

The test results provide necessary information for estimating rock drain design factors such as rock sizes and drain height of naturally segregated coarse rock.

INTRODUCTION

A key design factor for some waste dumps in southeastern British Columbia coal mines is the use of rock drains to transmit water flows through a dump. These rock drains consist of naturally segregated coarse fractions which collect at the toe region of the dump. This coarse rock provides an excellent conduit for transmitting run-off and creek water flows.

One important factor in determining the flow capacity of a natural rock drain is the amount and extent of the coarse segregated material. In order to quantify the type and amount of segregation, the author attempted to measure actual rock sizes along dump faces at Fording Coal and Byron Creek Collieries. It was found that, aside from the obvious safety concerns, a collection of meaningful data was not feasible. Photographic methods were also unsuccessful. The major problems encountered were the variability in rock sizes and shapes, trying to measure rocks in a consistent manner, selecting a "representative" slope to sample and determining which rocks should and would not be sampled to give unbiased results.

Thus, a relatively simple testing method was undertaken using gravels along a slope under simulated dumping conditions. The purpose of these tests was to determine the nature and amount of segregation under different conditions. Although the test apparatus was rudimentary, the sampling and testing gave consistent results which can be applied to large waste piles.

TESTING APPARATUS AND METHODS

The key elements in producing meaningful and consistent results were the slope, gravels, shovel and sieve analyses. These elements were kept simple with only minor variances throughout the testing program. Figure 1 shows the test apparatus. The tests were performed outdoors during August and September, 1985 at Byron Creek Collieries.

Slope

The slope over which the gravels were dumped was fabricated from 18 mm thick plywood on a wooden frame of 50 x 200 mm studs. The plywood was covered with Netlon CE 121 mesh to add frictional resistance and prevent material from bouncing along the board. The frame rested on a gravel pile to provide a solid base so that vibration of the slope was minimized. Equal spaces were marked off to allow sample collection from consistent and known distances along the slope. Slope lengths used were 2.0 m ("long" slope) and 1.0 m ("short" slope).

An important part of the testing was to determine the slope angle. It was found by trial and error that a 35° slope provided the most consistent and realistic results. Steeper slopes resulted in all the gravel rolling to the bottom. Shallower slopes tended to prevent the gravel from being evenly distributed down the slope because the bulk of material stayed at the slope crest.

In order to achieve a consistent dumping height and position over the slope, it was necessary to "dump" from a plywood platform above the top end of the slope. The rate of dumping was not specifically timed but a "slow" rate gave most realistic results compared to actual dumping conditions.

Gravels

Gravels used for the tests were dry and well-graded. The "coarse" gravel had a top size of 37 mm and contained 1 to 3% silt and clay. The "fine" gravel had a top size of 19 mm with 4% silt and clay. Quantity of gravel used for each test ranged from 8.8 kg to 29.7 kg with an average 17.2 kg per test.

Shovel

The gravels were "dumped" down the slope with a 250 mm wide, flat-mouth, long-handled shovel. There were from five to eight "shovel-fulls" used in a test. In order to simulate end-dumping, the shovel was extended approximately 50 mm beyond the end of the dump platform and raised slowly. Push-dumping was simulated by placing material on the dump platform and slowly pushing gravel off onto the slope. Once a test was complete, the shovel was used to collect samples at predetermined intervals along the slope.

Sieve Testing

A total of 77 dry sieve analyses were performed by the author. The shaker, sieves and scales were provided by Artech Consulting Limited of Cranbrook, B.C. and conformed to ASTM standards. Sieve sizes used were 19.0 mm (3/4"), 9.5 mm (3/8"), No. 4, No. 8, No. 16, No. 30, No. 50, No. 100 and No. 200.

TEST PERFORMED

A total of eleven tests with four different conditions were completed. Nine of the tests simulated end-dumping while the remaining two tests simulated push-dumping. Seven samples were collected from each test for sieve analysis. Six samples came from premarked equal intervals along the slope. The seventh sample consisted of material that had rolled beyond the slope toe.

End-Dumping, Long Slope

Five tests were performed on the long slope. Simulated end-dumping with the coarse gravel was the testing method used in these tests. The purpose of completing five tests under these conditions was to confirm that consistent results could be obtained.

End-Dumping, Short Slope

Two end-dumping tests were performed on the short slope with coarse gravel. These tests contained 11.1 kg and 15.4 kg of gravel. Due to the consistent results for these two tests, additional tests under these conditions were not necessary.

End-Dumping, Fine Material

Two tests were performed using the fine gravel. Test conditions simulated end-dumping on the short slope. The quantity of gravel in these tests was 10.9 kg and 8.3 kg. Again, the consistent results from these two tests precluded the necessity of further tests.

Push-Dumping

Push-dumping of coarse gravel on the short slope was performed in two tests. These tests also showed consistent results. A total of 14.9 kg and 29.7 kg of gravel was used in the tests.

TEST RESULTS

The grain size analyses for samples taken at various slope intervals were plotted on a conventional grain size graph for each test. In order to confirm test consistency, there were five tests done on the "end-dumping, long slope" condition. Figure 2 illustrates the closeness of the results for these five tests at three selected slope intervals. Tests for each of the three other conditions also showed similar consistency, although only two tests were done.

Segregation Along the Slope

The end-dumping tests clearly showed three distinct zones of segregation along the slope. These three zones are:

1. a concentration of fines near the slope crest;

2. an evenly distributed, evenly graded material along the remainder of the slope to the toe; and,

3. a wide dispersion of coarse material beyond the toe.

Figure 3 illustrates these three zones for the "end-dumping, long slope" cases. Note that the middle zone between crest and toe of the slope actually consists of five sample intervals. Although the intervals are not exactly the same, they are considered close enough to be within the same group. These grain size curves are significantly finer than the original material because the coarse particles are segregated out.

The average shift of grain sizes for D_{80}, D_{50} and D_{20} of the original material is listed in Table 1. These multipliers are based on a weighted average of the five tests simulating end-dumping on a long slope. The weighted average of the crest to toe material was used to determine the respective multipliers.

Effect of Slope Height

The two tests simulating end-dumping on a short slope resulted in the same type of segregation as on the long slope. Figure 4 illustrates the grain size curves at various slope intervals. Again, note the three distinct zones of segregation.

One possible effect of the shorter slope is to reduce the amount of segregation. As seen in Table 1, the multipliers for the upper and remainder slope sections are slightly larger than for the long slope.

Thus, although the amount of segregation may vary slightly, the type of segregation is consistent for end-dumping on long and short slopes.

Effect of No Coarse Material

The two tests simulating end-dumping on a short slope using finer material show similar segregation as using coarse material. Figure 5 shows the grain size curves for various slope intervals. The three zones are still evident but not as pronounced as with the coarse gravel.

The effect of the finer material tends to reduce the amount of segregation. The grain size multipliers for these tests, listed in Table 1, are slightly closer to original than for the coarser material.

Thus, the effect of reducing the material grain size is to lessen the amount of segregation. However, segregation into the three distinct zones along the slope is still apparent.

End-Dumping Versus Push-Dumping

Push-dumping was simulated in two tests using the short slope and coarse material. As illustrated in Figure 6, there is no segregation of fines but, coarse particles still do collect beyond the slope toe. The grain size multipliers, listed in Table 1 for these tests, indicate that coarse material segregates and collects beyond the slope toe in a similar manner as end-dumped material. However, the amount of coarse material segregated out is much less when push-dumped.

In the end-dumping tests, an average 75% of the largest particle size rolled beyond the slope toe. This compares to only 40% for the push-dumping tests. Figure 7 illustrates the distribution of material along the slope for coarse material which is end-dumped and pushed-dumped. The profiles confirm that in these tests, most of the material collects along the lower part of the slope area when end-dumped and along the upper part of the slope when pushed-dumped.

One of the main reasons for the difference is the momentum gained by the coarse particles as they slide out of the shovel when end-dumped. This momentum causes the rocks to roll, as opposed to slide, down the slope. The angular momentum developed by an individual rock is much greater than the frictional resistance of the material. This results in the particles not coming to rest unless the slope flattens or they hit particles greater than half their size. Thus, most of the end-dumped material comes to rest at the lower part of the slope. On the other hand, when the material is push-dumped, the coarse particles get "hung up" in the fine aggregate at the slope crest. The only coarse rocks that roll down the slope are those pushed over the slope crest. Thus, although coarse particles accumulete beyond the toe region in both cases, end-dumping yields the most amount of coarse particle segregation.

One other interesting observation was the oversteepened, unstable condition of the push-dumped slope. This condition was common and frequently resulted in mass failures. Although the failures were not catastrophic, all the material on the slope would slide "en masse" and then come to rest. This movement occurred each time more material was push-dumped onto the slope crest. Thus, the slope was continually over-steepened and at or beyond the critical angle of internal friction.

There were no failures evident during end-dump tests. Again, the momentum gained by the material as it slides out of the shovel was sufficient to cause the particles to roll down the slope. This results in a slope angle somewhat less than the friction angle. Thus, end-dumping tends to "flush" out any potentially unstable conditions and produces a non-critical slope angle.

TEST RESULTS COMPARED TO ACTUAL DUMP CONDITIONS

Comparison of the test results with actual dump conditions is based upon observations made at open pit coal mines in the Elk Valley area of southeastern British Columbia. In particular, the author became most familiar with waste dump conditions while employed at Byron Creek Collieries and Fording Coal Limited. Although the author has not worked on waste dumps at Line Creek and Westar mines, observations made during brief tours of these operations indicate that conditions at all the mine dumps are similar.

Most of the waste dumps observed were developed by end-dumping. The material is typically dry and well-graded with a top size of one to two metres. These dumps all exhibit segregation very similar to the test results. These conditions are: fine material is concentrated near the dump crest; there is no segregation along the remainder of the slope to the toe; and the coarsest particles collect beyond the slope toe. Figure 8 shows segregetion along a typical rock dump slope and along one of the test slopes.

The dump heights at Elk Valley mines generally range from 30 to 300 metres. The difference in dump height does not appear to affect the nature of material segregation into the three distinct zones. For example, a 10 m high dump at Byron Creek Collieries exhibited the same segregated zones as did a 300 m high dump at Fording Coal.

The effects of developing dumps with waste rock containing no coarse particles was not observed at the Elk Valley mines. The material gradation of waste rock, although not measured, appears to be similar for all the Elk Valley mines. The top size of two metres is rarely exceeded because it is undesirable for material handling purposes. A top size of under one metre is also rare because of optimized blast patterns and powder factors employed by the minas.

Push-dumping is not a common practice at the Elk Valley mines waste dumps. At Byron Creek Collieries and Fording Coal it is employed when: constructing fill roads less than five metres in height; or when building up berms at the dump crest; or when the material is so fine and soft that bulldozers are required to push the material rather than risk getting a truck stuck. In the first two instances where well-graded and dry material was push-dumped, it was observed that numerous coarse boulders got "hung up" in the finer matrix. Figure 9 illustrates the comparative conditions observed on a dump at Byron Creek Collieries and a slope on one of the push-dump tests. As with the testing, some of the coarse boulders in the dump do break free at the crest and roll beyond the slope toe.

Aside from oversteepening at the dump crest, the adverse stability problems of push-dumping were not observed. This is probably due to the relatively small dump heights as well as the limited use of push-dumping on normal dumps.

DESIGN IMPLICATIONS OF ROCK SEGREGATION ON WASTE DUMPS

In evaluating the test results and comparing with actual dump conditions, several significant design implications are apparent.

Firstly, when a rock drain is required along a dump base, the height of the rock drain formed by natural segregation will be equivalent to the size of the coarsest particles. This height could be increased by concentrating the coarse particles in a confined area such as at the intersection of a dump slope with another slope or excavating a catchment ditch along the desired rock drain route. The grain size distribution of the material in the rock drain zone (beyond the slope toe) can be estimated by applying the appropriate multiplier (Table 1) to the original material size distribution. Figure 10 illustrates how these multipliers can be applied.

Secondly, material from the toe to near the top of an end-dumped pile will be evenly distributed and finer than the original blasted material. Figure 10 illustrates how the shift in grain size can be determined.

Thirdly, the material at the top of an end-dumped pile will contain a higher percentage of fines than the remaining portion of the pile. Again, Figure 10 illustrates the relative shift from original material to this segregated fine material. This concentration of fines may act as a filter at the top of the dump, preventing fines from percolating through the pile to the dump base. This is an important consideration in estimating the sediment load resulting from drainage through dumps.

Fourthly, segregation is most effective and dump stability is maximized when end-dumping is employed.

CONCLUSIONS

Testing of granular materials has successfully defined the amount and nature of rock segregation along a slope. These results are comparable to actual conditions observed at Elk Valley coal mines and can be used to aid in the design of rock dumps and naturally formed rock drains.

When end-dumping is simulated using dry, well-graded gravel, three distinct zones occur in the pile:

1. a concentration of fines in the upper 10%;

2. a well-graded, evenly distributed material down to the toe;

3. a dispersion of predominantly coarse particles beyond the slope toe.

This segregation occurs along short slopes and with finer material. Push-dumping results in coarse particles distributed throughout the pile as well as beyond the slope toe. It also results in oversteepening at the slope crest and unstable conditions in the pile.

The grain size distribution for materials occurring in any of the three segregated zones in an end-dumped pile can be estimated by multiplying the original material grain size by multipliers determined in this testing program. These estimates can be used to aid in the design of rock drain parameters at the base of waste dumps.

ACKNOWLEDGEMENTS

The author wishes to thank Esso Resources Canada Limited and Byron Creek Collieries for permission to publish this paper. The sieve analysis equipment provided by Artech Consulting Limited, Cranbrook, B.C. is greatly appreciated. The author is also grateful to Dillingham Construction Limited for their generous donation of gravels and slope testing materials.

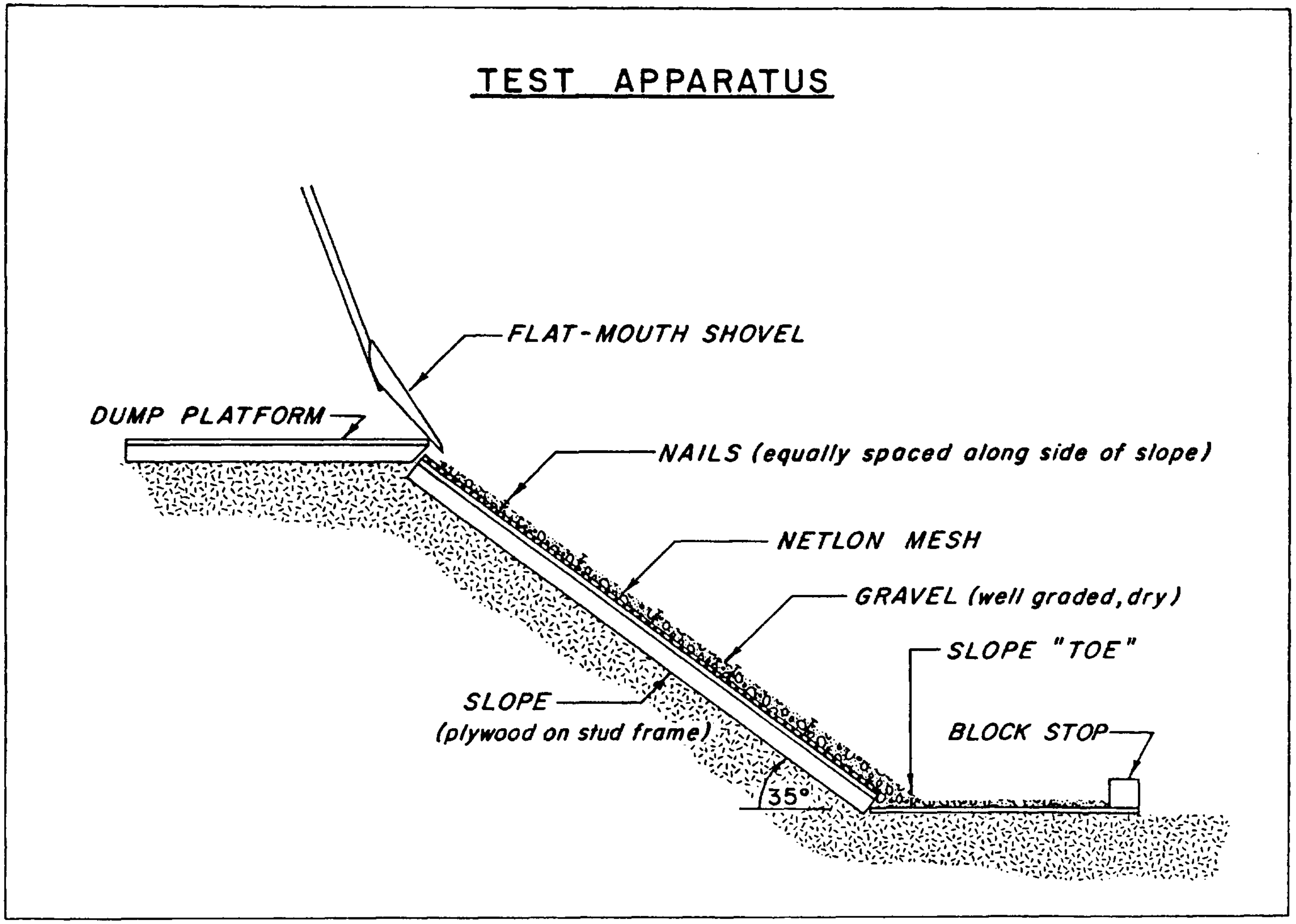


FIGURE 1: Schematic section showing the set-up and apparatus for rock segregation tests.

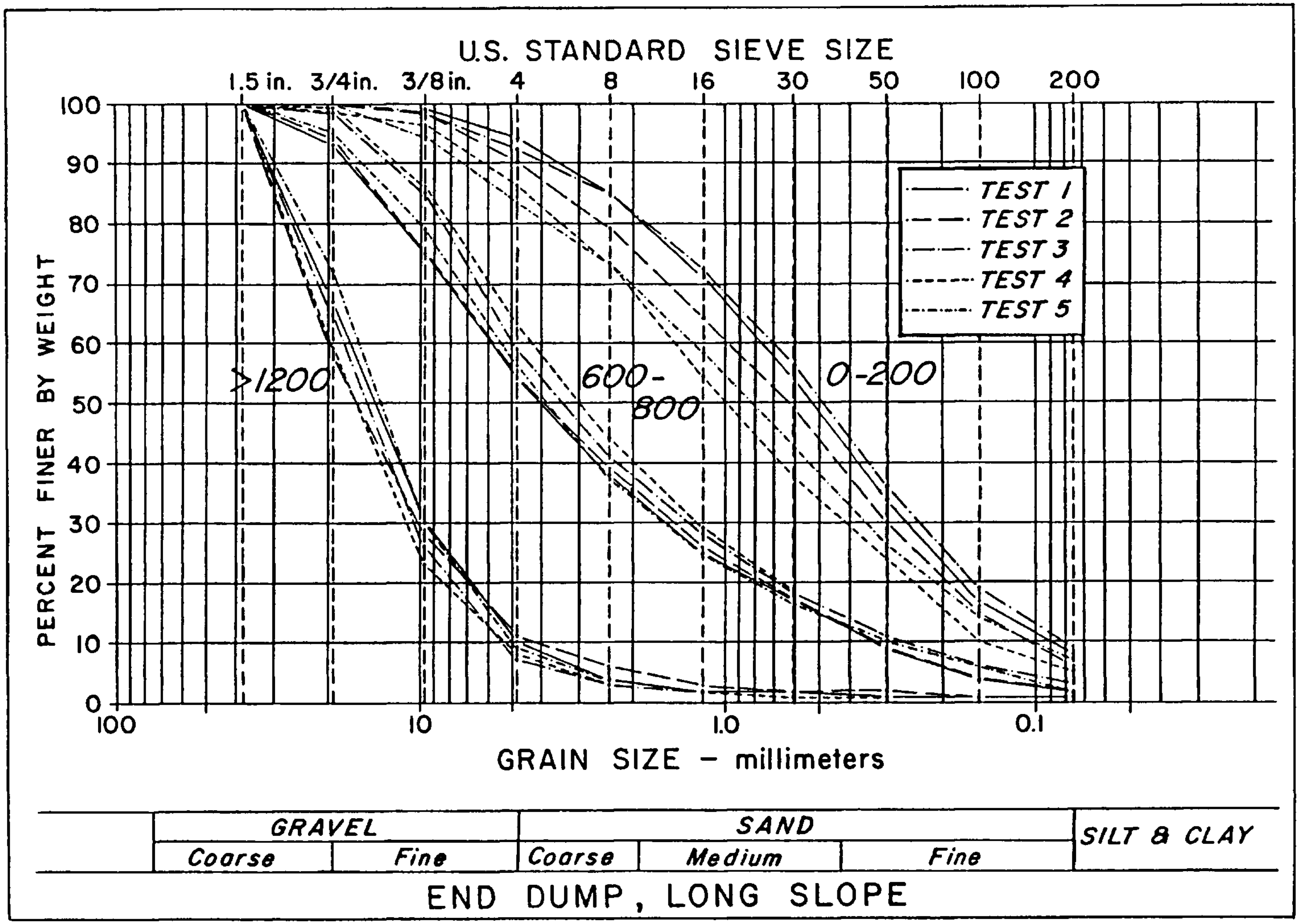


FIGURE 2: Grain size graph illustrating the closeness of results for five tests using end-dumping, long slope and coarse material. The slope intervals selected were 0-200 mm, 600-800 mm and beyond the toe.

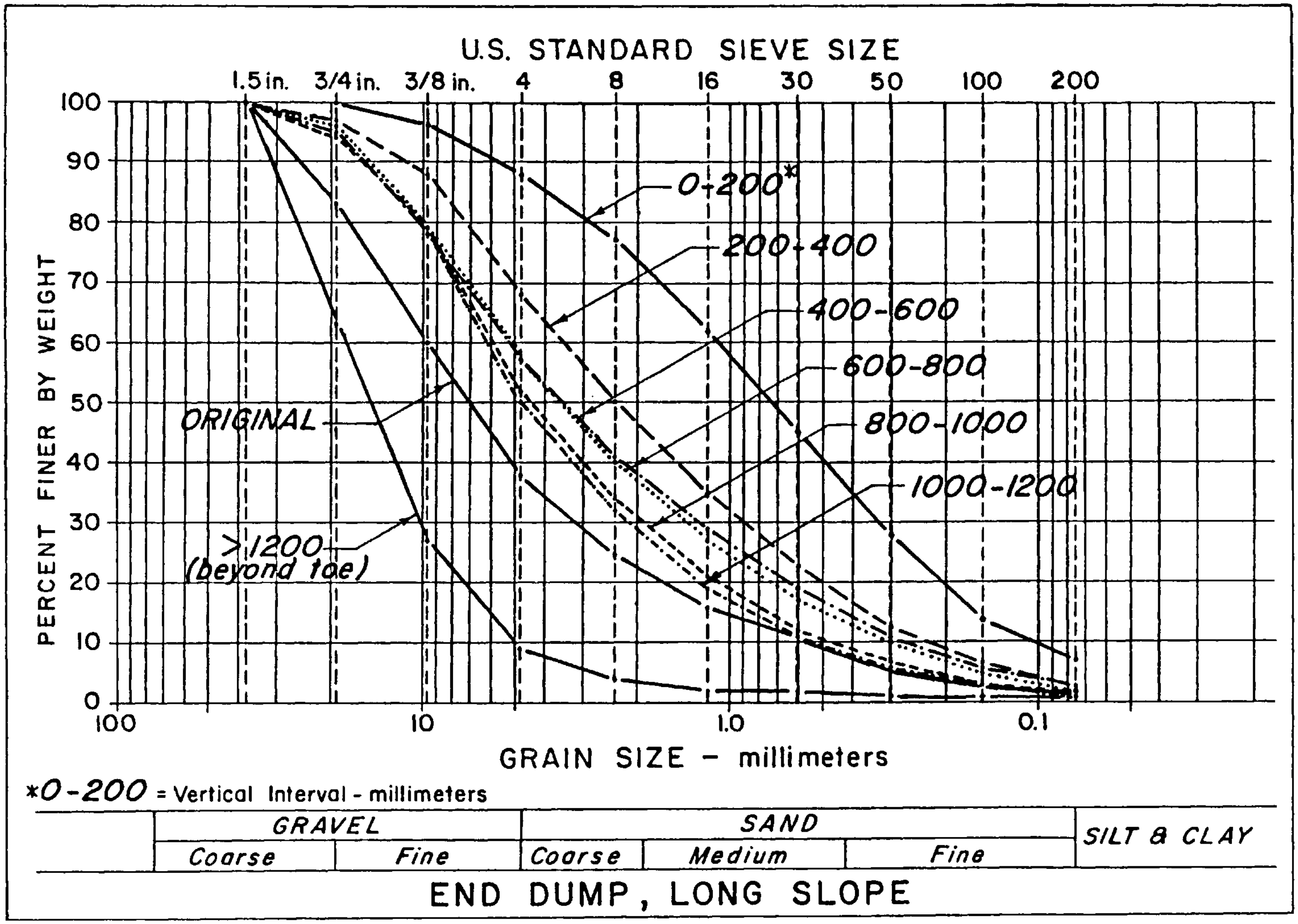


FIGURE 3: Grain size graph of the end-dump, long slope condition. The grain size curve for each slope interval is the weighted average of five tests. Note the three distinct horizons of segregation: 0-200, 200-1200 and >1200.

TABLE I

GRAIN SIZE MULTIPLIERS FOR SEGREGATED MATERIAL *

	D80				D50				D20			
	1	2	3	4	1	2	3	4	1	2	3	4
Original Material	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upper Slope	0.17	0.26	0.59	0.92	0.12	0.16	0.42	0.84	0.11	0.17	0.33	0.63
Remainder of Slope	0.53	0.83	0.83	0.92	0.57	0.77	0.75	0.81	0.53	0.75	0.67	0.68
Beyond Slope Toe	1.39	1.57	1.22	1.75	2.17	1.96	1.88	1.91	3.94	4.17	4.44	3.68

1 = End-Dump, Coarse Material, Long Slope

2 = End-Dump, Coarse Material, Short Slope

3 = End-Dump, Fine Material, Short Slope

4 = Push-Dump, Coarse Material, Short Slope

* That is: the number by which the original material is multiplied to determine the size of segrated material. For example, if the original material has a D50 of 9.0 mm, the D50 of material along the "remainder of slope" for Test 1 conditions is 9.0 mm x 0.57 = 5.1 mm.

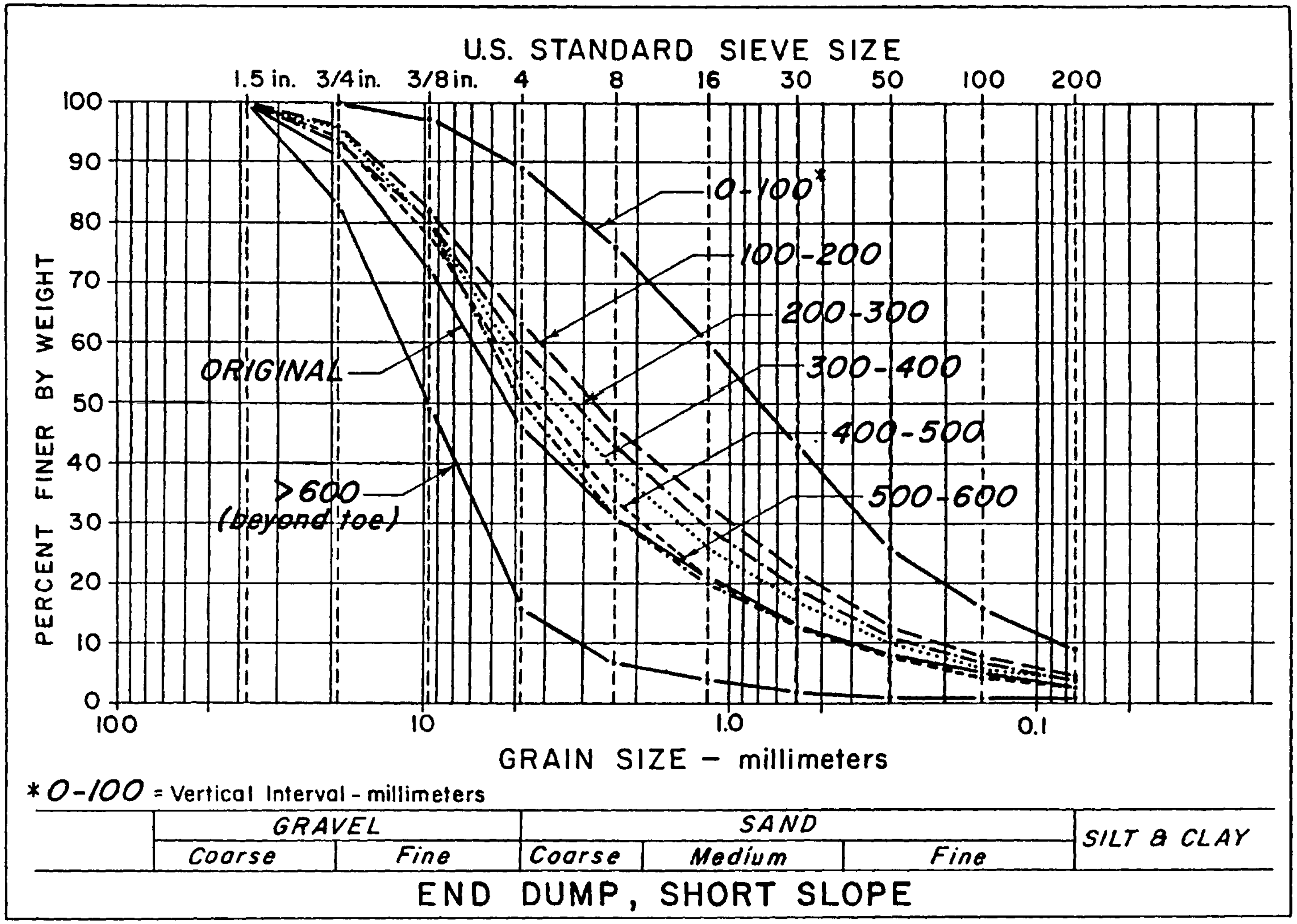


FIGURE 4: Grain size graph of the end-dump, short slope condition. The grain size curve for each slope interval is the weighted average of two tests. Note the three distinct segregated horizons of 0-100, 100-600 and > 600.

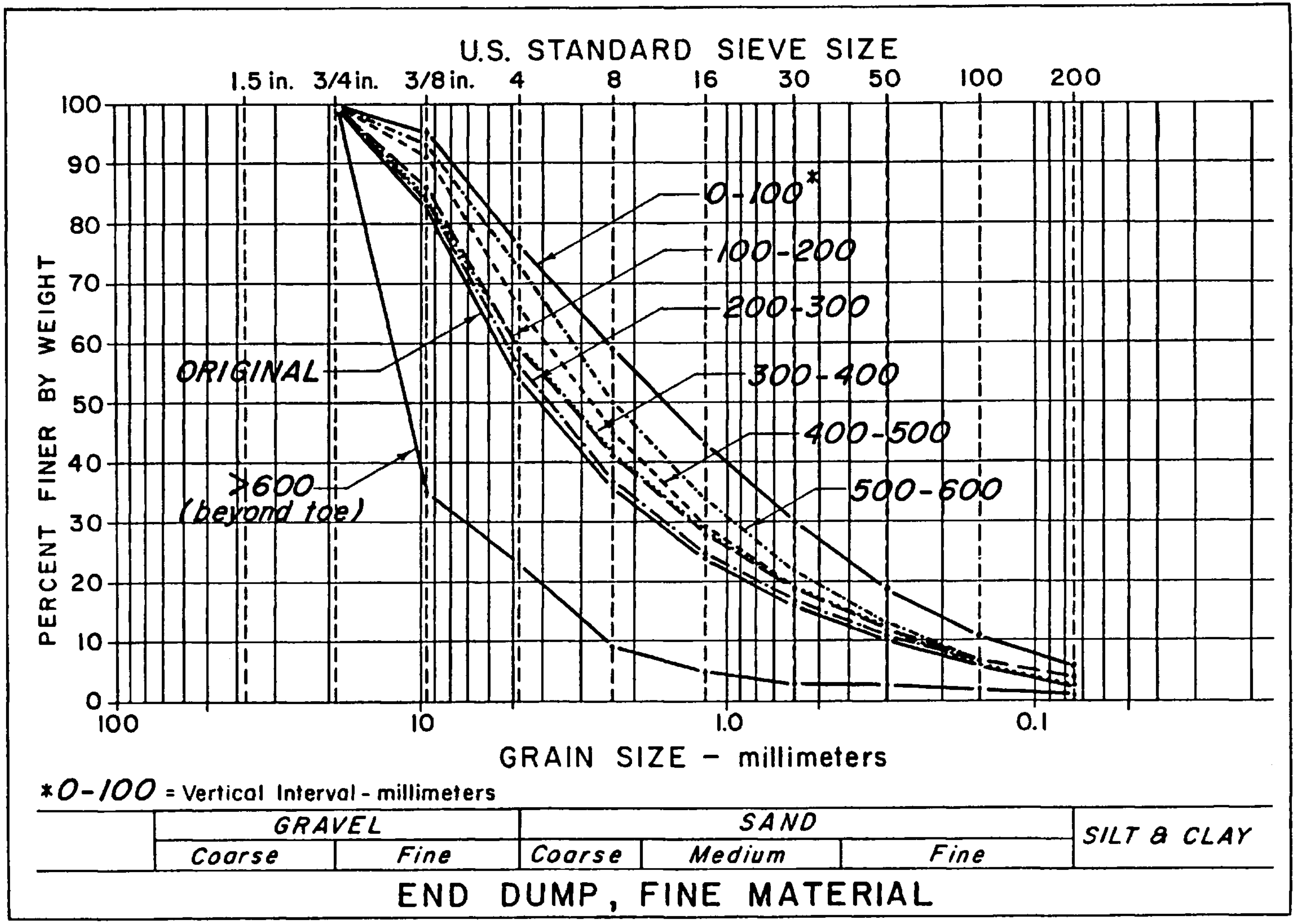


FIGURE 5: Grain size graph of the end-dump, fine material on a short slope condition. The grain size curves for each slope interval is the weighted average of two tests. The three segrated horizons are not as pronounced as for the coarse material conditions.

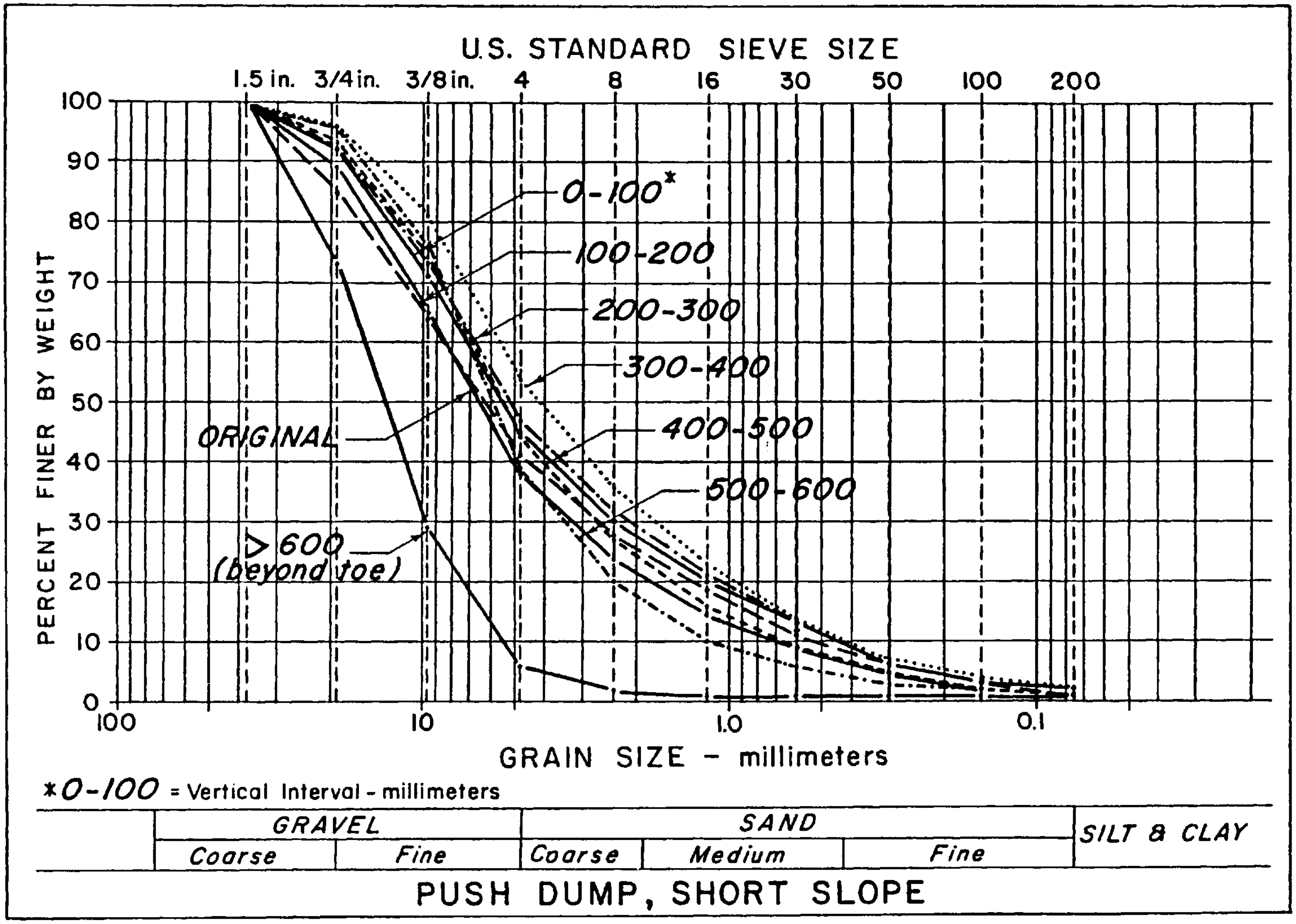


FIGURE 6: Grain size graph of the push-dump, short slope condition using coarse material. The grain size curves for each slope interval is the weighted average of two tests. Note there is no fines segregation. The > 600 curve indicates that coarse material still collects beyond the toe when end-dumped.

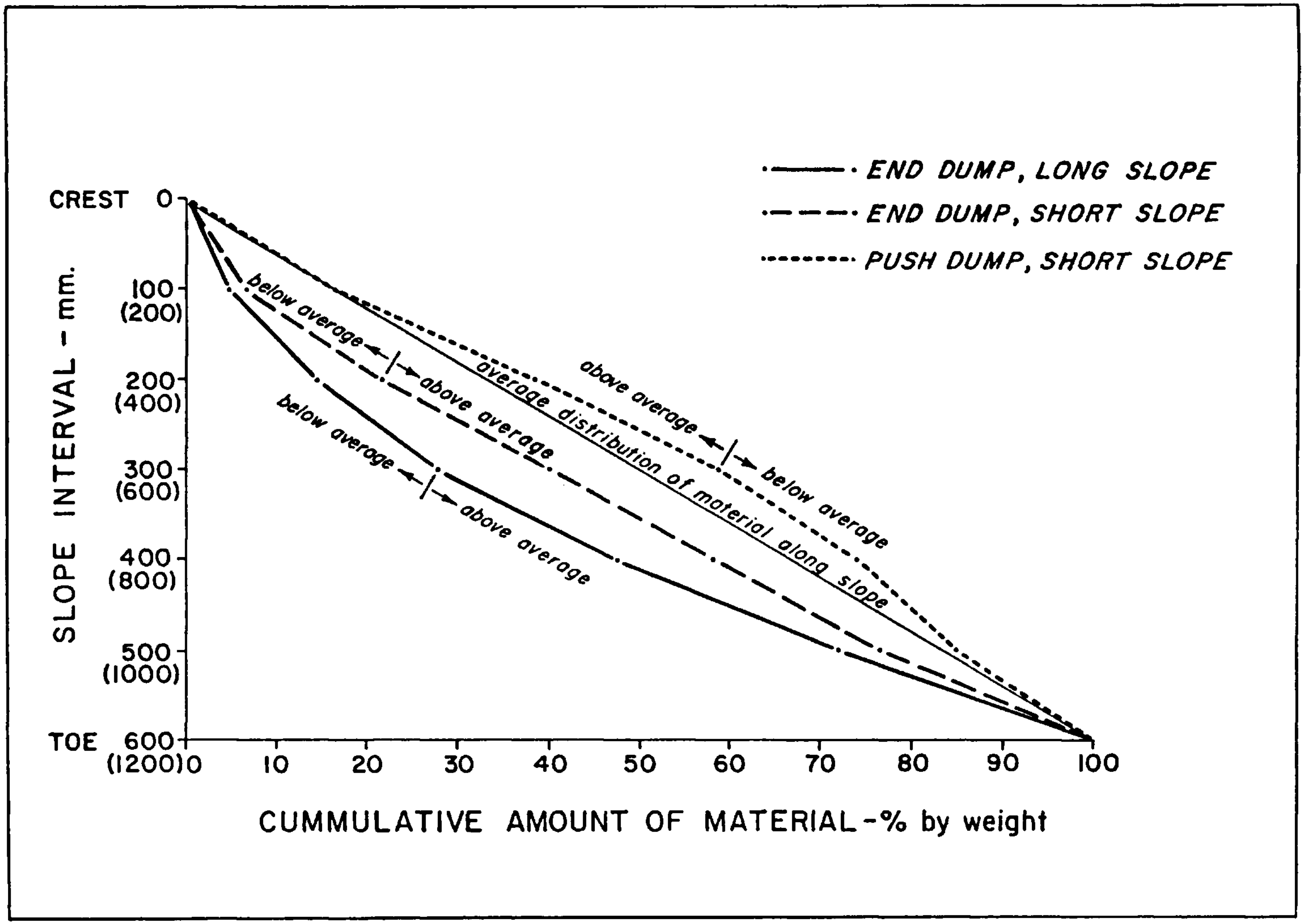


FIGURE 7: Graph showing the relative amount of material occurring at slope intervals in end-dump and push-dump conditions. Note that the push-dump curve shows above average amount of material at the top, and below average amount at the bottom of the slope. This illustrates the oversteepened, understable condition of push-dump slopes.

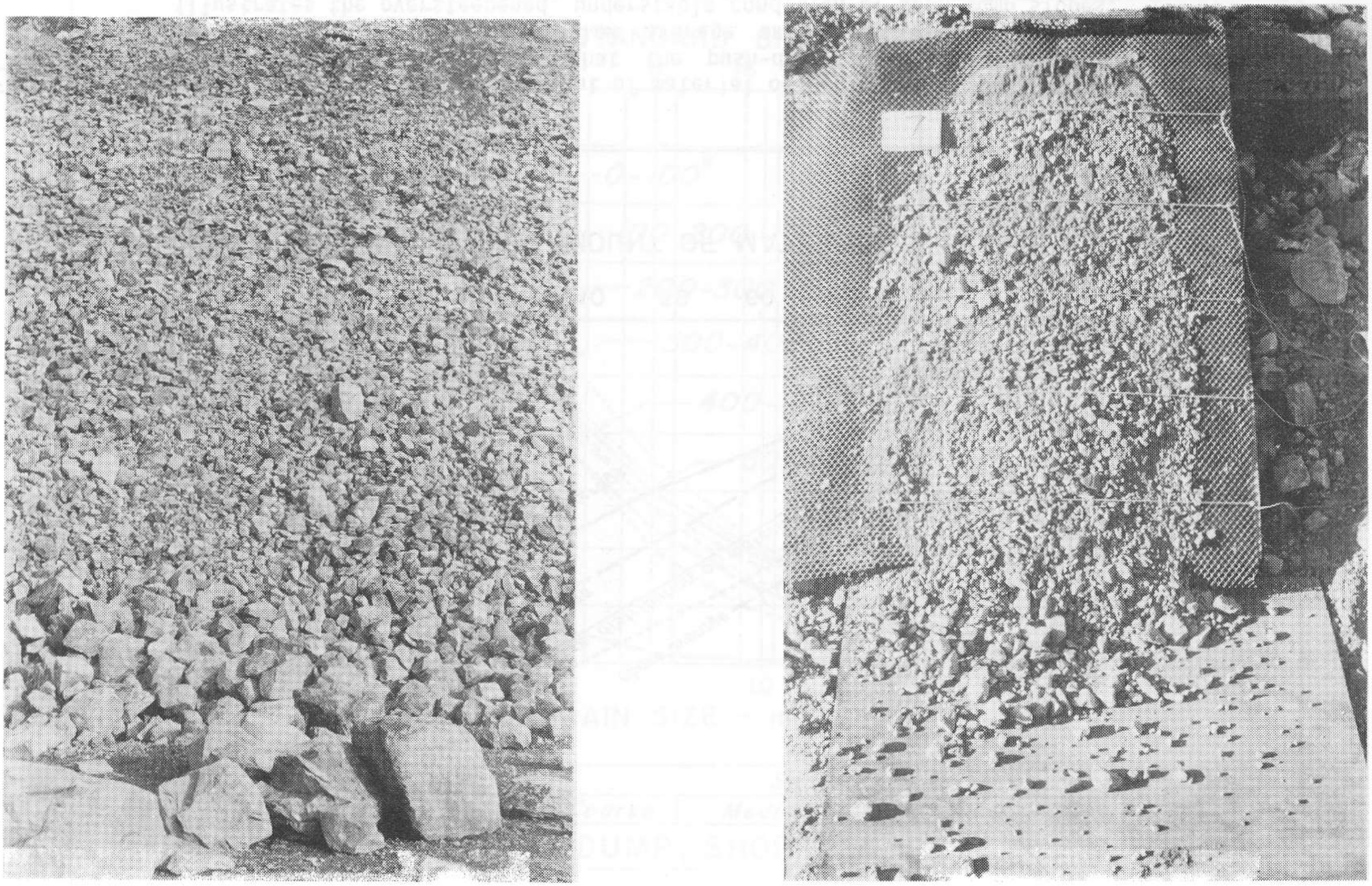


FIGURE 8: Photographs of (a) a 60 m high dump formed by end-dumping at Byron Creek Collieries and (b) end-dumped material distribution along the short slope with coarse gravel. Note the similarities in segregation between actual and test dump conditions.

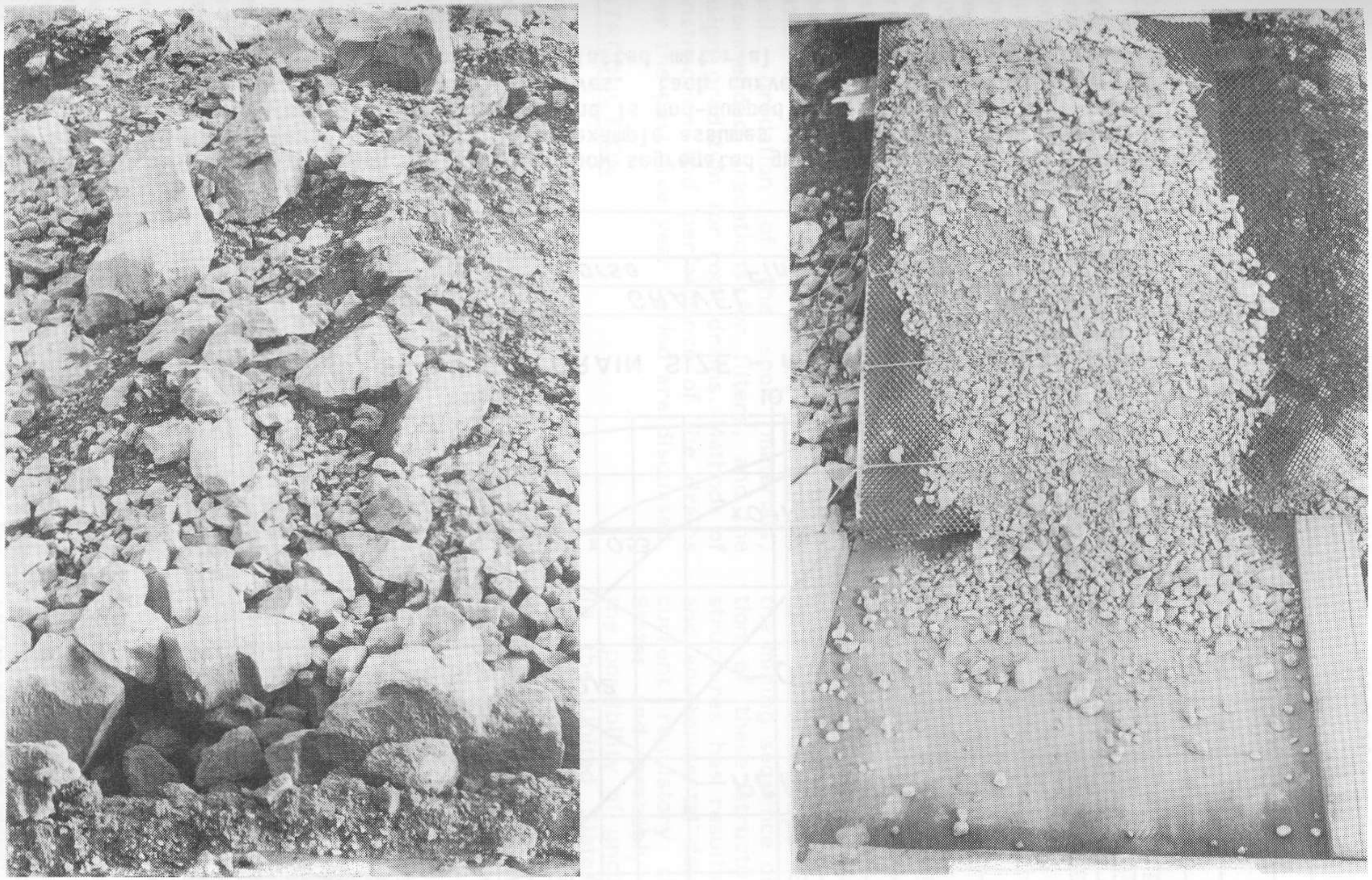


FIGURE 9: Photographs of (a) a 5 m high dump formed by push-dumping at Byron Creek Collieries and (b) push-dumped material along the short slope with coarse gravel. Note the coarse particles "hung-up" at the slope crest in the actual and test dump conditions.

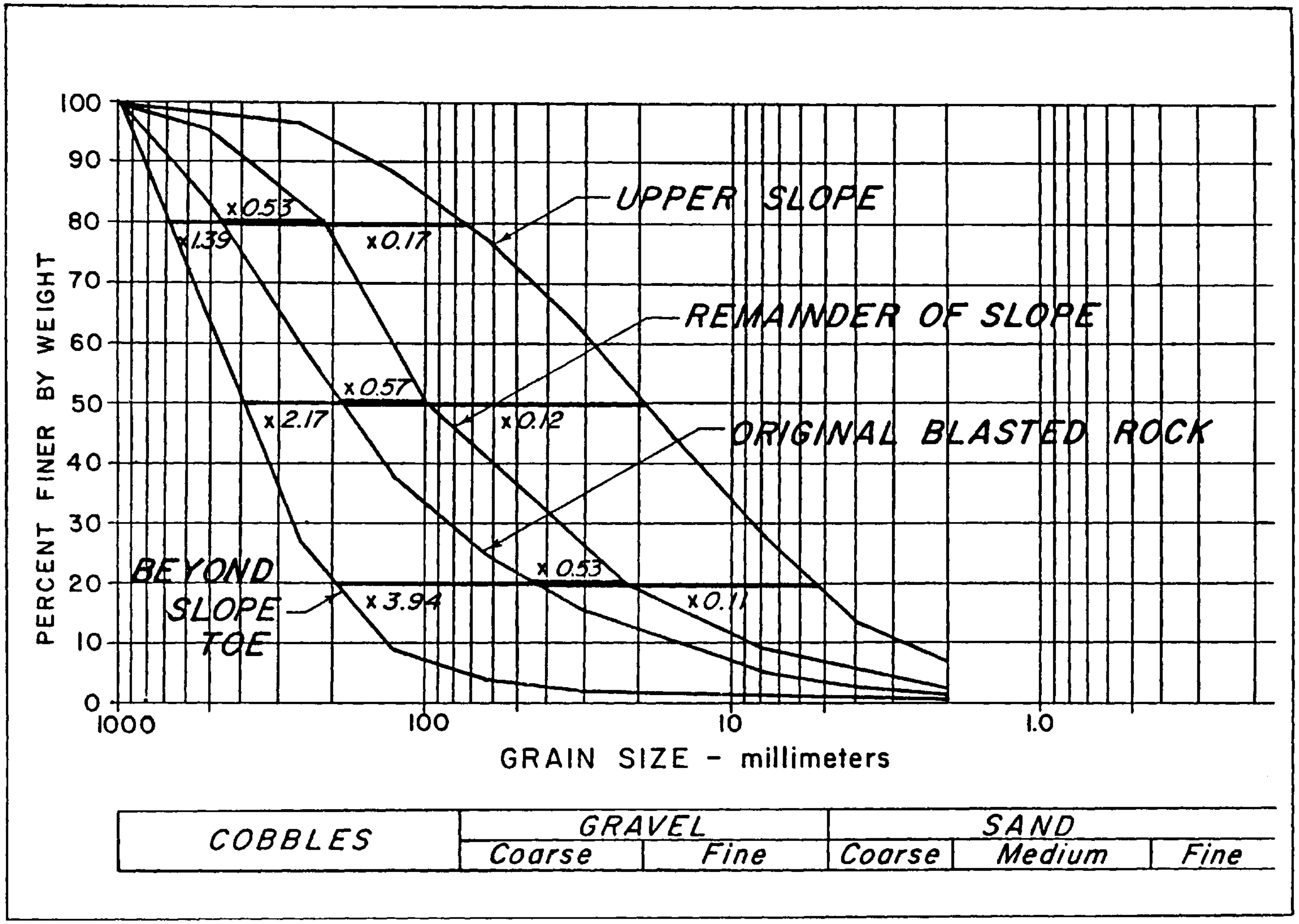


FIGURE 10: Grain size graph illustrating how segregated grain size curves can be estimated using the multipliers in Table 1. This example assumes the original blasted material has a top size of 1000 mm, is evenly graded and is end-dumped on a long slope. The resulting segregation produces three grain size curves. Each curve is based on multiplying the D_{80}, D_{50} and D_{20} values of the original blasted material by the multipliers listed in Table 1 and shown here.

ROCK DRAINS SPOIL DISPOSAL AREAS
ECKMAN PARK MINE
STEAMBOAT SPRINGS, COLORADO

by

Robert W. Thompson, President
CTL/Thompson, Inc.

ABSTRACT

Data regarding construction of two large spoil embankments is presented. The embankments range from 6 to 7 million cubic yards in size and were constructed in north central Colorado. The mine is located in an area where the altitude ranges from 7,000 to 8,000 feet and receives most of its annual precipitation during the winter months in the form of snow. Large rock drains were installed below these structures to limit the possibility of accumulation of water in the embankments. Data presented includes information on characteristics of the overburden spoil, gradation of the spoil materials, recommended gradations for filters, and the design gradation for rock drains. Methods of construction and performance of the drains over the past five year period are discussed.

INTRODUCTION

Drainage is incorporated into any large embankment to protect the structure from de-stabilization forces caused by accumulation of water in the embankment or foundations. Failure of mine generated waste structures can result in loss of life and serious property and environmental damage. Failure of a tailings dam last year in the Italian Alps resulted in flooding of a small town, numerous deaths and extensive property damage. In 1972, failure of a mine waste dump during a period of moderate rainfall in West Virginia resulted in considerable property damages and over 100 deaths downstream of the structure.

Extensive drain systems are normal practice in the design of rolled earth dams or the construction of large rock fill dams. Usually these drains are designed to control the phreatic surface in the downstream portions of the structure and in some instances to reduce uplift pressure on the downstream toe of the structure. Quantity of flow to be controlled by these drains can be large particularly in the case of rock fill dams. The problem facing the designer usually includes determination of the nature of material to be incorporated into the embankment, approximate volumes of water to be discharged by the drain and availability of materials suitable for construction of the drain.

Incorporation of drain systems into mine waste disposal sites is a relatively new consideration. Unfortunately, historical practices in mining have usually resulted in dumping of the extra materials at the most convenient location that did not limit the possibility of future mining and minimized haul distance. Operational requirements of the mining sequence dominated the construction of these structures. Failure of these structures has resulted in serious economic and environmental damage in the past. The current regulatory climate in the United States end probably in Canada precludes the possibility of uncontrolled construction. In the United States, the Department of Interior, Office of Surface Mining, developed regulations in the late 1970s to address various problems associated with large surface mining operations. Disposal of waste generated by these operations was addressed in detail in Federal Regulations which were later incorporated into mining regulations at various state levels.

The Energy Fuels Corporation was operating a large surface mine south of Steamboat Springs, Colorado in the late 1970s. The mine had been in operation for several years prior to the Surface Mining Regulations. In 1979, a substantial addition to the mine was proposed. This addition was known as the Eckman Park Mine. This report includes data which was developed regarding the nature of spoil from the Energy Fuels mine and the design of waste disposal facilities for the Eckman Park extension of the mine. The report primarily addresses considerations with respect to design of drains and a subsequent installation of the drains.

At the time of planning for the Eckman Park Mine, the Office of Surface Mining was in the last stages of establishing requirements for the design of excess fill disposal structures. The regulations published May 5, 1979 generally governed the design which was submitted to Energy Fuels on September 11, 1979.

SITE SELECTION

The mine plan imposed several constraints on areas which could be selected for disposal of excess spoil. The excess spoil was generated from initial box cuts which were anticipated to occur during 1980 and 1981. The spoil disposal sites were selected in areas where overburdrn thicknesses were sufficient to make future surface mining not feasible. Additionally, the sites were selected where the ground surface conditions were reasonably favorable and haul could be accomplished without excessive environmental damage. To minimize environmental disturbance, Energy Fuels specified locations to be within the same drainage area. Area I was a very gently sloping valley bottom located east of existing Haul Road "C" and north of the active mine. Area II was a large sloping area near the southwest end of the proposed Eckman Park Mine. Area III was located along a moderately sloping ridge. This area had such limited potential for storage volume that it was dropped from consideration after exploratory borings were drilled.

GENERAL GEOLOGIC CONDITIONS

The mine area is underlain at relatively shallow depths by the Williams Fork formation which is a part of the Upper Cretaceous Mesa Verde Group. Within the mine area, the Williams Fork formation includes a sandstone series which is fine to very fine grained sandstone interbedded with thin beds of siltstone and shale. The sandstone is a light gray color. There are massive marine shale members within the formation. The shale series are predominantly claystones with some siltstones. Generally, the shales are darker gray in color. The soils which mantle the surface are probably the result of in-place weathering of the formation materials. The thickness of surficial soils ranges from nearly zero to as much as 10 feet on the upper, higher portions of the site. In the valley area, borings encountered up to 20 feet of soft to medium stiff residual clays. The formations are relatively impervious and where water percolates down through the near-surface soils it tends to perch on the surface of the relatively unweathered formation rock.

SPOIL PROPERTIES

Surface mining was occurring at several locations at the Energy Fuels Mine south of Steamboat Springs. The spoil disposal areas were part of an extension of an existing mine operation. The overburden to be developed as excess spoil was believed to be very similar to overburden from the existing mining operation. To evaluate the nature of overburden which would be used to construct the spoil disposal areas, spoil from the operating mine was tested.

Specific Gravity

The sandstone and shales which overlie the coal were exposed in existing cuts. Four samples which appeared to cover the range of typical materials in the exposed overburden cuts were obtained. A description of the samples and the moist unit weight is presented in the table below:

Sample	Description	Specific Gravity	Unit Weight (pcf)
I	Dark gray, massive shale	2.39	149
II	Light tan to gray sandstone	2.13	133
III	Banded gray shale	2.29	143
IV	Dark gray shale	2.35	147

The specific gravity tests were performed on samples obtained from bulk relatively intact pieces of the above described materials. The average unit weight of the individual particles is 143 pcf. This unit weight includes natural field moisture.

Field Density Tests

In order to evaluate the amount of bulking which occurs when the overburden is excavated and dumped into loose piles, as is the case when the mining is accomplished using a dragline, numerous field density tests were made at various locations at the existing mine. Large sand cone density tests were attempted at four locations. These tests were performed by excavating a hole in the spoil pile and filling the hole with sand, using sand to determine the volume of the hole. Because of voids occurring in the spoil pile, this type of test can be misleading. In our opinion, Test No. 2 was influenced by sand penetrating voids within the pile. In addition to the sand cone procedure, nuclear moisture density tests were made using a troxler Model 3411 B gage. The device used can be extended into the test area 12 inches. The test area was leveled and the test was run by inserting the probe into the ground in 2-inch increments and making a reading at each 2-inch increment. Additionally, the tests were checked by rotating the nuclear gage 90 degrees and making four readings at each level. The results of these tests are shown in tabular form in Table 1. The reported density is the average of a series of readings for the nuclear gage.

A. Big Sand Cones

Test No.	Material	Moist Weight (pcf)
1	Loose spoil, shale	95
*2	Loose spoil, shale	76
3	Loose spoil, shale	90
4	Loose spoil, shale clayey	88

*(test unreliable)

B. Nuclear Tests**

Test No.	Material	Moist Weight (pcf)
5	Partially compacted spoil	113
6	Loose spoil	93
7	Partially compacted spoil	107
8	Loose spoil	91
9	Partially compacted spoil	121
10	Partially compacted spoil	118
11	Partially compacted spoil	121
12	Loose spoil	107
13	Loose spoil	102
14	Loose spoil	97

**Nuclear test results are average of four tests obtained by rotating nuclear gage 90 degrees each time (in spoil only).

The average unit weight of the overburden material was believed to be about 143 pounds in place. Where tests were made in very loose spoil piles, the density ranged from 88 to 107 with the average approximately 95 pcf. Where the spoil had been partially compacted, the density ranged from 113 to 121 pcf, with an average estimated at about 116 pcf. Determination of the actual bulking factor is difficult because tests could not be made in the lower portions of the spoil piles. The only areas accessible were at the surface and on the upper portions of existing loose spoil piles. Based on the experience at the mine, and the results of these tests, the actual bulking factor was estimated to range from 26 to 30 percent and for design purposes recommend a bulking factor of 28 percent. Density tests made in the existing haul roads indicated that compaction could be achieved from travel with the haul trucks. Earlier in the summer, compaction of fill in an area where spoil had been placed to support a fuel tank was checked. Where a thick layer had been placed subject to compaction by haul units, the average in-place density ranged from 88 to 93 percent standard Proctor maximum dry density. The near-surface tests indicated compaction above 100 percent for standard Proctor density. The proctor density for the materials checked at that point was 108 pcf dry. All of this data was used in evaluation of the typical properties of the materials to be placed in the spoil piles.

Observed Angles of Repose

Numerous observations of the angle of repose for various materials encountered at the existing mine were made. The properties of the overburden rock were tested and reported to Energy Fuels Corporation in a report designated "Preliminary Design of High Wall Slopes", Kenneth C. Ko and Associates, dated March 1977. The observed angles of repose ranged from 34 to 40 degrees for the loose dumped spoil. For purposes of design, 37 degrees was selected. The Canadian Pit Slope Manual reports typical values of the angle of internal friction for shale to run on the order of 34 degrees and for sandstones to range from 35 to 45 degrees. These values are well within the range observed at the mine.

Gradation

A sample of the spoil from an existing mined area was obtained to evaluate the gradation of future spoils. The test sample weighed about 400 pounds. Sizes larger than 12 inches were excluded from the test sample. The spoil rock tended to break down during handling and compaction. Based on observations of fills constructed with the spoil for haul roads and other uses at the active mine area, we estimated that about 30 to 40 percent of the spoil will be 12 inches or larger after compaction. The minus 12-inch fraction of the spoil is, in our opinion, important in determining the requirements for a transition zone between the spoil and the fill drains. Gradation of the sample was as follows:

Sieve Size	% by Weight Passing
8-inch	87.1
6-inch	68.7
3-inch	55.1
1.5-inch	43.1
3/4-inch	33.1
3/8-inch	24.9
No. 4	16.3
No. 8	10.2
No. 16	6.6
No. 30	4.8
No. 50	3.7
No. 100	3.0
No. 200	2.3

As indicated in the table above, the sample contains only 2.3 percent passing the No. 200 sieve and about 10 percent passing the No. 8 sieve. It is our opinion that compaction will further break down the spoil and the actual permanent disposal areas will have more fine material than indicated by this gradation test.

Permeability

In a loose state, the spoils have very, very high permeability. The material is an angular broken rock consisting of sandstone and shales. After compaction, these materials decrease in permeability. The method of placement as well as the type of compaction may have significant influences on the permeability. For purposes of evaluating the possibilities for saturation, the overall permeability of the fill was estimated to range between 100 and 1,000 feet per year, with the 1,000 feet per year a more likely figure. Placement of the fill in layers and the possibility of developing highly compacted surfaces increased the risk of the horizontal permeability being greater than the vertical permeability. Regulatory requirements specified fill layers not exceed 4 feet.

DRAIN DESIGN

The two spoil disposal facilities that were constructed in Eckman Park were designed to contain between 6 and 7 million cubic yards of spoil in each facility. As much as possible, surface drainage was routed around these large fills. However, the natural topography in both locations included incised drainages and experience at the mine indicated that seepage would collect on the surface of the bedrock. Rock drains were specific in the incised drainages. Because of regulatory requirements, the size of rock incorporated in the drain as well as the size of the drain had to meet specification minimums. The minimum acceptable drain size was 16 feet in width and 16 feet in height. The rock in the underdrain had to be 90 percent larger than 12 inches with no rock any larger than 25 percent of the width of the drain. Further, the regulations required that the drain be installed along the natural drainage system.

A sandstone lense about 10 feet thick occurred above the Lennox seam in the mine area. Slake durability tests on samples of this material were performed using the method developed by Franklin and Chandra. These tests indicated the rock was suitable for use as drain rock fill. Considering the gradation of the materials to be Included in the spoil pile, drain rock was specified as follows:

a. 100 percent - 3 feet or smaller in size
b. 35 percent - 2 feet or smaller in size
c. 10 percent - 1.5 feet or smaller in size

This material was developed by selective excavation of the sandstone above the Lennox seam. A filter zone between the drain rock and the spoil was designed using the method of U.S. Bureau of Reclamation published in "Design of Small Dams". The method was developed by the Bureau of Reclamation and the U.S. Corps of Engineers and is a generally accepted method for design filters. The filter design was based on the gradation of materials smaller than 12 inches in the spoil. Based on this gradation the recommended filter gradation was a maximum size of 24 inches with at least 90 percent passing the 18-inch size, 45 percent passing the 8-inch size and 10 percent passing the 3-inch size. A maximum of 3 percent passing the No. 200 sieve was specified. A minimum of 2 feet of this select material was placed between the coarse open-graded rock drains and the spoils fill. The filter material was substantially more expensive than the rock and was developed using a crushing and screening operation. Because of the regulatory minimums, the discharge capacity based on estimated flow velocities from the cross-sectional area was substantially greater than the anticipated flow.

DRAIN CONSTRUCTION

As discussed in the "Drain Design" portion, a seam of sandstone located above the Lennox coal seam was checked and found to be suitable for construction of the drains. Picture No. 1 illustrates the appearance of the rock in the quarry. Picture No. 2 shows the approximate size of fragments included within the rock drains. The construction sequence consisted of cleaning all vegetation and loose materials from the natural drainages and in most cases exposing bedrock in the upper spoil disposal area. In the lower disposal area, trenches above 16 feet in depth were excavated which, for the most part, bottomed in weathered sandstones and shales but in some areas bottomed in residual clays. The material was then dumped from haul trucks into the prepared excavations. The transition zone was placed over the drain from smaller haul trucks. Picture No. 5 shows the exit point for one of the fill drains. Picture No. 6 provides an overview of the upper spoil area showing benches at approximately 50 feet vertically and the nature of the surrounding terrain. Picture No. 3 shows a lift of fill being placed in the lower spoil disposal site. Picture No. 4 shows appearance of the fill after it had been spread with a dozer. Each lift was spread and placed in approximately 4 foot thickness. The entire lift was then subject to compaction from the haul equipment as the succeeding lift was placed. Density tests taken during the initial three months of construction generally indicated the desired compaction was achieved by this procedure. Spreading operations followed by traffic from the haul trucks tended to both break down the larger pieces of spoil and form layers of higher density and finer material at the surface of each of these layers. For this reason, we believe the horizontal permeability of a given layer may be substantially greater than vertical permeability. The performance of the fill to date has not shown any evidence of seepage at intermediate benches. Discharge from the drains has been limited, usually for short periods during the annual spring thaw and runoff.

CONCLUSIONS

1. Two large fills for disposal of excess overburden material were constructed in 1980 and 1981 with large rock underdrains. The fills contain nearly 13 million cubic yards of material and are located in an area that receives annual rainfall of about 20 inches; most of which comes as snow during the winter months. To date, the spoil disposal facilities have performed well with no evidence of instability. Only minor discharge during the spring runoff has been observed at the drain locations.

2. Based on field experience at this mine, development of a suitable filter material was the most expensive part of the construction operation on a unit basis. The large rock for the rock drain was relatively inexpensive and developed by sective excavation within the mine.

3. Control of construction of this type of fill and drain is difficult and relatively crude when compared with experience gained from construction of earth and rock dams.

ACKNOWLEDGEMENTS

Considerable information in this paper was derived from studies conducted for Energy Fuels Corporation at their Eckman Park Mine. This mine was subsequently purchased by Colorado Yampa Coal Company. The author wishes to thank the companies for their permission to make site specific data available.

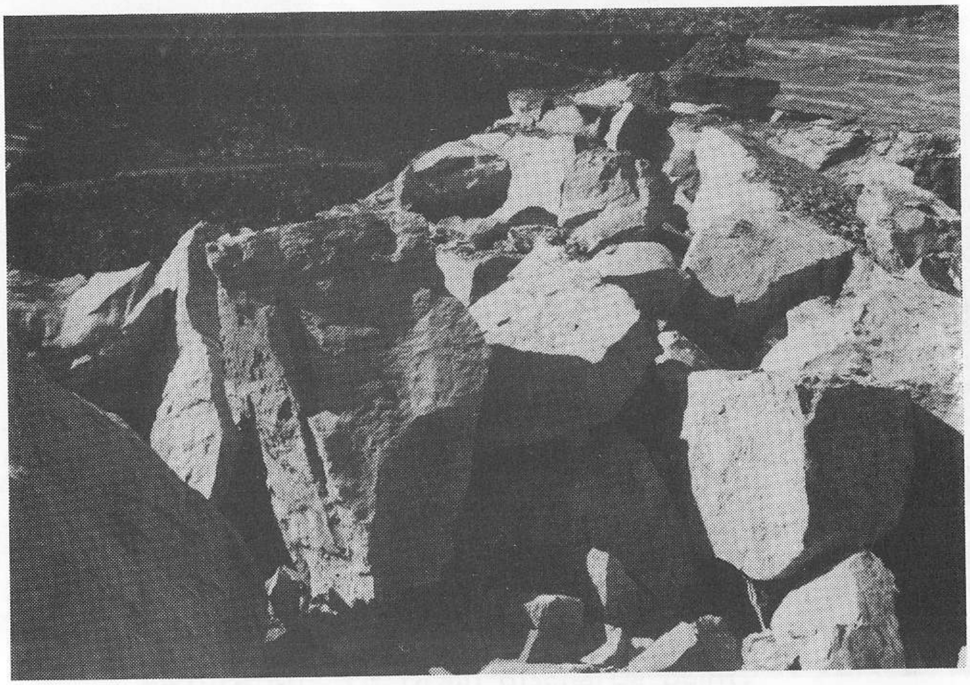
REFERENCES

CTL/Thompson, In., 1979, "Excess Spoils, Eckman Park Mine," For Energy Fuels Corporation, Steamboat Springs, Colorado.

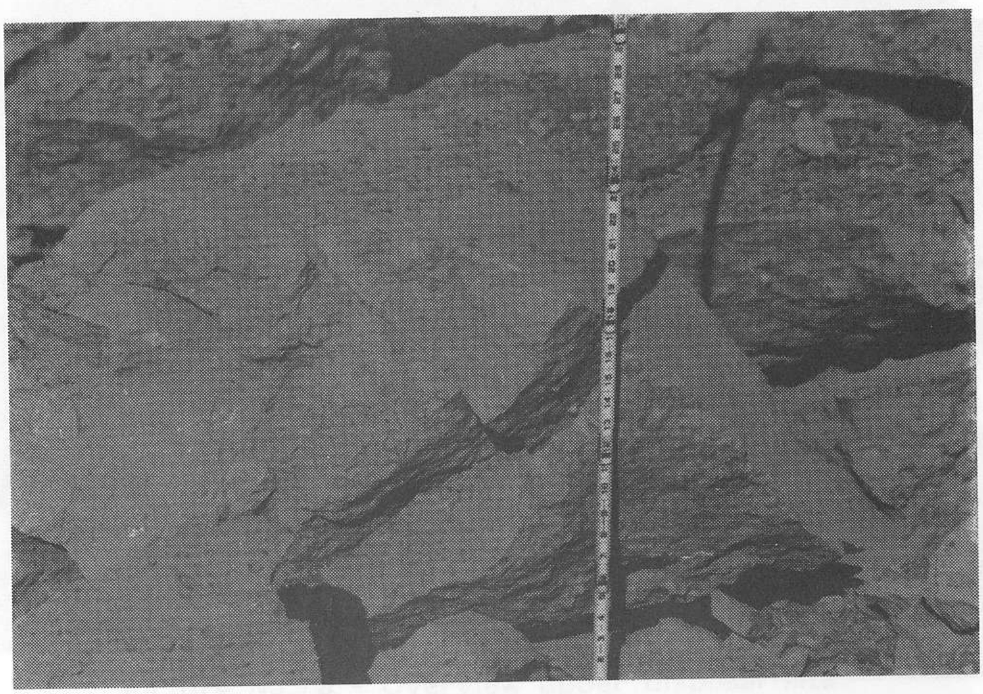
Roth, Cesare, Allison, 1977, "Rapid Monitoring of Coal Refuse Embankments", ASCE Proc. Geotechnical Practice for Disposal of Solid Waste Materials, Ann Arbor, Michigan.

____, 1977, "Engineering and Design Manual - Coal Refuse Disposal Facilities", Prepared for Mine Safety and Health Administration by D'Appolonia Consulting Engineers.

____, 1979, "Surface Coal Mining and Reclamation Operations - Permanent Regulatory Program", Department of Interior, IN Federal Register, March 13, 1979.

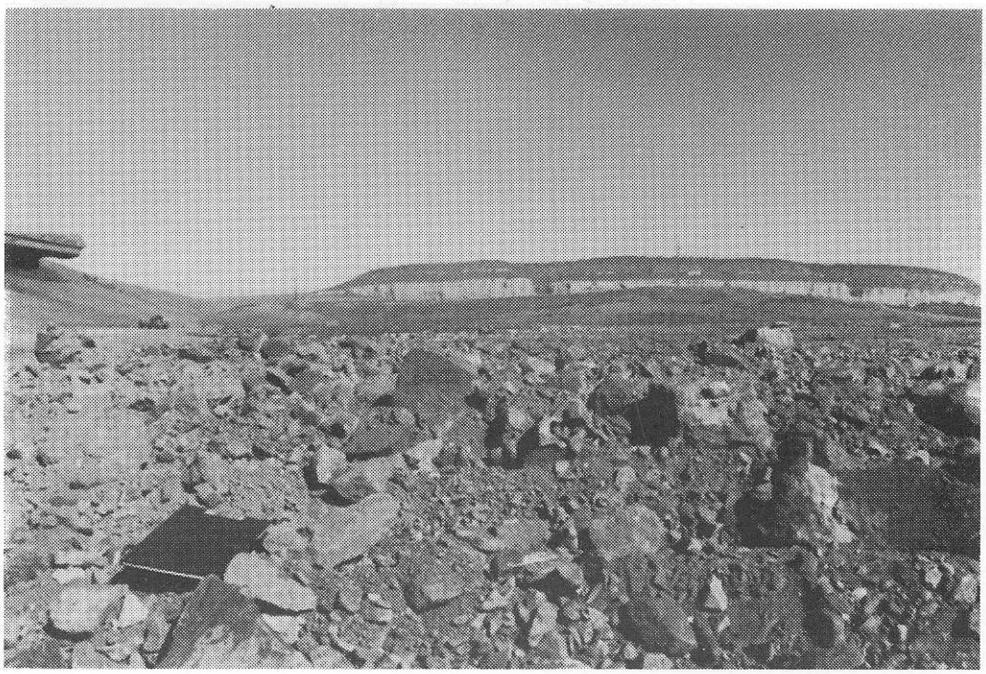


Picture No. 1 - Rock drain material at quarry.



Picture No. 2 - Detail of typical rock fragment.

Picture No. 3 - Placing spoil lift. Lower area.



Picture No. 4 - Spoil lift after spreading.

Picture No. 5 - Drain discharge point.

Picture No. 6 - Overview upper disposal site.

ROCK DRAIN BEHAVIOUR AT BYRON CREEK COLLIERIES SEDIMENTATION POND

by

Frederic B. Claridge, Director
Piteau Engineering Ltd., Calgary

Robert S. Nichols, Engineering Supervisor
Byron Creek Collieries, Sparwood

Willem N. Diddens, Project Engineer
Piteau Engineering Ltd., Calgary

ABSTRACT

During the construction of an onstream sedimentation pond - water supply reservoir at Byron Creek Collieries' mine, it was necessary to construct a rock drain to convey stream and groundwater flows along the valley of Corbin Creek. The drain had to have sufficient transmissivity to conduct all of the creek flow which could not be diverted upstream of the pond, and which entered the valley bottom from springs.

A trench was excavated along the pond, over a distance of 300 m, and filled with select coarse sandstone obtained from the Byron Creek mine. The rock is similar to that which will be placed in the downstream toe of the future East Waste Dump, which will be constructed across the entire Corbin Creek Valley. Hence, the performance of the sedimentation pond rock drain is of significance to the design of a rock drain to be incorporated into the East Waste Dump.

The sedimentation pond rock drain has performed up to expectations and to date, there is no evidence of any decline in the drain transmissivity.

INTRODUCTION

A coarse rockfill drain is presently operating under a synthetically-lined, fresh water reservoir at the Byron Creek Collieries Mine in southeastern British Columbia. The site location is shown on Figure 1. The drain was constructed to provide effective site dewatering during the liner installation and reduce potential post-construction hydrostatic uplift pressures that may affect liner integrity.

This paper discusses the performance of the drain and confirms the high potential flow rates that are possible through coarse rockfill. The experience gained from this case history can be applied to the design of the much larger rock drain, which is to be constructed under the East Waste Dump at the Byron Creek Mine, as well as other mine waste dumps.

BACKGROUND

Project Description

A water storage reservoir and dam were recently constructed in the Corbin Valley, just upstream of the plant site facilities at Byron Creek Collieries, British Columbia, as shown on Figure 2. The earth and rockfill dam is 18 m high and retains an artificially-lined reservoir 35,000 m^2 in area, having a capacity of approximately 150,000 m^3. The pond serves as a supplementary fresh water source for the coal processing operation. The reservoir will also be utilized to collect sediment originating from the upstream East Waste Dump, which will be constructed across the upper Corbin Valley.

The dam, pond and associated drain were constructed during the period of April to early October, 1985; a period characterized by generally poor weather and higher than average creek flows. The full supply level of the reservoir was reached in late October, 1985.

Site Geology and Hydrogeology

The pond is located in the bottom of the broad U-shaped valley of Corbin Creek. Recent alluvial deposits in excess of 6 m in depth predominate along the length of the pond and are relatively coarse grained, ranging from fine sand sizes to boulders averaging 500 mm in diameter. The average hydraulic conductivity of the alluvial materials is in the order of 10^{-5} m/s.

The groundwater table is at, or near, the surface in the reservoir area. Minor springs were also present in the lower part of the valley.

Corbin Creek Flows

The creek flows normally vary from nearly zero in the winter to typically 1 to 2 m^3/s during the spring freshet. Average flows during the summer are usually in the order of 0.02 m^3/s. During the 1985 construction season, creek flows range from 0.03 m^3/s to 0.06 m^3/s.

ROCK DRAIN EXPERIENCE

Purpose of Construction

Prior to initiating construction, Corbin Creek was temporarily diverted into a channel along the west slope of the reservoir. As clearing and stripping operations progressed in the pond, significant groundwater inflows were encountered, resulting in wet operating conditions. Exposure of sidehill seeps also contributed to the surface flows to the point where as much as 0.06 m^3/s was flowing through the reservoir area. Although the alluvial subgrade materials were coarse and pervious, infiltration was minimal as a result of a high groundwater table, as well as the presence of fine grained material introduced by the heavy earthmoving equipment. In order to facilitate lining installation, drainage of the water was required. A rockfill drain was considered as the best alternative to achieve this objective, rather than having to pump the flows from a series of collector sumps along the valley.

Drain Rock Properties

Rock for the drain was obtained from waste stockpiled in Pit No. 12. The ability of a drain to convey flow will depend on the physical properties of the rock and fluid, as well as the hydraulic gradients. The most important physical property is the rockfill gradation, which controls the transmissivity of the placed rock material. Its gradation is, in turn, determined by the susceptibility of the various rock types to degradation by mechanical and, in the long term, by physico-chemical processes. Mechanical degradation applies to rock breakdown during mining and dumping. Physico-chemical breakdown applies to weathering processes (e.g., leaching, slaking, etc.) which may cause further size decay following dumping.

At the Byron Creek mine, waste rock consists of varying amounts of sandstone, siltstone and shale/mudstone. Sandstone and siltstone are observed to break down less than shale and mudstone and are also more resistant to weathering processes. Hence, the former rock types, especially the sandstone, were selected for use in the rock drain.

The relative susceptiblity of different rock types to mechanical degradation during mining and transportation can be evaluated by means of the Los Angeles Abrasion Test, as well as Unconfined Compressive strength tests, or Point Load strength index tests. In general, a Los Angeles abrasion index of more than 40 percent indicates a friable rock, which will tend to decompose into fines during mining and dumping. Abrasion losses from Moose Mountain Formation sandstone were measured in the range of 15-31% (Table 1). This sandstone also exhibits compressive strengths ranging from 80 to 200 MPa, indicating a relatively hard rock which will tend to resist excessive breakdown during mining and dumping.

The potential for physico-chemical degradation can be evaluated by slake durability tests, as well as by freeze/thaw tests. Although the freeze/thaw process is an important factor in long term breakdown of rock near the ground surface, it is unlikely to be a significant factor at depth, either in the rock drain beneath the sedimentation pond, or within a waste dump, where freezing is unlikely to occur. A slake durability index of less than approximately 90% indicates a rock that will tend to break down excessively upon exposure to water. However, as shown in Table 1, the slake durability of source rock for the Byron Creek drain is greater than 98%, indicating high slake resistance.

Rock gradations encompass a wide envelope in practice, reflecting the wide range of material properties and extent of breakdown involved in the mining process. Figure 3 indicates a typical range of gradations encountered in sedimentary rock waste dumps in the Rocky Mountains region. The estimated mean gradation of the rock placed in the drain is also shown on this figure. The approximate mean (nominal) rock size is 300 mm. The corresponding in-place porosity is estimated to be 0.25.

The preferred properties of drain rock which is to be placed into a water course are summarized in Table 1. It can be seen that the drain rock properties exceed the minimum criteria suggested for each category of evaluation.

Water Properties

Seasonal temperature fluctuations will affect water density and viscosity. It is expected that these changes will have no practical impact on flow rates through the drain.

Hydraulic Gradients

Hydraulic gradients along the drain ranged from approximately 0.02 at the outlet to 0.05 at the inlet and averaged 0.04.

Theoretical Drain Capacity

The analysis of through-flow in rockfill is complex and is generally derived by means of empirical relationships. Heterogeneity of particle sizes and the phenomenon of turbulent flow through porous media contribute to modelling difficulties. The basic equation of flow-through rockfill was presented by Leps (1973), which was based on laboratory test relationships developed by Wilkins (1956). This equation, which has been used successfully in past rockfill flow applications by others, is as follows:

$$V_v = W \cdot m^{0.5} \cdot i^{0.54}$$

Where:

V_v = average velocity of water in the voids of the rockfill

W = an empirical constant for a given rockfill nominal particle size

m = mean hydraulic radius

i = hydraulic gradient

Flow-through the rockfill can be calculated from the equation:

$$q = V_v \cdot A \cdot n$$

Where:

q = flow

V_v = as defined above

A = cross-sectional area

n = porosity = $\frac{e}{1 + e}$, where e = void ratio

The average interstitial velocity through the voids, based on an average hydraulic gradient of 0.04, was calculated to be 0.2 m/s. For a cross sectional area of 2 m^2, a flow of 0.1 m^3/s was determined to be the capacity of the drain. Areas of lower gradient would require an increased cross section in linear proportion to the drain cross-sectional area. Although the through-flow capacity of the neighbouring alluvium would augment the flow capacity, the hydraulic conductivity of the alluvium was at least 3 orders of magnitude less than that of the rockfill and, as a result, the component of flow within the alluvium was not considered to be significant.

Rock Drain Section

A typical cross-section showing the design of the rock drain is provided on Figure 4. A cap of selectively graded alluvial material was placed over the rockfill as a filter zone for the overlying liner bedding sand.

Construction of Drain

The drain was constructed along the length of the reservoir, following the sinuous course of the original creek bed and an old waterline installation, as shown on Figure 2. A large (CAT 235) hydraulic excavator advanced the trench to at least the minimum design dimensions. The average cross-sectional area of the trench obtained was 3 m^2, or 50% larger than the minimum cross section required to convey the design flow of 0.1 m^3/s.

Coarse competent sandstone rockfill was brought down from the mine and stockpiled along the trench. The hydraulic excavator then selectively placed the rockfill in the excavation, as shown in Figure 5. The coarsest rock particles were placed on the bottom, with the gradation becoming progressively finer towards the top of the drain. Particle sizes ranged from fine gravel to boulders of up to 1 m in diameter. To ensure a reasonable degree of "compaction", the rockfill was dropped into place from approximately 8-9 m in height, corresponding to the maximum height of the excavator boom. Vibratory rollers would not have been effective due to the coarse nature of the fill. Figure 5 illustrates typical views of the rockfill drain during construction.

A cap of finer rockfill and alluvial material was placed as a transition layer beneath the sand bedding prior to installation of the liner. The final surface was well compacted with a 10 ton, vibratory, smooth drum roller.

Experience During Construction

After the trench excavation was completed, all surface water was redirected into the trench. Backfilling of the trench with coarse rockfill predictably raised the water elevations through displacement, but the water remained well below the ground surface. An average of 0.06 m^3/s was now conveyed by the drain through the reservoir.

In July, 1985, it was decided that the portion of the Corbin Creek flow being diverted around the west abutment of the dam (Figure 5) would be directed into the drain, resulting in a combined flow through the drain in the order of 0.1 m^3/s. Since the capacity of the drain would likely be exceeded in the downstream area of the pond, where gradients were low, pumps would be placed in the rockfill to depress the water level to below the adjacent ground surfece. This modification to the dewatering operation performed successfully for the duration of construction and liner installation. The drain satisfactorily conveyed the combined subsurface and Corbin Creek flows through the reservoir without the need for pumping assistance, except in the lower part of the reservoir, where a 6-inch submersible pump was utilized. There, the hydraulic gradient was less than 0.02 and would have required a drain with a cross-sectional area of at least 5 m^2 to convey the combined surface and subsurface flows. As only 3 to 4 m^2 of the cross-sectional area was available, local ponding developed above the rockfill surface.

During liner installation in September, 1985, a period charactertized by frequent precipitation and high creek flows, preservation of dewatered conditions was essential to prevent floatation of the thin PVC liner, prior to final cover material placement. On one occasion, the pump failed overnight and hence, the lower sections of the drain were subjected to the full flow conditions. The lower sections of the drain, sized for a theoretical flow of 0.10 m^3/s, had conveyed an estimated 0.12 m^3/s. Hence, it can be concluded that the performance of the rockfill drain during construction exceeded the design requirements.

Post-Construction Flows

After commissioning of the reservoir and dam in October, 1985, the drain was left in operation to perform a dual function: (a) to reduce the potential for hydrostatic uplift pressures against the liner; and (b) to provide a supplementary source of water in the event that the reservoir cannot supply all of the demand for water. In January, 1986, Byron Creek staff monitored the flows at the downstream exit of the drain and found that no diminishment in previously experienced construction flows had resulted. Although the Corbin Creek flows had been redirected into the reservoir, in October, 1985, a flow of approximately 0.08 m^3/s was still discharging from the rockfill. As background groundwater levels were low, a leak in the liner system was suspected. Detailed investigations of the reservoir with dyes, divers and remote controlled submersible cameras ensued. A single very localized tear in the liner was discovered directly over the drain, approximately 150 m upstream of the dam. The location of the tear over the drain resulted in a large influx of water. After repair, drain flows dropped from approximately 0.1 m^3/s to a level of approximately 0.06 m^3/s.

IMPLICATIONS FOR WASTE DUMP ROCK DRAINS

Several mine waste dumps are being planned, or are under construction in stream valleys in mountainous terrain in western Canada, as discussed by Claridge et al (1985). Typically, the rockfill drains have been designed to convey substantial flood flows.

At Byron Creek Collieries' East Waste Dump, the flow of Corbin Creek is estimated to be up to 8.8 m^3/s for a 200 year return period. Rather than constructing a rock drain along the entire dump, it was felt to be more efficient to concentrate select coarse rockfill in the downstream toe region. Access is possible from the open pits to the valley floor. The sequence of dumping constitutes what is essentially an "upstream" method of construction in the early stages of dumping. An underdrain and downstream toe containment berm will be constructed in a series of lifts that will assume an overall slope of only 20°. A conservative design for the final face is warranted because of the existence of the on-stream sedimentation pond and plant facilities immediately downstream of the dump. The design of the underdrain and ultimate downstream dump slope is illustrated in Figure 6. For a median rock size of 300 mm, the unit flow capacity of 0.04 $m^3/s/m^2$ (for i = 0.04) estimated for the underdrain was very similar to that measured in the sedimentation pond drain.

The experience from the Byron Creek drain may be cautiously extrapolated to the anticipated performance of rock drains through waste dumps at other sites where the rock materials are similar. A rock drain can be produced naturally from coarse segregation effects of end dumping blasted run of mine rock. The resulting concentration of larger rock sizes will tend to increase the transmissivity of the drain rock above that observed in the Byron Creek sedimentation pond drain. The high through-flow capacity observed in the Byron Creek drain provides confidence that larger drain systems can be constructed with sufficient capacity to convey large flood flows. Over time, as more waste rockfill dumps are constructed and through-flows monitored, it should be possible to refine the predictive model to account for factors such as the actual gradation of the rockfill, turbulent flow conditions and migration of fine grained material through the voids.

CONCLUSIONS

The performance of the rock drain during reservoir construction surpassed expectations. The high transmissivity of the rockfill was illustrated when a pump failure subjected the drain to a full flow condition, resulting in flows in excess of the theoretical design capacity. Approximately 0.12 m^3/s was conducted through the rock drain, as compared to the theoretical capacity of 0.10 m^3/s. Hence, the ability of an engineered rockfill drain to convey a high flow was clearly demonstrated.

On the basis of this case history, the Leps flow equation is considered to be realistic for the gradation of rockfill use in the drain. Close correlations were observed between the predicted flows and those observed in the drain.

The drain experience at the Byron Creek project has demonstrated the potential high through-flow capacity of small rockfill drainage systems under relatively low head conditions. The drain may also be considered as a performance model for much larger systems, such as rock drains through waste dumps, where through-flow capacity is of primary concern.

ACKNOWLEDGEMENTS

Appreciation is extended to the management of Esso Resources Canada Limited, for permission to prepare and submit this paper. Esso's managerial and technical staff who were most extensively involved with the reservoir and drain construction include Mr. M.E. Maleod, Mr. R.F. Broom and Mr. M.C. Tressler. The review provided by Dr. T.L. Dabrowski of Piteau Engineering Ltd. is also gratefully acknowledged.

REFERENCES

Leps, T.M. 1973. "Flow Through Rockfill", Embankment - Dam Engineering (Casagrande Volume), J. Wiley & Sons.

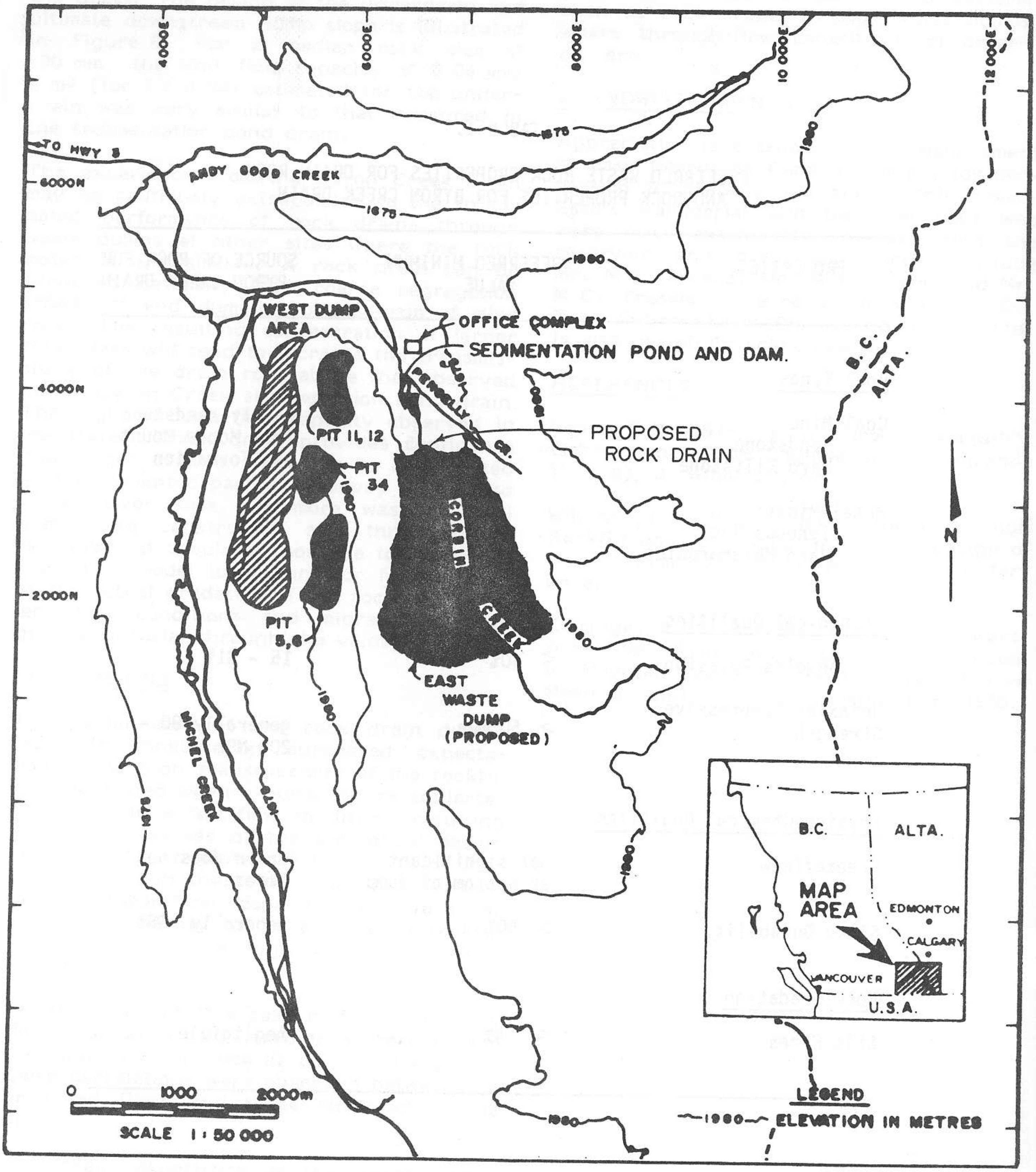
Wilkins, J.K. 1956. "Flow of Water Through Rockfill and its Application to the Design of Dams", Proc. 2nd Aust.-N.Z. Soils Conference.

Claridge, F.B., Nichols, R.F. and Stewart, A.F. 1985. "Mine Waste Dumps Constructed in Mountain Valleys". Second District Five Meeting, Hinton, Alberta, September, 1985.

TABLE 1.

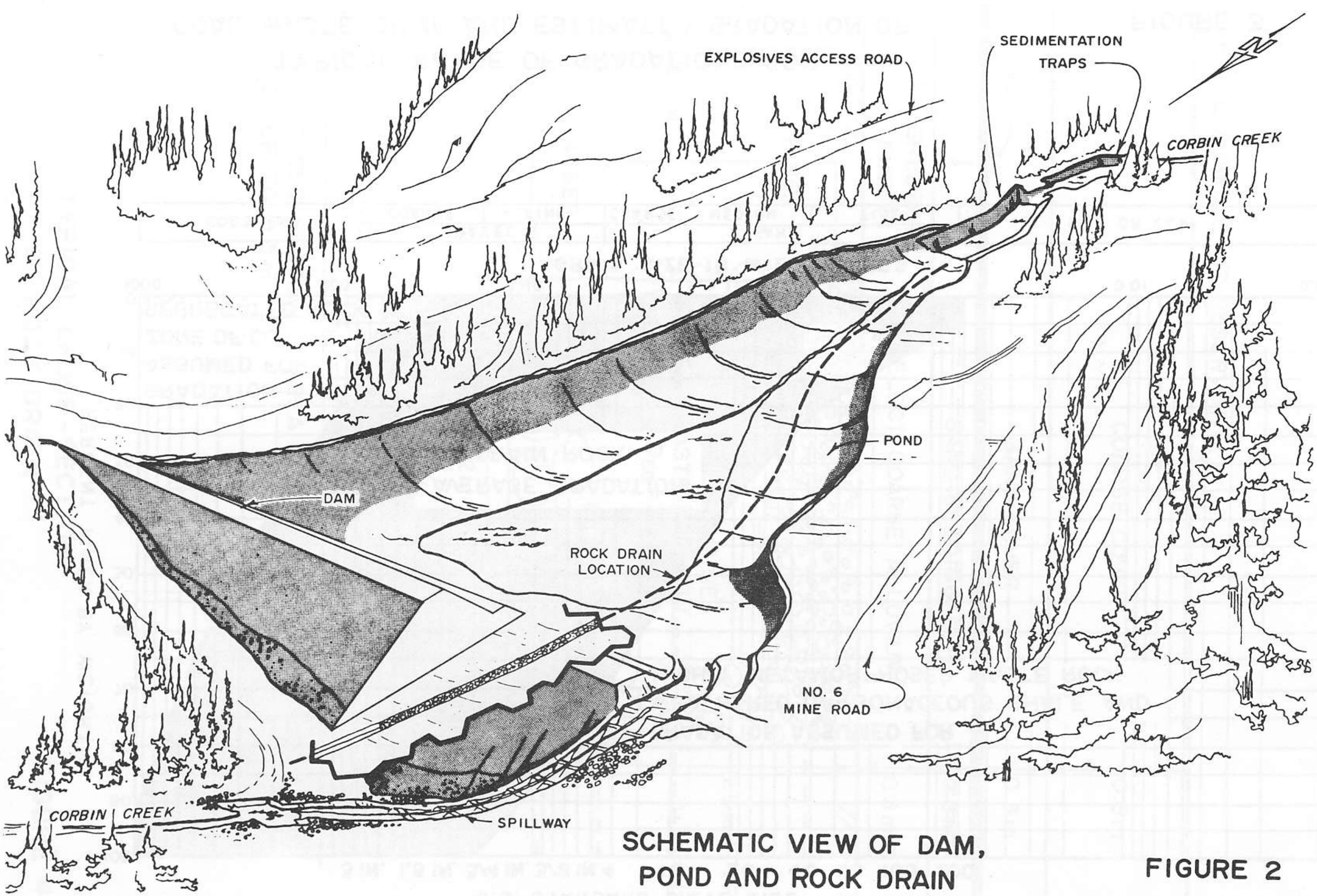
PREFERRED WASTE ROCK PROPERTIES FOR DRAIN ROCK
AND ROCK PROPERTIES FOR BYRON CREEK DRAIN

PROPERTIES	PREFERRED MINIMUM VALUE	SOURCE OF ROCK FOR BYRON CREEK DRAIN
Rock Types		
Coal Mines: Sandstone, Hard Siltstone		mainly sandstone from Moose Mountain Formation
Metal Mines: Igneous Rock, Hard Metamorphics		
Mechanical Qualities		
Los Angeles Abrasion	< 40%	15 - 31%
Uniaxial Compressive Strength	> 50 MPa	generally 80 - 200 MPa
Physico-Chemical Qualities		
Freeze/Thaw	not significant at bottom of dump	drain does not freeze
Slake Durability	> 90%	generally 98%
Rock Gradation		
Silt Fines	< 5%	negligible



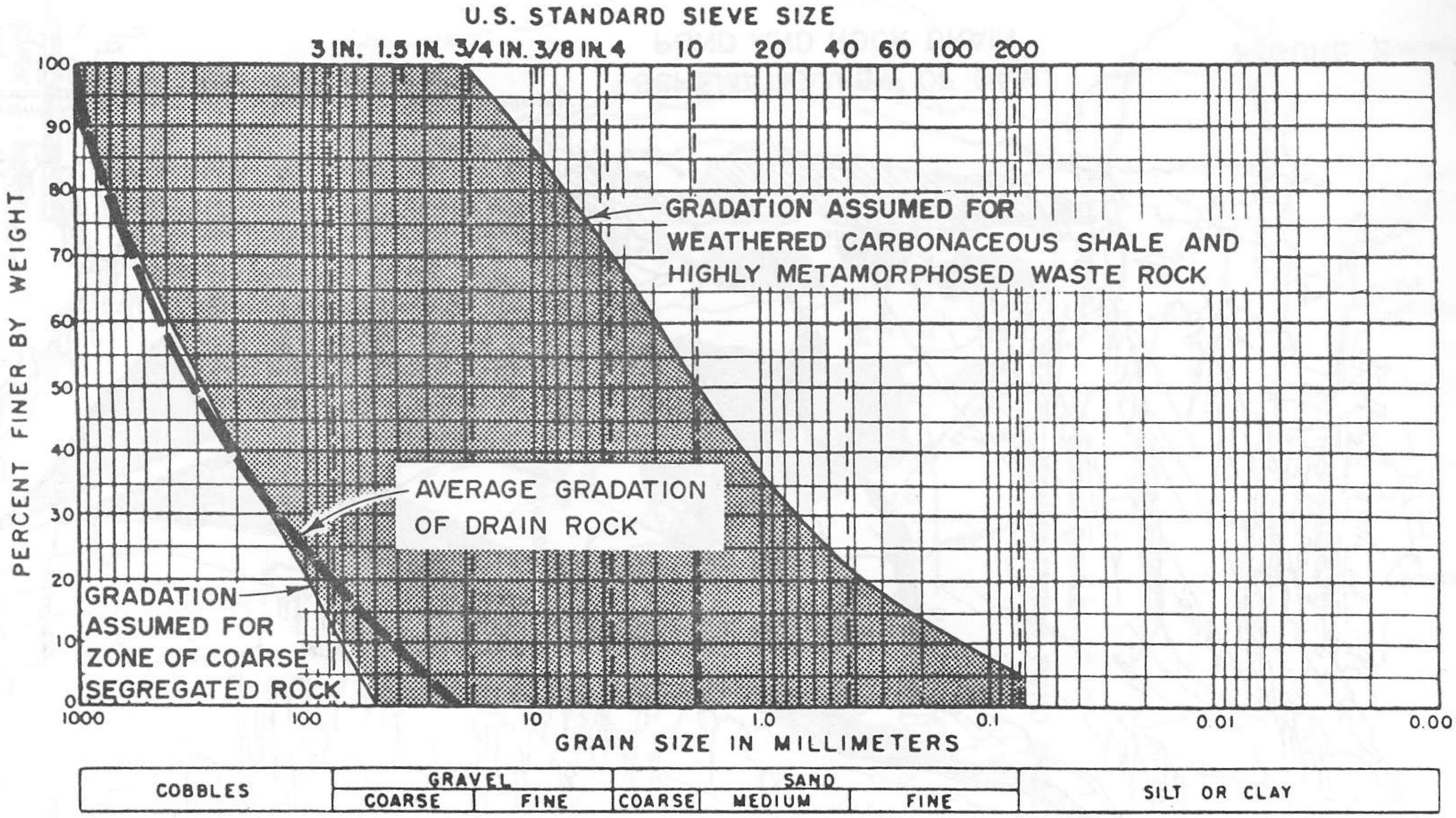
LOCATION PLAN

FIGURE I



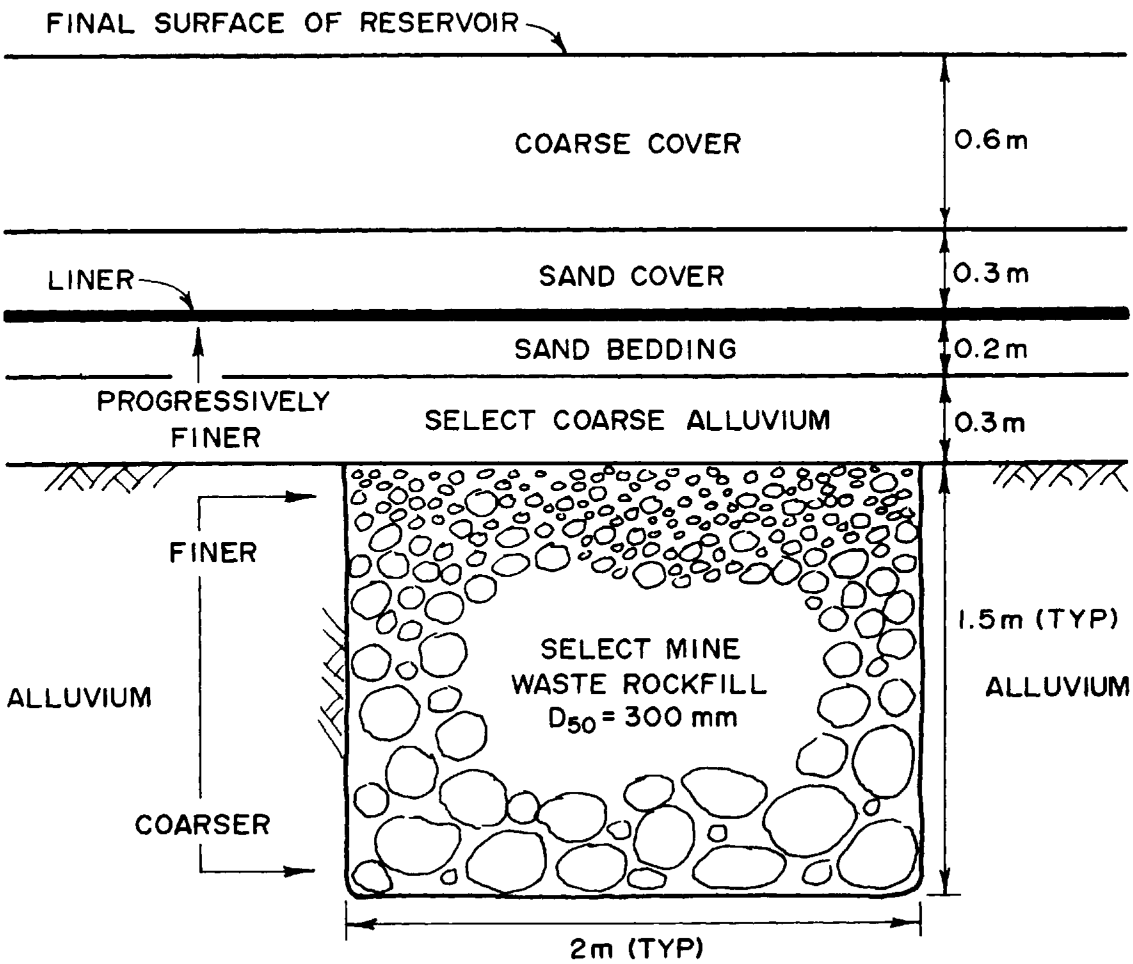
SCHEMATIC VIEW OF DAM, POND AND ROCK DRAIN

FIGURE 2



TYPICAL RANGE OF GRADATIONS FOR
COAL WASTE DUMP AND ESTIMATED GRADATION OF
SEDIMENTATION POND DRAIN ROCK

FIGURE 3



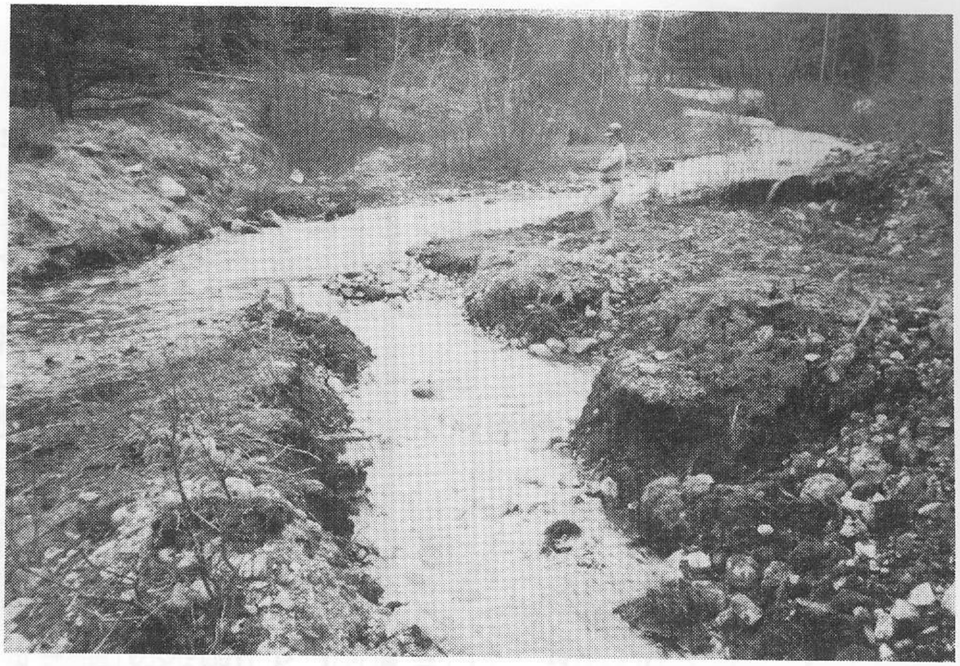
NOTE :

AVERAGE CROSS-SECTIONAL AREA IS 3m . DEPTH AND WIDTH MAY VARY BY UP TO 0.5 m.

TYPICAL CROSS-SECTION OF ROCK DRAIN

FIGURE 4

Excavation for rock drain collector along west slope of reservoir.



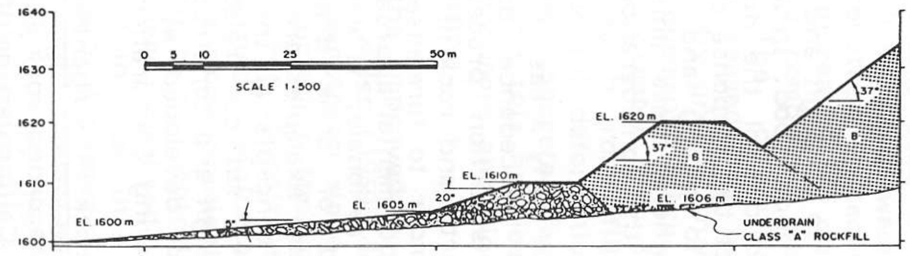
Corbin Creek flow being diverted along west side of reservoir (fork at bottom of photo). Flow was subsequently directed into rock drain.

Rock drain consisting of coarse sandstone. Subsequently capped with minimum 300 mm thick finer gradation Class A rockfill.

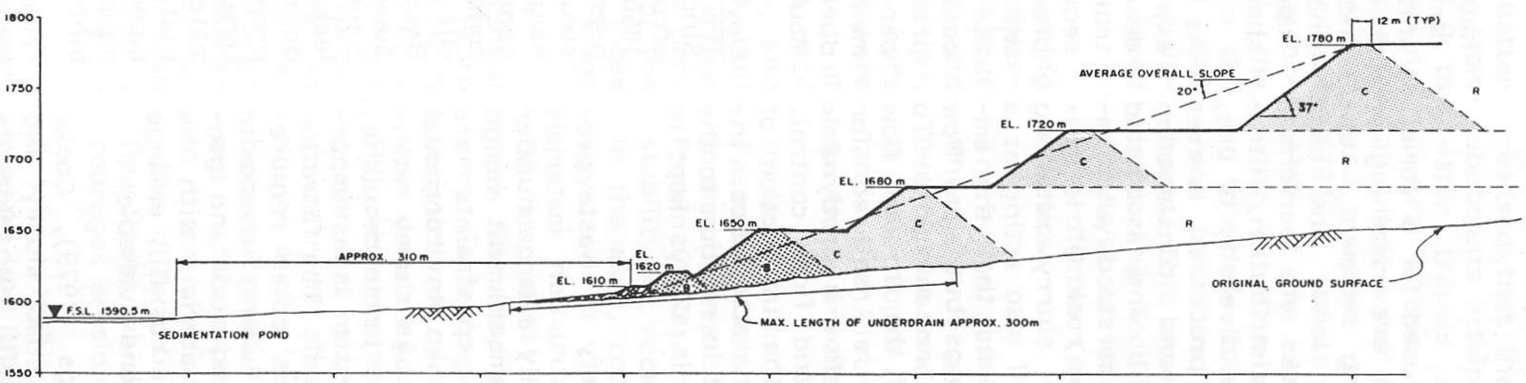
Figure 5. Site Photos

SPECIFIED GRADATIONS
FOR SELECT ROCKFILL

ROCKFILL ZONE	ROCK TYPE	DIMENSIONS MINIMUM NOMINAL SIZE	MAXIMUM 30% FINER THAN	MAXIMUM 10% FINER THAN	APPROXIMATE QUANTITY (m^3)
CLASS A AND UNDERDRAIN	SELECT SANDSTONE / LIMESTONE	400 mm	100mm	40 mm	50 000
CLASS B	SELECT SANDSTONE AND SILTSTONE	200 mm	50 mm	20 mm	820 000
CLASS C	ALL ROCK EXCEPT DEGRADABLE MUDSTONE, SHALE AND SOIL OVERBURDEN	100 mm	25 mm	10 mm	8.5 MILLION
CLASS R	RANDOM ROCK	AS OBTAINED FROM MINE			313.6 MILLION



TYPICAL SECTION OF UNDERDRAIN



LONGITUDINAL SECTION THROUGH
DOWNSTREAM FINAL SLOPE

BYRON CREEK COLLIERIES - EAST WASTE DUMP
DESIGN OF FINAL DOWNSTREAM SLOPE USING SELECT DURABLE ROCKFILL

FIGURE 6

ROCK FILL DAMS IN HANDLING MINE WASTE MATERIAL

by

B.A. Chappell

INTRODUCTION

Rockfill embankments are used for a variety of tasks, some of which are retaining or draining water, Impounding sediment, supporting loads, reclaiming land, and mine waste disposal. These tasks are performed by the processes of load distribution, filtration, and water flow. This diversity of use, coupled with the different processes, is ever widening as knowledge and construction techniques, using rock fill and associated materials, are better understood and improved. In this paper, the rock fill is considered a water and or slurry retaining structure with the rock fill also acting as a service spillway. This means that the embankment must retain tailings but allow flow through and over the embankment. To achieve this, knowledge of the through flow hydraulic characteristics is required for slurry retention, and over flow hydrodynamic characteristics for river and flood control. In addition to defining the interaction of these two flow regime characteristics, the stability of the embankment in relation to the foundation and abutments, is always kept in mind.

Because of the wide variety of waste products produced and construction materials available, there are generally a large number of constraints imposed on embankment design and construction. These constraints are added to and intensified when environmental considerations are included. As slurry retention and drainage is also a prime requisite, the design of the filter system is as important as the embankment itself. The filtration and drainage provisos plus other requirements are facilitated by using composite materials, such as reinforced rock and geofabrics. These composite materials with the range of rock types used as rockfill, enlarge the number of design solutions available.

It is now known, Jennings (1979), Cooke (1984), that both the retained slurry and dumped or compacted rock fill embankments are anisotropic. These anisotropies, though difficult to measure require quantification, if the mechanisms controlling embankment behaviour related to through and or overflow dams are to be understood. In addition, the anisotropies also control the design of the filters and effect the response of the slurry and rock fill to piping and liquefaction. Another area where these anisotopies are important is in the consideration of seismic shocks.

As is the case in many areas of engineering, experience and precedence dominate the design and construction process. This is true of both earth and rockfill dams. Consequently, in order to formalise the design process, the experimental approach, coupled with theoretical rationale, is examined with the background of some experiences in construction. Some examples are used in the presentation of concepts at the end of the paper. The work set out relates, in the main, to the design and construction procedures used and developed at the Snowy Mountain Engineering Corporation.

Development of rock fill embankments

Since the first recorded rock fill dams constructed on the Californian goldfields in the 1850s, eg. French Lane dam 1859, there has been slow but steady progress in the construction of dumped rockfill dams. The height of these dams up to 1960 were generally below 60 m and the impermeable zones were made up of materials such as silt-clay earth, concrete, wood, and steel. Because of excessive settlements inherent in dumped and sluiced rock fill, many of the concrete faced rock fill dams cracked and leaked copiously. These excessive rock fill deformations and the economics of the project made, at this stage, the earthfill core the most widely used impermeable zone material. It is important to note, however, that though many of the higher concrete faced rock fill dams cracked and leaked, none of them failed from embankment instability.

From 1960 on, there were considerable improvements in construction techniques, equipment, and better methods of rock fill quarrying. Instead of just dumping and sluicing the rock fill in 10 to 30 m lifts, the rock fill was set out in 0.9 to 2 m layers and compacted with 3 to 6 passes of a 4 to 10 t smooth wheeled vibrating roller. The moisture content used in the compaction of the rock fill varies between 4% to 12%. These improved construction techniques with better machinery increased the dumped density from 0.9 t/m^3 to compacted densities of 1.9 to 2.1 t/m^3. The concomitant deformational stiffness increased from less than 11 Mpa to be consistently greater than 80 Mpa. Initially, these improvements were used to increase the height (greater than 300 m) and performance of earth rock fill dams. With the reduced rockfill deformations, cracks in the impermeable concrete face were all but eliminated and this has led to the construction of many more and higher concrete faced rock fill dams.

Prior to the improved rock fill compacting techniques, Weiss (1950) described the overflow and downstream slope protection of four dams constructed in Mexico in the 1930s and early 1940s. For downstream slope protection, grid patterned reinforcement steel bars protecting a slope gradient of 1 in 1.4 were used. Though no controlled compaction of the rock was formalised, the dumped rock was laid out in small lifts, 1 m to 3 m, and trafficked with 4 passes of tracked, D8, bull dozer. Though overtopped, with overflow depths up to 3.5 m, the rock fill dams performed very well indeed with only one of the dams being slightly damaged. All these protective measures were at this stage temporary.

Wilkens (1956) describes the developmental work performed by the Tasmanian Hydro Electric Commission in constructing the 12 m high Laughing Jack marsh dam with a through flow spillway rock embankment. This was the first time through flow was considered a permanent feature of the dam. In 1963, Wilkens sets out the added experiences in Tasmania gained from the use of through and over flow embankments. These embankments, as were those in Mexico, were protected with reinforcing bars and steel mesh on the downstream rock slope which had a gradient of 1 in 1.4, and rock size up to 0.45 m.

Many important and invaluable lessons came out of these experiences in constructing the through and overflow rock fill dams in Tasmania, Wilkens (1963), Fitzpatrick (1977) and Bowling (1980). Two important requirements were the necessity of protecting the rockfill toe, and the need to prevent damage of the protective mesh caused by rock and log debris slamming the downstream slope. In addition, it was soon realised that the overflowing water near the abutments attains higher velocities while the water travels along the toe towards the central stream. Because of this, the surface abutment water has a high erosive capability and causes considerable damage, Pells (1978), unless specifically designed for. Also, if the crest of the dam is incomplete when overflow occurs, flow is concentrated in channels causing rilling and the eroded rock from the crest slams the downstream mesh. This means that prior warning of the impending flood is essential so that the rock fill layer or lift is complete and ready, that is protected (meshed) before accepting the overflow.

There are now many examples in Australia where reinforced rock fill has been safely overtopped to depths up to 2.5 m. Googong in New South Wales, Borumba in Queensland, Ord River and Serpentine in Western Australia and many more.

In conjunction with the work carried out in Tasmania, the University of Melbourne performed research work, Sandie (1961) and Parkin (1963), related to the hydraulics and stability of rock fill. This work examined, in the main, rockfill dams with inbuilt spillways.

Olivier (1964) performed experiments on rock fill model embankments and produced a very useful paper on the hydraulic characteristics of unreinforced through and over flow dams. Hydraulic stability of these rock fill embankments, in the main, relied on the grade and size of rocks making up the downstream slope. Many countries such as South Africa, India, Australia, and others have successfully applied the principles set out in this paper. One of the main problems in these unreinforced embankments is possible differential settlement along the over flow crest which concentrates flow and encourages erosion.

Sparks (1967) examined and defined the erosion and sloughing characteristics of the downstream slope. Once the hydraulic and hydrodynamic forces, which were not really quantified in this paper, were found, classical slip circle analyses were used to determine the reinforcement required to stabilise the embankment from sloughing. The forces in the mesh reinforcement were unduly small and unrealistic, ie 7 Kpa for the case of Xonxa dam in South Africa, Shand and Pells (1970).

Though much value is obtained from the above research work, it is the field experiences and techniques which give the precedence on which future works are designed and constructed. New techniques and materials such as geofabrics and reinforced earth have given greater scope to the design and required stage construction of waste control rockfill dams. However, in order to optimise the use of these materials in design and construction, an understanding of their behaviour is obtained by applying rational theory coupled with available experimental information. Added to this are the experiences obtained by using these materials in prototype structures.

Slurry and associated embankments

Slime dams are such that generally the embankment is made up of the waste product in the form of hydraulic fill, Gowan (1980). The question of whether to use downstream, centre line, or upstream embankments is a function of the waste product being handled and land topography available. With the new found requirements of environmental awareness and safety, the need to engineer and manage the problem of mine waste disposal is essential. Because of this, earth and rock fill embankments are now part of the mine waste disposal scene. The material used for constructing these embankment dams varies from a clay to rock with associated reinforcement, and drainage materials.

Irrespective of the material used to construct the embankment, drainage of both the embankment and slurry fill are prime factors. When rock fill is used for the embankment, retention of the waste product is just as important as drainage of the waste product itself. Waste product and embankment drainage here means that water is draining from the waste into and through the rock fill. When water is also flowing through and over the embankment from surface water runoff, both drainage and hydrodynamic forces are now acting on the embankment. In order to retain the tailings, a transition filter and drainage zone is required between the slurry and rockfill. As the flow in the smaller voids of the slurry is laminar and that in the larger voids of the rockfill is still lower, there are no induced instabilities. That is the major part of the hydraulic gradient resides in the slurry and drops to atmospheric on entering the rock fill, Parkin (1963), Figure 3.

If the rock fill is soft rock, which breaks down on compaction, Mackenzie and McDonald (1980), special drains are required to ensure adequate drainage and reduction of pore pressure. This soft material (friable sandstone and shale) is generally not suited to resist the hydrodynamic forces of over and through flow yet alone the through flow drainage. If soft rock is used as a water retaining structure, special drainage layers are designed and included in the embankment.

No matter what type of rock fill is used, when assessing embankment behaviour, it is important to recognise and include the induced anisotropies. In the tailings, hydraulic settlement causes particle size differentiation and separation with the resultant material laminations and anisotropy. This anisotropy in material size and permeability increases the potential of the tailings to pipe, erode, and also possibly liquify, (Jennings 1979).

In designing the transition zone for filtration and drainage, the filter requirements defined by Giroud (1981) and Sherard (1983) are used. These criteria, in the main, corroborate those set out by Terzaghi but with the added conditions of defining the minimum pore size related to the filter particle size represented by D_{15}. Where D_{15} is the sieve size which allows 15% by weight of the sample being tested to pass. With knowledge of the minimum pore size, the size of particle which can pass through this minimum pore size is defined. Criteria such as hydraulic gradient and boundary electro kinetics have an effect but under high hydraulic gradients, the relative pore size is the controlling factor. Sherard (1983) found that D_{10} and D_{20} sieve sizes for the filter material being examined could alternatively represent the average pore size of the filter material as does D_{15}. The minimum pore space allowing the possible passage of a particle, Sherard (1983), varies as:

d base = (0.11[min] to 0.6) D_{15} filter

Where d base is the particle size which passes through the pores of the material,

D_{15} filter is the equivalent sieve size of a particle of which 15% passes that sieve size.

The smallest pore size opening is the one which constrains the movement of a particle. Therefore:

D base = 0.11 D_{15} filter

is the base particle size which can pass through the filter.

This relation was found to apply not only to uniformly graded filters but also well graded filters with Cu ranging up to 12. Cu is the uniformity coefficient which is a measure of the range of grading and is the ratio D_{60} / D_{10}.

For base materials from exceptionally fine clays with d_{85} less than 0.02 mm, a filter with D_{15} equal to or less than 0.2 mm is required. For a fine grained silt of low cohesion, a filter with D_{15} equal to or less than 0.3 mm is required. For fine grained clay with d_{85} from 0.1 to 0.3, a filter with D_{15} equal to or less than 0.5 mm is required. With these added conditions in mind, internal erosion or piping of the slurry through the rock fill is inhibited if the filtration criterion, namely

$$\frac{D_{15}\text{ filter}}{d_{85}\text{ base}} < 5 \quad \text{---------} \quad (1)$$

is satisfied. This has an inherent factor of safety of approximately 2.

Though Sherard (1983) negates the usefulness of other filter criteria, it is felt that the requirement for drainage is fundamentally as important as filtration. The requirement of drainage not only controls the movement of water but, more importantly, it controls the magnitude and distribution of pore pressure. The drainage or permeability criterion generally used is,

$$\frac{D_{15}\text{ filter}}{d_{15}\text{ prototype}} > 5 \quad \text{----------} \quad (2)$$

For granular filters as defined above, Sherard (1983) found that the permeability k in cm/sec varied from 0.2 $(D_{15})^2$ to 0.6 $(D_{15})^2$ with an average of 0.35 $(D_{15})^2$, where D_{15} is in mm. With these permeability relations and drainage stipulation (2), the permeability criterion is:

$$(D_{15})^2\text{ filter} > 25 \times (d_{15})^2$$

therefore

$$k\text{ filter} > 25 \times k\text{ base}$$

With a safety factor of the 2 the permeability of the filter is at least 10 times that of the base. If the tailings are gap graded, then the individual peaks in the base grading curve must meet the above criteria separately. This particular aspect of filtration and drainage still requires better definition.

<u>Geotextile filtration and drainage criteria</u>

With the development of geofabrics, more flexibility in the design and layout of filtration and drainage transition zones is readily achieved, Figure 1. Caution is essential because the design parameters for geofabrics are not, as yet, standardised. Much work is still required to define the criteria for turbulent and or reverse flow. Giroud (1981) sets out the design criteria for geotextiles and highlights the need to consider both the retention and drainage aspects of the geotextiles.

<u>Filtration</u>

Nonwoven geotextiles, such as Bidem, have been widely used by the Snowy Mountain Engineering Corporation. The retention ability of a geotextile is governed by its largest opening which is characterised by the O_{95} apparent opening size. If the base soil is considered cohesionless, the retention capability of the geotextile is primarily a function of the coefficeint of uniformity, Cu, relative density, Id, and equivalent base particle size d_{50}. Table 1 gives the retention criteria given in terms of the aforementioned parameters.

The average size of the base soil is taken as d_{50}. This is really not a good parameter as it is dependent on the particle size distribution of the base soil and not necessarily on the coefficient of uniformity.

TABLE 1

	Relative Density	Coefficient of Uniformity	
		$1 < C'u < 3$	$C'u > 3$
Loose soil	$Id < 35\%$	$O_{95} < C'u\ d_{50}$	$O_{95} < \frac{9}{C'u} d_{50}$
Medium dense soil	$35\% < Id < 65\%$	$O_{95} < 1.5\ C'u\ d_{50}$	$O_{95} < \frac{13.5}{C'u} d_{50}$
Dense soil	$Id > 65\%$	$O_{95} < 2\ C'u\ d_{50}$	$O_{95} < \frac{18}{C'u} d_{50}$

Drainage or Permeability

The permeability criterion given by Giroud (1981) for the imposed hydraulic head as shown in Figure 2(a), which is perpendicular to the plane of the filter, is:

$$\text{permeability of filter } k_f > \frac{\text{permeability of base soil } k_s}{10}$$

By considering the geometry of the base soil and filter, it is noted that this imposes an anisotropic features on the base soil and filter, Figure 2(a) and (b). For example, if the imposed hydraulic head is as shown in Figure 2(b), the drainage requirement is:

$$\text{permeability of filter } k_f > 10 \times \text{permeability of base soil } k_s$$

This is the drainage requirement for granular filters.

Through and Over Flow Embankments

Experience at Mangrove Creek dam has shown that a wide range of materials, namely soft rock such as shale and crumbly sandstone, are suitable for use as rockfill. When, however, this soft rock breaks down on compaction, the permeability of the rock fill is considerably reduced, with permeabilities down to 10^{-3} cm/sec in the horizontal direction and 10^{-5} cm/sec vertically Mackenzie and McDonald (1980). Special drainage blankets, such as selected hard siltstone and or basalt layers, were required to ensure drainage and reduction of pore pressures. Consequently, if the rock breaks down on compaction, this negates the free drainage aspects generally assumed for rock fill and its implications must be carefully examined. In addition, the resistance of soft rock to both internal and external erosion is suspect. Because of this, a minimal strength requirement of the rock is often specified which is generally about 20 MPa for the unconfined saturated compressive strength.

There are two ways of controlling the stability characteristics of a through and over flow embankment, they are:

1. reinforcement of the downstream slope with gradients of 1 in 1.4 to 1 in 2,
2. grade the downstream slope from 1 in 3 to 1 in 10 according to the magnitude of flow and size of armouring rock.

1. Embankment controlled by downstream slope reinforcement

Weiss (1950), Wilkens (1963), Shand and Pells (1970) all describe their experiences with reinforced rock embankments which were overtopped (up to 2.5 m) with considerable flows of water. Wilkens (1957) from a series of laboratory models and experiences in the field, defined some useful design criteria. For stone with equivalent diameters up to 8.0 cm, the semi rational velocity function was determined as:

$$V = K\ m^{0.5}\ i^{0.54} \qquad \text{------- (3)}$$

where

V = velocity in m/sec

m = hydraulic mean radius = $\dfrac{e}{\text{Surface area/unit volume}}$

i = head loss gradient

K = parameter representing particle shape and roughness 18.16 (rough) and 25.7 (smooth) rock

V is defined by Q the hydrological design flow discharge and the variable crest length. This also gives q which is the flow per unit width.

When there is a sloping spillway, Figure 3, there are two control depths, Parkin (1983); one is the crest height, hc, and the other is the downstream exit height, he. From these control depths and using open channel gradually varied flow formulae, the upstream and downstream water profiles are readily determined.

From model studies, Sandie (1959) and Parkin (1963) found q proportional to hc, where hc is determined on the assumption of horizontal velocity at the crest and an energy gradient of 0.8. The exit control point, the depth he, was found on a semi rational basis to be:

$$h_e = q \, \frac{1+e}{e} \left(\frac{a}{\sin B} \right)^{1/n} \qquad \text{---------- (4)}$$

where

- B = downstream slope gradient
- l/n = 1.86
- q = flow per unit width
- e = void ratio
- a = 0.004

From the above equations and using open channel formulae, the phreatic or flow surface is determined. Then using continuity and orthogonality of flow and pressure lines, a flow net diagram is drawn and hydraulic pressures evaluated. These hydraulic forces are superimposed on the gravitational forces used in slope stability. Up to now limit analysis does not generally recognise anisotropy, yet this is an important factor controlling the deformational response and strength of the embankment. Computers now offer realistic opportunities of including these anisotropies in analyses; however, this requires the quantification of these anisotropies.

Wilkens (1963) used, after a series of model studies, a plastic limit wedge analysis rather than slip circles to determine the required length of reinforcing bars. Sparkes (1967), Shand and Pells (1970) used slip circle analyses to design the length and layout of reinforcement for the Xonxa dam. In a later paper, Brown and Pells (1983) used an isotropic elastic finite element programme to reanalyse the Xonxa dam. It was found that the reinforcement loads from the finite element programme were 70% of the reinforcement loads found when using circular slip analysis.

At this stage, the surface and form drag aspects of hydrodynamic forces as opposed to hydaulic forces have not yet been considered. The drag and plucking forces erode the surface rock from the down stream slope, especially in the area of the toe. In order to circumvent this problem, a surface mesh covers the section of the downstream slope over which water flows. Quantification of these forces in relation to the mesh requirements is poorly understood. Leeder (1982) studying the lift forces on sediments in river beds found that plucking forces greater than 100 KPa are readily mobilised. For example, if the water velocity is 3.3 m/s around a pebble 5 cm equivalent diameter, then the uplift force is 100 KPa. Nevertheless, experience has shown that if these meshes remain intact, that is they are not broken by debris or underscoured, then the stability of rock fill embankment is well nigh assured and secured.

Antisotropic deformational response of embankment

An understanding of the interaction between the reinforcement, and earth or rocks surrounding it, is greatly improved if the work performed and experience gained on reinforced earth are examined. It is important to realise, however, that when it comes to assessing the response of reinforced rock fill to load, there is still much to be done.

In a reinforced vertical embankment the relative deformation between the reinforcement and surrounding material (earth or rock fill) develop two zones, Figure 4, namely the active and resistant zones. In the active zone, the interactive shear forces are directed towards the face while in the resistant zone, the shear forces are directed towards the interior of the embankment. This results in the maximum tensile reinforcing force occurring approximately 0.3 H (H is the height of embankment) from the vertical surface as shown in Figure 5, Schlosser (1981). In reinforced rock fill, the interactive shear force between the reinforcing bars and surrounding rock is not as definitive as it is with fine granular material, such as earth or sand. To obviate this uncertainty, the reinforcing bars are generally bent to that a significant volume of the rock mass is constrained, Figure 6. This constraint gives the rock mass en apparent cohesion which added to that caused by compaction, greatly increases the stiffness and strength and enhances stability.

With the embankment now constructed, there are two imposed or induced anisotropies. One is from the placement and compaction of the rock fill layers, and the other is from the reinforcement. With the superimposition of the through and over flow water, hydraulic and hydrodynamic loads are then applied to the downstream slope which will respond anisotropically to both deformation and permeability.

Just as material anisotropy controls the deformational response so does it relate to the anisotropy of strength and permeability. These anisotropies introduce inhomogeneous stress distributions or gradients causing the mechanism of rotation which in itself is a prime stress redistributer, Chappell (1986).

The behaviour of reinforced earth does not agree with that behaviour predicted by Rankine's wedge limit analyses, Schlosser (1978). Model and prototype tests have shown that the behaviour of the reinforced earth wall is a function of the interactive shear force between the earth and reinforcing member. When sufficient interactive shear stress is mobilised, the reinforcement either slips within the earth or breaks if enough tensile stress is generated within the reinforcement. These mechanisms are dependent on the stress distribution and redistribution within the enclosing soil and reinforcement which, in turn, are functions of the anisotropic material.

These anisotropies are readily quantified. For example, a 1 m layer of rock fill has a vertical and horizontal deformational modulus of 50 MPa and 150 MPa respectively. 5 cm diameter mild steel reinforcing bars are placed at 1 m vertical and 0.25 m horizontal spacing. These bars acting with the anisotropic rock fill, Figure 6, accentuate the anisotropy as shown, if:

Relative volume of rock material vertically $L_r = 0.99$

Relative volume of reinforcement vertically $L_s = 0.01$

Relative volume of rock material horizontally $A_r = 0.99$

Relative volume of reinforcement horizontally $A_s = 0.01$

Deformational modulus of rock fill vertically $E_{rv} = 50$ MPa

Deformational modulus of rock fill horizontally $E_{rh} = 150$ MPa

Deformational modulus of mild steel $E_s = 200$ GPa

The directional aspects of the anisotropy are depicted in Figure 6. Using composite theory, Chappell (1986), it is readily shown that the vertical composite modulus is a lower bound value, Evc, of stiffness and is:

$$\frac{1}{E_{vc}} = \frac{L_r}{E_{rv}} + \frac{L_s}{E_s}$$

which gives $E_{vc} = 101$ MPa

and the horizontal composite modulus is an upper bound value, Ehc, of stiffness and is:

$$E_{hc} = A_r E_{rh} + A_{sh} E_s$$

which gives $E_{hc} = 2.2$ GPa

2. Embankments controlled by <u>downstream slope and rock size</u>

Olivier (1967) performed a series of experiments with dumped rock where initially he separated and examined first over flow and then through flow. A combination of the two flow regimes was examined qualitatively. The slopes examined were 1 in 5, 1 in 7.5, and 1 in 10. For the through flow experiments, a sloping face spillway was used, Figure 7. Under these conditions free fall occurs and the internal stability of the rock embankment is not effected, Parkin (1963). At the point of exit, however, erosion of the surface rocks is possible if the size of the rock is too small and or the slope gradient too steep.

In the over flow study Olivier (1967) rationalised the formulations of overflow in terms of qr, namely the overflow per unit width, as:

$$q_r = K\ 11.84\ d_s^{3/2} \left[\frac{ws - w}{w} \right]^{5/3} i^{-7/6} \qquad \text{---------}\ (5)$$

where

- K is a dimensionless factor which is a function of grading and packing of the downstream rocks
- q_r = flow per unit crest width
- d_s = equivalent diameter of rock
- h = depth of flow over crest
- w_s = density of rock
- w = density of water

The gravitational shear force tc acting on a stone or diameter d_s, is:

$$t_c = 0.667\ ds\ (ws-w)\ \cos\theta - \tan\phi) \qquad \text{---------}\ (6)$$

where

- ϕ = angle of repose of rock fill
- θ = angle of inclination of bed to the horizontal

The exit height or depth, he, of the through flow water was found by Olivier (1967) to be:

$$h_e = qr\ \frac{(1+e)}{e}\ \frac{i^{7/6}}{C(\sin a)^b} \qquad \text{----------}\ (7)$$

where

- e = void ratio
- a = angle of inclination of downstream slope to horizontal.

Midgley (1979) used this approach and formulations to design and construct a slurry retaining embankment at Bafokeng, Figure 8. From the above equation (5) and K, for a packed angular material, is 0.235 and a downstream slope of 1 in 3, and equivalent diameter rocks, the discharge is q_r = 0.59 m/s/m width. As the flow is free flow over the crest, the depth of flow is evaluated from the equation:

$$h_c = \frac{q_r^{\ 2}}{g} = 0.33\ \text{m}$$

Rock material and reinforcement

Blight (1969) noted that when a rock fill dump fails, the cause of failure is associated with the condition of the foundation near the toe of the dump. (When, however, a slimes dam fails, the failure is generally associated with the material making up the slimes dam.) With water passing through and over the rock embankment, the stability criteria are changed and the rock is prone to failure. Either erosion (plucking) and or sloughing (liquefaction) are now major factors effecting the stability of the embankment. The main difficulties, as are generally well appreciated, are the choice and measurement of parameters representing the rock material. This is exacerbated by the anisotropic characteristics of the rock fill. Techniques and developments are such, however, that these parameters are now being defined and measured.

When compacting rock fill in 0.9 to 2.0 m layers, segregation occurs, Figure 9. As the top section receives most of the compactive effort, the top material is broken down accentuating the anisotropy of permeability and deformation. From the results of embankment monitoring, the deformational modulus in the horizontal direction is generally 3 to 5 times that in the vertical direction, Cooke (1984). From personal experience the horizontal permeability is approximately ten times the vertical for a coarse reasonably graded rock fill (Cu=3 to 12). Much work is still needed to quantify the moduli variation with confining stress.

With compaction and reinforcement, the rock pieces wedge together and interlock, giving the rock mass an apparent cohesion which increases the deformational stiffness and rock mass strength.

Reinforcement of a rock fill embankment strengthens the downstream slope by reducing the deformational response in the horizontal direction. The steel reinforcing bars used are generally about 3.0 to 4.0 cm diameter and 6 to 13 m long. With rock fill the bars are generally bent, Figure 10, enclosing a volume of rock and imposing an apparent cohesion. If these factors are quantified and used in a mechanistic finite element programme which models the anisotropies, the resultant tensile forces in the reinforcement are about 45% those that are determined from slip circle analyses. If the anisotropy is neglected, the generated forces in the reinforcement evaluated from an isotropic elastic analysis are approximately 70% those determined from slip circle analyses, Brown and Pells (1983).

EXAMPLES OF TAILINGS DAMS IN AUSTRALIA

Introduction

Climate and topography vary considerably throughout Australia and this, combined with the ever increasing environmental issues, results in a wide range of methods used for handling tailings. Many mines separate the size of the tailings and use the coarse fraction for ground support while the fine fraction is disposed of in tailings dams. The two most important general issues effecting tailings over the last 25 years have been the environment and rehabilitation after the completion of mining.

The predominant types of tailings have come from the Alumina, Coal, Uranium, Gold, Heavy Sands, and Ferrous mineral industries. Other tailings are derived from quarrying, tunnelling, and smaller projects such as ceramics, control of floods, and erosion. Each of the aforementioned activities produce tailings with different characteristics which are important to recognise and design for accordingly.

The main methods of tailings disposal used in Australia are:

1. discharge to waste;
2. discharge to mining or quarried pits;
3. valley (fill) containment using borrow or tailings material;
4. ring (reservoir) embankments using borrow or tailings material;
5. thickeners

Associated with these disposal methods the techniques of upstream, centreline, and downstream construction are used. In this paper, areas and mines where these methods and techniques are used are noted with a few controlling criteria.

In Australia, the cost of tailings disposal generally varies from A$ 10×10^3 to 10×10^6. In 1985 the approximate cost per tonne (dry) for tailings placed in dams ranged from 40c to $2.0 per tonne with a model of 50c per tonne, Kurzeme (1986). As a percentage of the total operating cost, this ranges from less than 1% to about 6% with a modal of approximately 3%.

It is becoming evident in Australia, that for the purposes of control and rehabilitation, it is important to separate the water out of the tailings as quickly as possible. In many situations, this is difficult to do, even in the long term with materials such as the red muds (Alumina) and coal scrubbings. Though much has been done by underdrainage and defining functions of the waste disposal system, there are still many problems requiring solutions.

METHODS OF DISPOSAL

Discharge to waste

At Panguna, Bouganville, Papua New Guinea, the tailings and overburden are dumped into the Kawerang River which then travels 34 km to the coast. Aggradation has endangered the mine water supply and could potentially cause off lease flooding. Plans are now in hand to transport the tailings to the coast and dump them into Empress Augusta Bay.

Mount Lyell from its inception early this century discharged its tailings into the King River. This has resulted in a large delta and much pollution in McQuarrie Harbour. The tailing contains 30% sand size (75µm. Other schemes at Mount Lyell were considered a few years ago, but because of the problem and cost of diverting storm water runoff and with its now limited life of the mine, river dumping was allowed to continue.

In pit disposal

Groote Eylandt, which mines manganese, through cycloning separates the sizes into sand between 100 µm and 1000 µm. This material is used to build bunds (embankments) within the pit which retain the fines. There is no underdrainage or subsurface water control.

Ardlethan is a tin mine which has been operating for 20 years, and over this period has adopted a number of methods for handling the tailings. The general gradation of the tailings has about 49% passing the silt size of approximately 75μm.

Initially an upstream spigotted beaching sedimentation embankment was used, Photo 1, then five years later a mine waste rock embankment with an associated downstream fine sediment enclosure was used, Photo 2. Yet give years later, a cycloned downstream dam was constructed which reached the height of 30 m, Photo 3. Recently, the tailings have been disposed of in an abandoned open mining pit.

Valley containment

No. 8 tailings at Mt. Isa mines is a zoned earth rockfill embankment 31 m high. The area enclosed is 4 km by 5 km which will ultimately store 100×10^6 m^3 of tailings (dry), presently 40×10^6 m^3 are stored. Copper, lead, and zinc tailings are cycloned from which the sand sizes are used for underground fill support. Fine tailings are thickened to a 50% to 60% solid concentration primarily to recover water. The tailings are discharged at a point some distance from the embankment and controlled by groyne embankments to equitably distribute the tailings. Slopes of the tailings vary from 1:75 to 1:850. There is no provision for stormwater diversion as evaporation controls the water capacity. A catch bund downstream of the main embankment and bore holes are used to intercept seeping water and return it to the main storage.

Luina No. 2 tailings dam is an upstream embankment constructed from coarse tailings sorted by cycloning on the crest. Tailings water is removed through a decant tower. Because the original starter dam was an impermeable structure, this resulted in an elevated phreatic surface causing seepage on the down stream face. To obviate this a stabilising rock berm was installed.

Ring embankment containment

This is the predominant disposal method in Australia because of the generally dry climate and flat topography.

Rosebury, which is a base metal mine in Tasmania, is located on a plateau above the gorge of the Pieman River. Compacted glacial till is used for the embankment using the downstream construction method. A toe rock drain is used to control the phreatic surface. The fine tailings, 95% $<$ 75 μm are delivered to the one side of the reservoir displacing the water to the other side where the spillway is located.

Woodlawn is a copper, lead, zinc mine which disposes its tailings using a partial valley and ring embankment. Two dams namely the north and south dams control all the water on the lease. The north dam which is the slightly higher one (2 m) collects water from the open pit and waste rock dump. The fine tailings, which is $<$ 100 μm with 25% $<$ 10 μm. No water discharge is allowed off the lease because of the high acidity and heavy metals content. Banks of sprays are used to increase evaporation and control the water balance. Catch drains collect the subsurface seepage which is pumped back to the lower dam.

The embankment is an earth rockfill water retaining structure which is stage constructed. To obviate the excessive use of rock fill, the second stage incorporates a geotextile, Figure 1, as the filtration unit. This saves a considerable quantity of compacted rock fill.

Norseman, a gold mine, is one of the few hand controlled embankments just recently completed in Australia. A lip 100 mm in height is spadded around the periphery of the ring embankment. The slurry is then delivered around the periphery in sections of spiggoted piping. Eighty percent of the tailings is less than 75 μm. The embankment slope is 70 degrees and at a height of 40 m. Water over the years has been decanted, pumped, and syphoned off.

Thickened discharge

Elura, a base metal mine, has tailings of which 79% $<$ 45 μm and 23% $<$ 8 μm and thickened to a solids concentration. The shape of the dam is circular where a third segment is used at a time. This segmentation is defined by radial training embankments with a perimeter embankment retaining water. Tailings are deposited from a fixed central discharge point. The resultant sloping surface has an average gradient of approximately 1 in 60 with the gradient near the discharge point of 1 in 47 which reduces to 1 in 150 near the toe. At the toe of the mound the water is pumped back into the plant circuit.

CONCLUSIONS

In disposing of mine waste, it is important to know what are the characteristics of the waste and what are the associated functions of the retaining structure. For example, is the dam to retain both tailings and water or just tailings? What is the hydrology of surface and subsurface flow and how are flood conditions to be handled? Once the functions are defined, they are readily designed for using cost effective staged construction. An attitude exists in many parts of the mining industry in Australia, that if a water retaining structure is built, there are no problems in the disposal of mine waste. However, when rehabilitation of the mine site is considered, the water retaining area is no asset.

With the supposition that the slurry is to be retained and the water drained, a rockfill retaining structure with through and over flow attributes is considered. The safety valve of a dam is generally its spillway, whereas for a rockfill through and over flow embankment, it is the embankment itself. Generally, the cost of providing freeboard and a separate spillway structure varies between 25% to 65% of the project costs.

Rockfill embankments without an impervious zone have for some time been built as coffer dams. Though failures have occurred, these have been, in Australia, few in number and when they have occurred, the reasons and possible remedies have been recorded. These experiences coupled with research and experimentation, have led to, at least, an understanding of some of the many mechanisms involved in the through and over flow drainage embankment. There are still, however, many aspects of the behaviour requiring clarification and definition. For example, limit analysis is still used in performing the design for the reinforcement. This brings in the requirement of recognising that the compacted rockfill is segregated and anisotropic both in deformation and permeability. With the inclusion of reinforcement, the anisotropy is accentuated with the interlocking of the compacted rockfill and concomitant increase of strength. Experience has shown that if the intactness of the downstream slope is assured, then the rockfill embankment as a whole remains stable. That is deep seated instability does not occur.

It is important to install the mesh within the horizontal and vertical embedded reinforcing before overtopping occurs. That is the layered embankment must be complete and ready to receive the overtopping water, otherwise the loose rock slams and breaks the downslope mesh. The toe areas, especially around the abutments, are prone to scouring and underscouring, hence there is an important need to protect these areas. If criteria such as those mentioned above, are adhered to, then it is possible to ensure a safe and sound structure.

Drainage and filtration of the slurry by the embankment are as important to consider as is stability. Here again, there are many problems such as the drainage of red mud (alumina tailings) and coal scrubbings. Geofabrics assist greatly in insuring the attainment and flexibility required in the design of filters. Filter design also needs to recognise the anisotropic nature of the tailings and rockfill. Consequently, using a composite materials approach, the rockfill embankment offers great versatility in the handling of tailings.

River diversion of high water flows, if required, is an expensive part of any waste disposal system. However, using a through and over flow embankment, the cost can be dramatically reduced. There are other techniques which also help in the reduction of costs such as rolled low cement content rockfill embankments and overlay slab spillways.

REFERENCES

Blight, G.E. (1969) "Shear stability of dumps and dams of gold mining waste". The Civil Engineer in South Africa, Vol. 2, No. 3, SAIC, Johannesberg, 49-54.

Bowling, A.J. (1980) "Investigation into the deformability of rockfill". Third Aust.-New Zeal. Conf. on Geomech, Wellington, 1.1-1.12.

Brown, B.S. and Pells, P.J.N. (1983) "Analysing anchor loads in rockfill dams:. Water Power & Dam Construction, Aug., 50-53.

Cooke, B.J. (1982) "Progress in Rockfill Dams". 1982 Terzaghi Lecture, ASCE.

Fitzpatrick, M.D. (1977) "Reinforced rockfill in Hydro-Electric Commission Dams". ANCOLD Bulletin No. 49.

Giroud, J.P. (1981) "Filter criteria for geotextiles". Second International Conference on Geotextiles, Las Vegas, 103-08.

Gowan, M.J. (1980) "Slimes dams: Some aspects of their design and operation". The Institute of Quarrying, Durban.

Jennings, J.E. (1979) "The failure of a slimes dam Bafokeng, Mechanisms of failure and associated design considerations". The Civil Engineer South Africa. June, 135-45.

Leeder, M.R. (1982) "Sedimentology, Process and Product". George, Allan, and Unwin (publisher) Ltd.

Mackenzie, P.R. & McDonald, L.A. (1980) "Use of soft rock in Mangrove Creek Dam". 20th A.G.M. of ANCOLD (Aust. Nat. Comm. on Large Dams), 23-33.

Midgley, D.C. (1974) "The failure of a slimes dam at Bafokeg, Hydrological aspects and a barrier to further escape of slimes". The Civil Engineer of South Africa. June, 151-56.

Olivier, H. (1967) "Through and overflow rockfill dams-new design techniques". Proc. Inst. Civ. Engrs., Vol. 36, 433-471.

Parkin, A.K. (1963) "Rockfill dams with built in spillways". Wat. Res. Found. Aust., Bull. No. 6.

Pells, P.J.N. (1978) "Reinforced rockfill for construction flood control". ASCE Journal of Construction Division, CO1.

Sandle, R.B. (1961) "A laboratory investigation of self-spillway rockfill dams". M.Eng.Sc. thesis, University of Melbourne.

Schlosser, F. (1979) "History, Current and Future developments of Reinforced Earth". Symposium on Reinforced Earth, Sydney Inst. of Tech.

Shand, N. & Pells, P.J.N. (1979) "Experience in design and construction of reinforced rockfill dams". Transactions, 19 ICOLD Congress 036, R19, Montreal.

Sherard, J.L., Dunnigan, L.F. & Talbert, J.R. (1983)A "Filters for silts and clays". ASCE

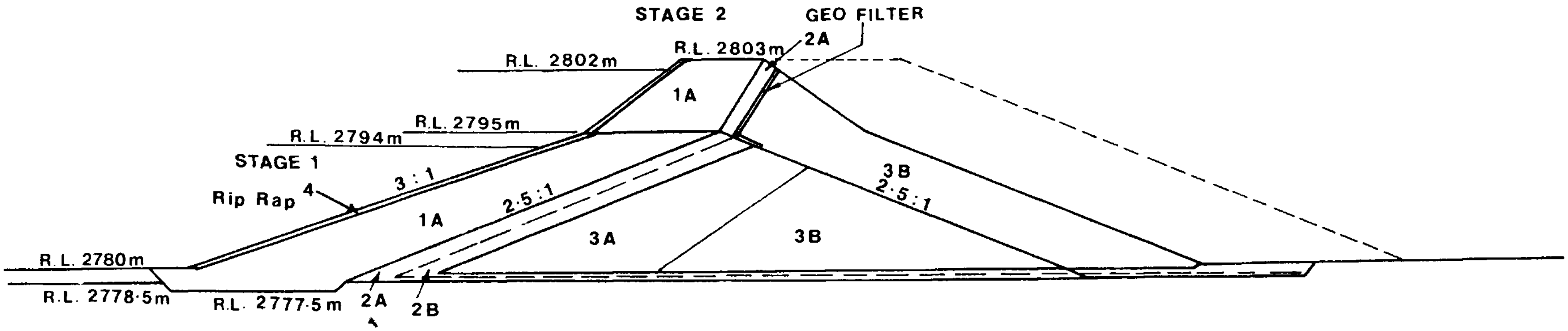
Sherard, J.L., Dunnnigan, L.F. & Talbert, J.R. (1983)B "Basic properties of sand and gravel filters". ASCE

Taylor, D.W. (1948) "Fundamentals of Soil Mechanics". Wiley, P 122.

Wilkens, J.K. (1956) "Flow of water through rockfill and its application to the design of dams". Proc. Aust-NZ Conf. on Soil Mech., 141-149.

Wilkens, J.K. (1963) "The stability of overtopped rockfill dams". Proc. Aust.-NZ Conf. on Soil Mech.

Weiss (1951) "Construction Technique of passing floods over earth dams". Proc. ASCE, Paper 2461.



EMBANKMENT NOTES

1A	Impervious Earthfill	3A	Earth Rockfill
1B	Semi impervious Eartfill	3B	Rockfill
2A	Earthfill Filter	4	Graded Rip Rap
2B	Earth Rockfill Filter		

FIG. 1

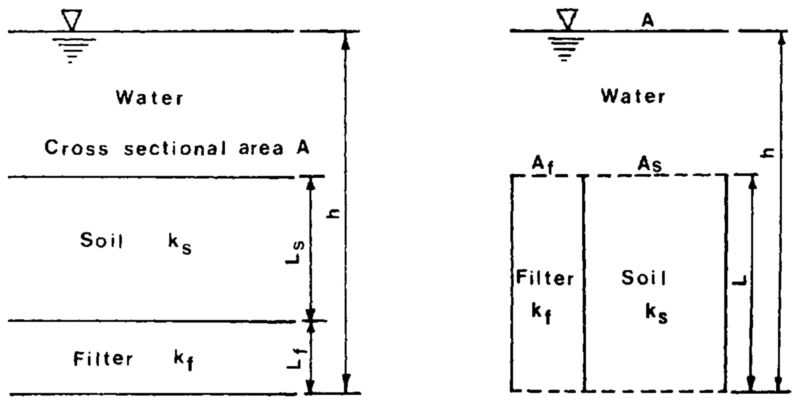

Water
Cross sectional area A
Soil k_s
Filter k_f
L_s
L_f
h
A
Water
A_f
A_s
h
Filter
k_f
Soil
k_s
L

FIG. 2

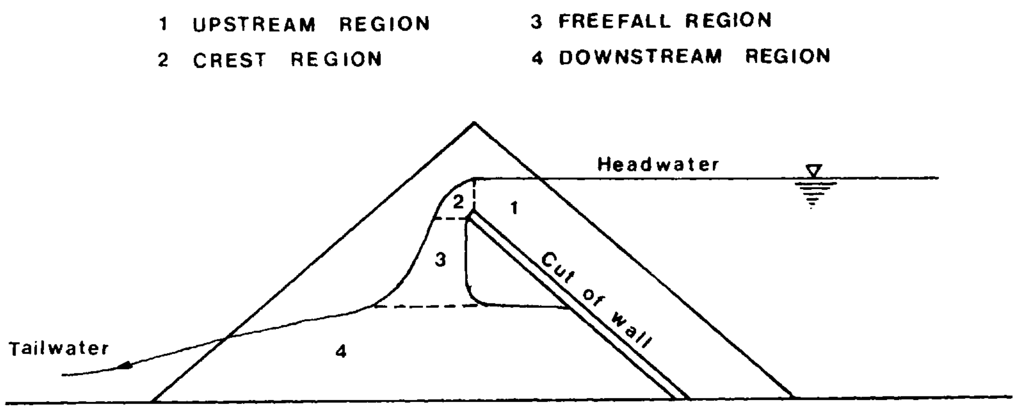

1 UPSTREAM REGION
2 CREST REGION
3 FREEFALL REGION
4 DOWNSTREAM REGION
Headwater
Tailwater
Cut off wall
1
2
3
4

FIG. 3

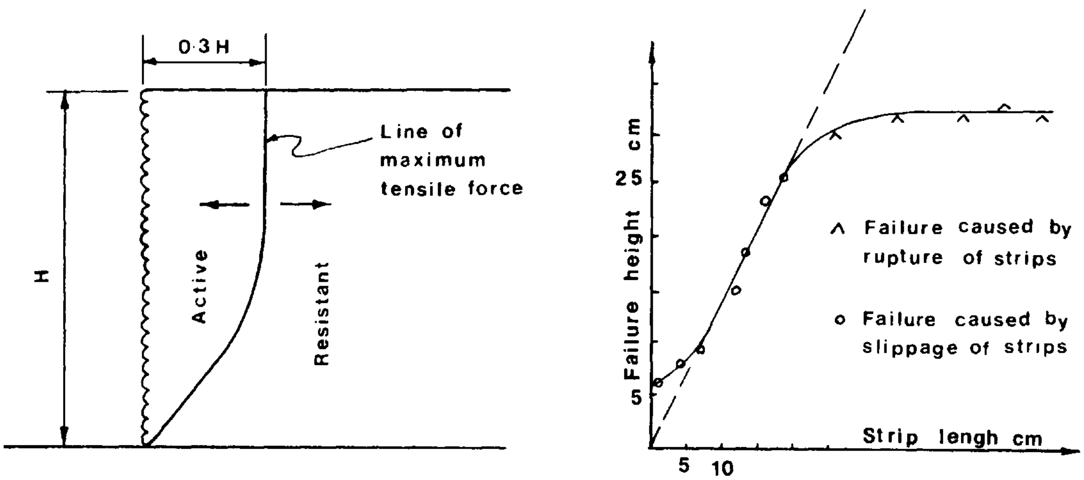

0·3 H
H
Line of
maximum
tensile force
Active
Resistant
Failure height cm
25
5
5
10
Strip lengh cm
Λ Failure caused by
rupture of strips
o Failure caused by
slippage of strips

FIG. 4

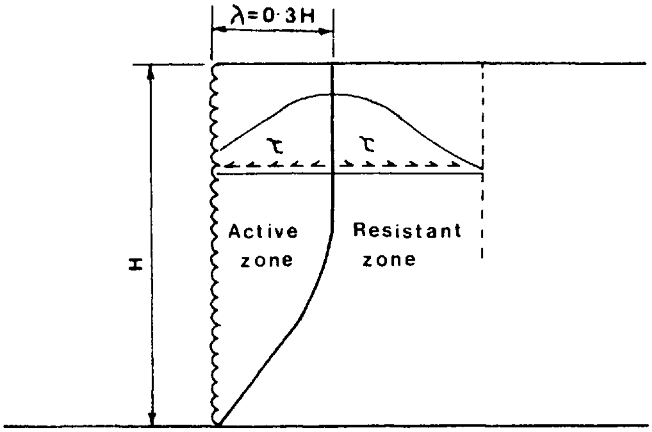
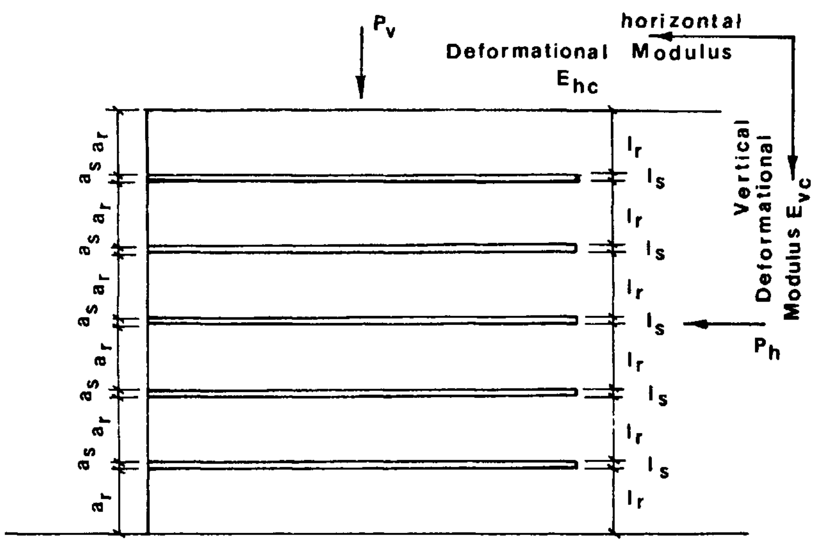


FIG. 5



$$\frac{1}{E_{vc}} = \frac{L_r}{E_r} + \frac{L_s}{E_s} \qquad L_r = \frac{\sum l_r}{\sum l_r + \sum l_s} \qquad L_s = \frac{\sum l_s}{\sum l_r + \sum l_s}$$

$$E_{hc} = A_r E_r + A_s E_s \qquad A_r = \frac{\sum a_r}{\sum a_r + \sum a_s} \qquad A_s = \frac{\sum a_s}{\sum a_r + \sum a_s}$$

FIG. 6

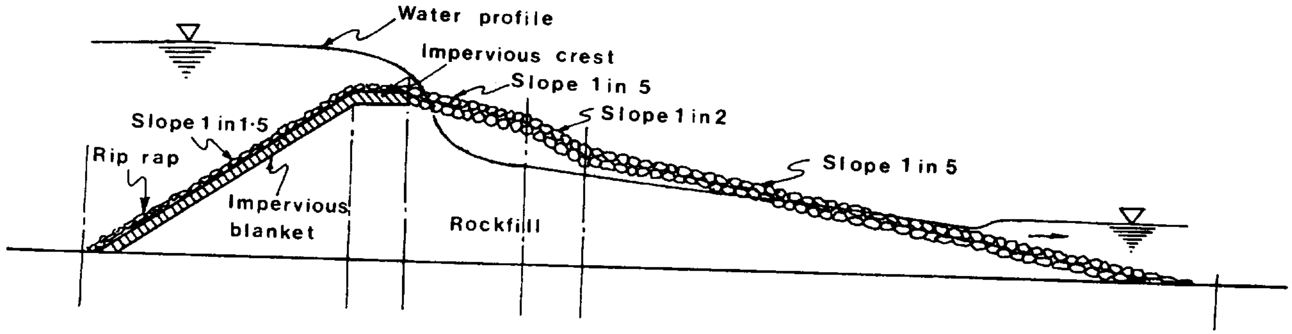
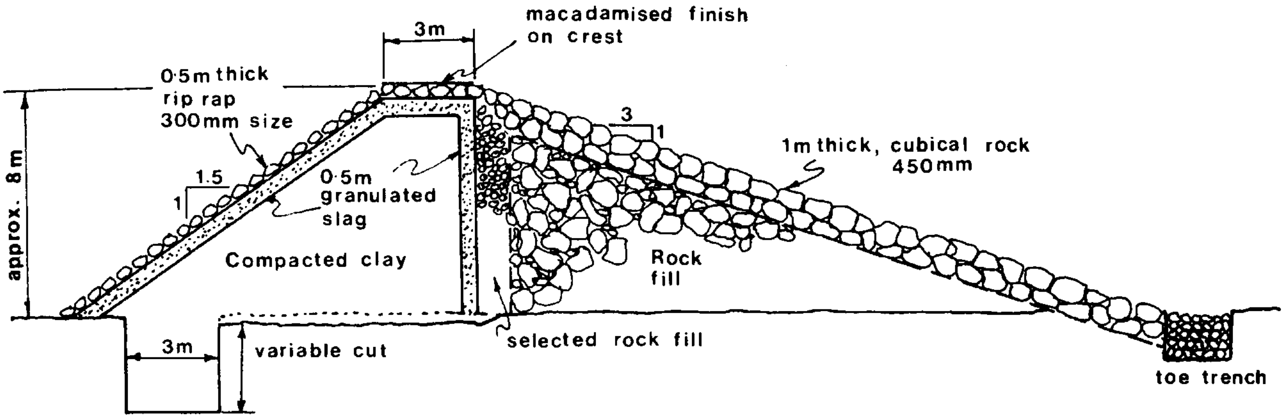
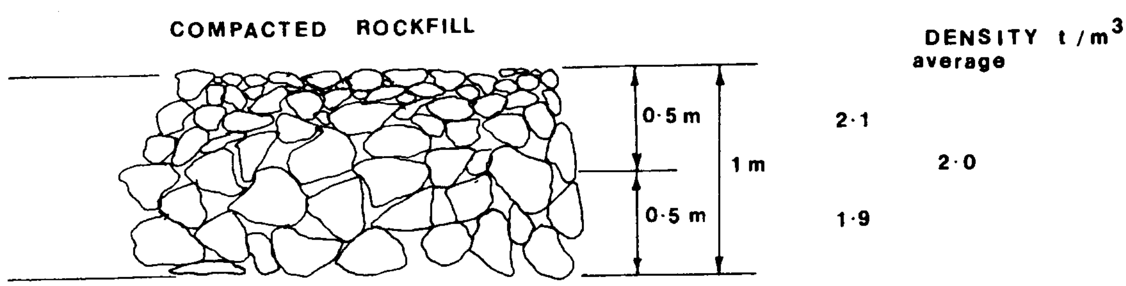

Water profile
Impervious crest
Slope 1 in 5
Slope 1 in 2
Slope 1 in 5
Slope 1 in 1·5
Rip rap
Impervious blanket
Rockfill

FIG. 7



macadamised finish on crest
3m
0·5m thick rip rap 300mm size
approx. 8m
1.5
1
0·5m granulated slag
Compacted clay
3
1
1m thick, cubical rock 450mm
Rock fill
3m
variable cut
selected rock fill
toe trench

FIG. 8



COMPACTED ROCKFILL
DENSITY t/m³ average
0·5 m
1 m
0·5 m
2·1
2·0
1·9

FIG. 9

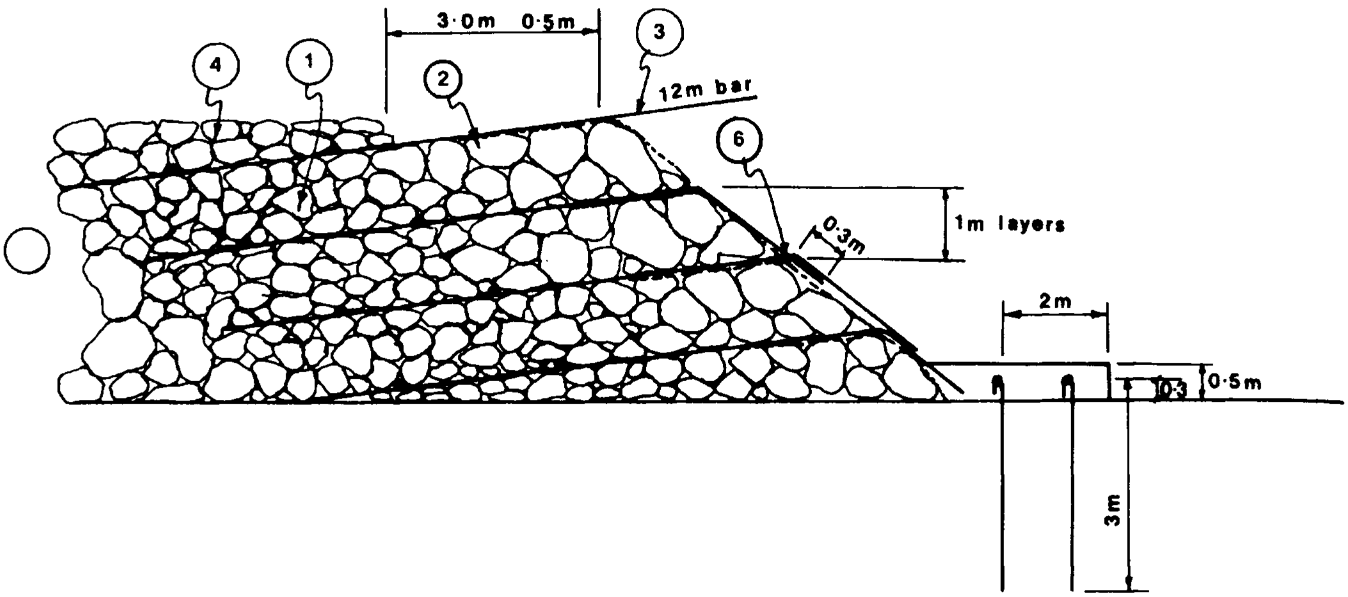


FIG. 10

(1) Loads are dumped at spacings that will give the approximate volume of the reinforcement region of 1m thick layer. The load should contain several pieces of rock about 0.5 to 0.9m size.

(2) Spread the dumps using a dozer to give the back slope. This spreading with the blade slightly raised pushes the larger boulders to the outer face of the slope.

(3) 12m bars are laid on the slope projecting at least 1.8m. Two transverse No 6 bars are welded to the sloping bars creating a good anchor. Vertical projecting bars are sometimes also welded.

(4) The sloping surface is covered with rockfill leaving a 3m gap to the face, FIG. 10. This rock is dumped near the face and dozed along and away from the face with the blade slightly raised controlling the rock size. That is larger boulders away from face.

(5) The mesh is placed and pulled taut. There are a number of techniques used for achieving this.

(6) The projecting bars are bent down (eg Hickey device used in bending reinforcement bars) to overlap the lower bars, FIG 10, and then welded.

(7) Good practice is to start each layer from the abutments towards the centre, and complete each layer before starting the next.

MODELLING OF FLOW THROUGH A MINE WASTE DUMP

by

W.A. McLaren, P.Eng
Western Canada Hydraulic Laboratories Ltd.

INTRODUCTION

AMAX of Canada Ltd. operated a molybdenum mine at an elevation of approximately 600 m at Kitsault on Alice Arm in northern British Columbia between 1981-1982. Operations have been shut down since 1982 for economic reasons. The mine property had been operated by previous owners between 1967 to 1972. Tailings at that time were discharged into Lime Creek at an elevation of approximately 500 m and were conveyed by natural stream flow down to Alice Arm. The majority of bed sediment in lower Lime Creek represented waste rock from the early operation.

Approximately 250,000,000 cubic metres of waste rock must be removed from Amax's open pit operation. Topographic and economic restraints indicated that portions of the Patsy Creek and Lime Creek Valleys should be used to store the waste rock. Piteau and Associates proposed that an initial test dump be constructed in Patsy Creek adjacent to the open pit operation, Figure 1, to determine what impact the later storage of larger volumes of waste rock would have on sediment production in the streams, and whether or not the proposed fills would be stable during design floods. The test dump would be used to assess the ability of a later, larger, fill to transmit Patsy Creek discharges and to assess sediment production downstream of the dumps in both Patsy and Lime Creeks.

Prior to receiving environmental approvels for constructing the small test dump, studies were required by regulatory agencies to assess its likely sediment production. A two-phase physical and mathematical model study was undertaken to provide an estimate of likely sediment production and recommend possible techniques to reduce any unsetisfactory sediment levels.

PATSY CREEK HYDROLOGY

Precipitation records were available from several stations in the general vicinity of Alice Arm. Figure 2 shows the mean monthly precipitation received during the period August 1, 1976 to July 31, 1977 at elevation 25 ft. near Alice Arm. This period was representative of the long term distribution. Autumn rains in the area generally begin in September and rainfall is highest during October. Toward the middle or end of November the precipitation changes to snow, the greater portion of which accumulates during winter months. From November to March the total mean monthly precipitation, mostly in the form of snow, steadily declines. The small peaks in the Lime Creek hydrograph indicate that the temperature rises occasionally to well above freezing during this period. Throughout spring and summer the precipitation remains low. The mean annual precipitation in the area varies from 63 inches near sea level to 80 inches et elevation 300 m.

An examination of the Water Survey of Canada records for 1977 to 1980 indicated that average flows in Patsy Creek could be represented by seasonal time periods in which flows were relatively constant. These periods and the approximate corresponding Patsy Creek flows were calculated as follows:

December-April	.07 m^3/s
May-June	.75 m^3/s
July-mid-August	.35 m^3/s
Mid-August-September	.20 m^3/s
October-November	.35 m^3/s

Typical summer and fall storms were estimated by examining the WSC data and by reports from Stevenson International Groundwater Consultants. Typical summer and fall storms were found to produoe peak Patsy Creek discharges of 2 and 3 m^3/s respectively, with storm lengths of 13 hours. The peak two year and 100 year storm discharges were determined to be 26 m^3/s and 60 m^3/s respectively. Corresponding hydrograph shapes were determined from those established by SIG. Hydrographs for typical fall and two year storm events are given in Figure 3. Subsequent hydrological studies by Klohn Leonoff Consultants Ltd. found that hydrographs for the two-year and larger storm events were based on erroneous WSC data and should have been reduced by about 60%.

EXISTING SEDIMENT PRODUCTION IN PATSY CREEK

In order to appreciate the possible effect of the test dump on sediment concentrations in Patsy Creek, it was necessary to first determine the existing sediment load in the creek. Initial field survey observations indicated that normal sediment transport in Patsy Creek was limited by the amount of material available for transport and not by the capacity of the creek to transport sediment. Observations in the upper reaches of Patsy Creek above the proposed test dump site, indicated that the active bed material transport size fraction was in the 7.5 to 15 cm diameter range.

Downstream from the waste dump site near the junction of Patsy and Lime Creeks, Patsy Creek flows along the toe of an existing mine waste dump. This provided additional finer material for transport as bedload and suspended sediment. An estimated 90 percent of the material transported to the mouth of Lime Creek originated from the mill tailings and mine waste deposited as a result of the previous operations.

In rivers where sediment is supply limited, field measurements of the transport must be made to determine the long term transport rate. Two Manning suspended sediment samplers were installed on Lime Creek which were triggered to sample at regular intervals of high stream flows by an automatic water level recorder. The sampling program showed that existing suspended sediment concentrations in Lime Creek varied from 300 to 1600 mg/l during periods of heavy rainfall.

GRAIN SIZE ANALYSIS

Calculations of the sediment production from the test fill required information on the grain size distribution of the material, particularly the amount of material below 2 mm available for transport out of the saturated zones of the fill. As operations had not yet commenced, direct prototype sampling was not possible. An estimated material size range was estimated for the overall dump from analysis of samples obtained by AMAX from the mine site and from other works, Figure 4.

TEST DUMP SEDIMENT PRODUCTION

In order to assess the effect of the proposed test dump on sediment concentrations in Patsy Creek, a combined physical and mathematical model study program was undertaken.

Physical model tests were undertaken to determine water levels in the test dump as a function of discharge, test dump configuration and material characteristics. The data was then used as input to a numerical model to predict sediment production from the test dump for several possible discharge sequences including a summer storm, an autumn storm and a once per 2 year storm event. The physical model was also used along with theoretical calculations to assess the stability of the downstream face of the test dump and to develop a bypass channel to handle flows in excess of the potential once per 100 year flood.

PHYSICAL MODEL DESCRIPTION

A model of the Patsy Creek valley stream was constructed at an undistorted horizontal and vertical scale of 1:125 in a flume at WCHL. Existing valley topography was constructed of sand-cement.

Previous studies by others had indicated that reliable physical model results of flow conditions inside coarse porous media could be obtained by adjusting the gravel size scale to maintain the correct relationship between the seepage and gravitational forces, rather than the gravitational and inertial forces as is normally used in a free surface Froude number scaled model. The porosity of the fill in the model was therefore increased to above prototype values to maintain the correct permeability under the Froude scale head and discharge conditions.

The test dump material gradation was scaled at approximately 1:85 to reproduce internal flow conditions. Riprap and other material on the downstream face was scaled at approximately 1:125 on the basis of Shields criteria for incipient motion as its stability was of primary concern. Grain sizes and porosity were determined for a uniform test dump material, for a test dump constructed with uniform material underlain by a coarse basal layer, and for a test dump constructed in three distinct layers. The three layer model anticipated that some segregation of rock sizes was likely to occur during construction through the dumping operations. Different material gradations were used in each layer.

A verification procedure was carried out by comparing results from measured flows through a simple physical rectangular block of model gravel with the results of analytic computations based on prototype material. Use of a rectangular block in the verification test eliminated the effect of irregular variation in flooded test dump area with rising elevation in the valley affecting model calibration results. Three verification test runs were made using each of three different grain sizes which would be used in the Patsy Creek test dump physical model. The verification results were compared against conditions calculated for the rectangular block using turbulent flow relationships for porous media. Comparison of the results showed that for a total of nine test conditions the model gave internal flow velocities which were on average 6% ebove those determined analytically. It was concluded that the scaling criterion reasonably represented conditions in the prototype and that the corresponding scaling relationships would give good test results for use in the main mathematical model of the test dump.

The three these dump material configurations were studied in the physical model by measuring the discharge introduced to the upstream end of the flume and piezometrically measuring the resulting water levels inside the model dump. Each test dump configuration was examined at several discharges. For each run the discharge and water levels at six points in the model test dump were measured, Figure 5.

For all discharges the test dump with basal drain showed significantly lower water levels within the dump than those for the dump without basal drain. This was significant as it exposed less potential sediment to flow through the test dump at a given flow than had the material been uniformly distributed. The results from the three layer test dump showed a further lowering of the phreatic surfaces in the model.

NUMERICAL SEDIMENT TRANSPORT MODEL

A mathematical model was developed to predict sediment transport rates through the Patsy Creek test fill based on results of previously published theoretical and experimental investigations on fine sediment transport through a coarse matrix. A sediment transport function through a porous medium was devised in relation to a critical shear stress concept, the critical shear stress which caused initiation of fine sediment motion through the large pores being evaluated through the hydraulic slope.

The two basic equations of the sediment transport model were:

i) A relationship between a given sediment size and a corresponding hydraulic slope necessary for initiation of sediment motion.

$$\left(\frac{dH}{dX}\right)_{cr} = \frac{W_{s-W}}{W} \cdot D \cdot Y$$

for which:

$\frac{H}{X}$	=	water surface slope
cr	=	critical condition
Y	=	constant
W_s	=	unit weight of sediment
W	=	unit weight of water
D	=	representative grain size

ii) A relationship between sediment transport and hydraulic slope for a given grain size.

$$Qs = \frac{K\left(\frac{dH}{dX}\right)^2 \cdot \left(\left(\frac{dH}{dX}\right) - \left(\frac{dH}{dX}\right)_{cr}\right)^D}{\left(\frac{dH}{dX}\right)^2_{cr}}$$

The value of the parameter K varied with discharge through the dump.

$$K = \frac{V}{\left(\frac{dH}{dX}\right)^n}$$

for which:

V = groundwater flow velocity
K = coefficient
n = groundwater flow exponent

As flow through the Patsy Creek test dump would be either partially or fully turbulent for high stream flows, the value of K incorporated a flow exponent, n, based on the relationship between flow velocity and medium permeability. The exponent varied between 0.5 for a fully turbulent flow and 1.0 for a fully laminar flow. Values of K were investigated by experimental calibration. A third equation was also added: a sediment conservation law used to evaluate remaining amounts of fine sediment left in the fill after every time step the sediment equation was applied.

For a given saturated volume, corresponding to a specific flow event, the sediment conservation law was written as:

$$Q_s = \frac{d(VE)}{dT}$$

where:

Q_s = sediment transport rate

VE = eroded volume of fine material within the saturated zone

T = time

Under unsteady flow conditions, for which the saturated volume, and hence the volume of fines available for transport, changed for each time step, the following steps were considered:

i For the ascending limb of the hydrograph, the sediment conservation law was not modified as volume eroded at time step (i-1) came from a smaller saturated volume than the one for time step (i). Therefore, volumes of eroded sediment could be directly added to each other.

ii For the descending limb of the hydrograph, the situation was slightly more complex as volume eroded at time step (i-1) came from a larger saturated volume than the one for time step (i). Therefore, the expression for total volume eroded at step (i) had to be modified to take this factor into account, the new expression was written as:

$$V_E(i) = V_E\,(i-1) \times \frac{Vsat.(i)}{Vsat.(i-1)} + Q_s(i) \times T$$

for which:

Vsat.(i) = saturated volume at step (i)

Vsat.(i-1) = saturated volume for precedent volume

The model also considered that:

i Downward sediment transport through percolation from above would not occur due to natural sealing of the test dump surface under construction traffic.

ii Downward sediment movement through the fill caused by the recession of a phreatic line was ignored.

iii During construction high suspended sediment concentrations could be anticipated in Patsy Creek. Occurrences such as slumping of material into the creek from advancing slope faces, caused by excessive rain, could cause unpredictably high, short duration concentrations which the model would not predict.

iv The surface of the dump would be at elevation 650 m and the length of the dump would be 400 m. The downstream surface slope would be between 1:3 to 1:4. An increase in the elevation of the top surface would not have any effect on sediment transport rates as this added volume would be above the saturated zone.

v The hydraulic gradient in the upstream section acted as driving mechanism for sediment feed to the wedge-shaped downstream section while the hydraulic gradient of the downstream section controls sediment output from the test dump.

vi All sediment within the flooded area of the dump which was fine enough to be moved by local flow velocities in the dump was considered to be displaced.

vii No new sediment was introduced to the dump by upstream flow.

Two numerical models were developed to simulate sediment transport from the fill:

i a simple model only able to simulate a given flow event, base flow or storm;

ii an expanded model able to simulate a year of flow events.

The first medel was used for the simulation of individual summer, fall and once per two year storm events with 2 m^3/s, 3 m^3/s, 26 m^3/s creek discharges respectively, Figure 6.

The second model synthetically combined a sequence of flow events including seasonal base flows between an assumed storm sequence.

MODEL PREDICTIONS

Estimates of base flow sediment production from the uniformly distributed test dump were started assuming an average fall flow as it was anticipated that construction of fill was to be finished by the end of the summer or early fall. Starting the model during the period of summer base flow and summer storm would have produced a more gradual washing of sediment from the dump with consequent lower peak sediment production values. One of the study objectives was to satisfy environmental concerns and therefore reasonable "worst case" situations were investigated.

Fall base flow sediment production estimates from the numerical model showed that for the first month the sediment produced from the test dump would increase the average concentration at the Lime Creek bridge from 30 to 50 mg/l with a prediction uncertainty factor of 2 (i.e., peak results could be as lows as 25 or as high as 100 mg/l). The main reasons for the degree of uncertainty were:

i the flashiness of sediment response to the hydraulic variables;

ii The estimates of sediment volumes in the upstream section available for motion.

As sediment production for the initially proposed uniformly distributed test dump was unacceptably high, the test dump model was redesigned to incorporate a 3 m thick basal layer of coarse rock. This layer was to act as a drain to conduct all normal flows under rather than through the dump. Further runs were also made to investigate the effect of vertical rock segregation during construction. The dump above the basal layer was considered as 3 distinct layers having different median rock sizes. It was concluded that modelling the test dump as uniformly distributed material would considerably over-estimate the level of the water in the test dump and consequent sediment production for a given discharge.

Calculated sediment production in mg/l from the test dump for the three individual storm and three dump conditions were:

Flow Event	Uniform test dump without basal layer	Uniform test dump with basal layer	Segregated test dump with basal layer
Summer storm $2\ m^3/s$	2000	600	20
Fall storm $3\ m^3/s$	3000	800	20
Two year storm $26\ m^3/s$	6000	2000	25

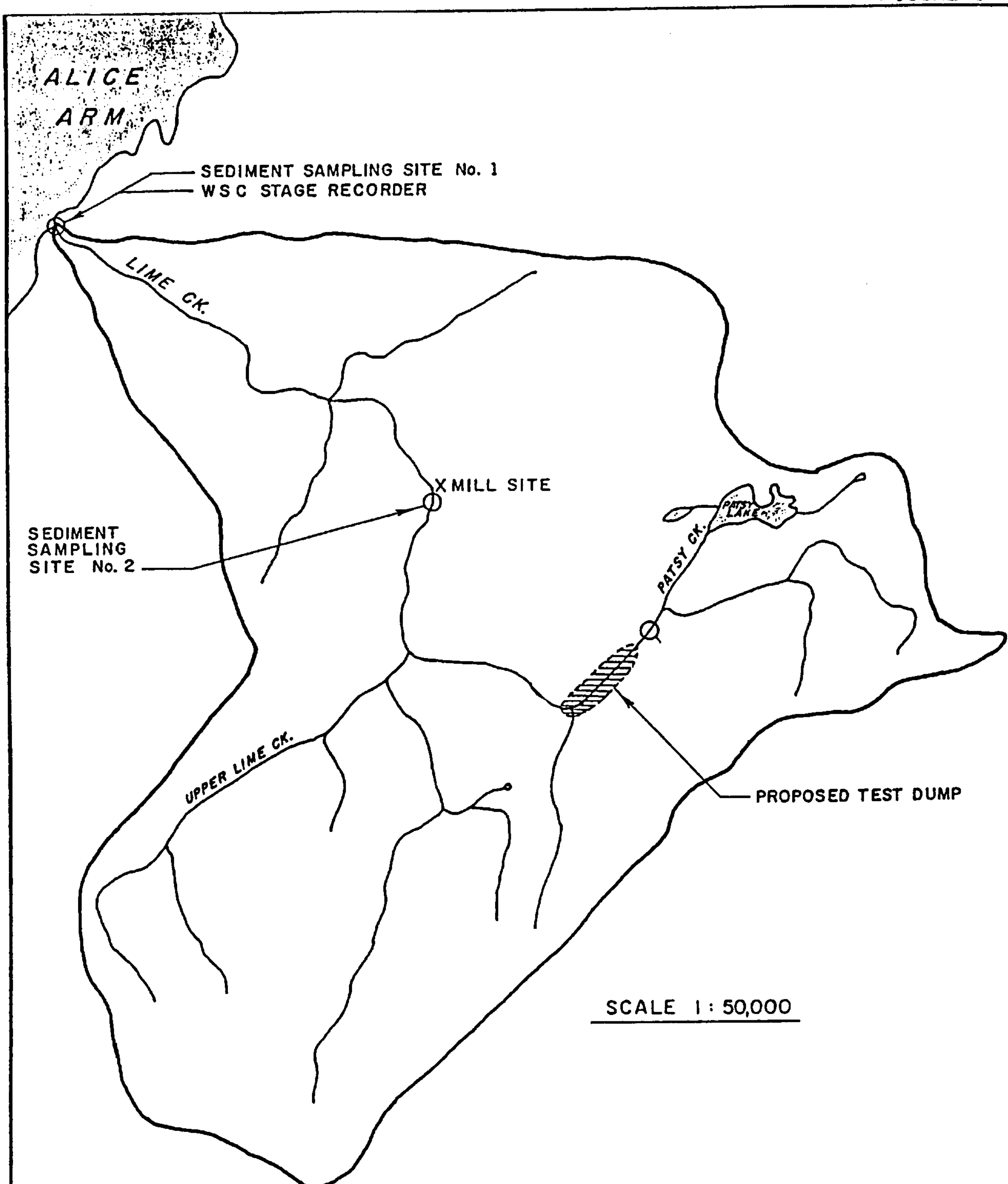
These results gave the most conservative, i.e., highest sediment estimates as they assumed that no sediment had been removed from the dump by low stream base flows prior to the storm onset. The sediment concentrations gave increases in the Patsy Creek stream values as flow passed through the dump rather than total sediment in the stream at the toe of the dump.

The results showed that the basal drain, plus natural segregation of the test dump rock during construction would result in generally acceptable sediment production levels for normal, or typical rainfall sequences. It would be possible, of course, that a sequence of exceptional storm conditions could occur to produce higher sediment levels. The levels of sediment increase in the stream, however, would still be low with respect to the measured existing site values.

DOWNSTREAM FACE STABILITY

The stability of the downstream face of the test dump was assessed in the physical model for a discharge of 60 m^3/s, representing the once per 100 year flow. Seepage forces at the toe of the dump were found to be equal to or less than those calculated during the initial geotechnical stability assessment.

The 1:125 scale physical model was also used to assess stability of the downstream face of the dump by potential overtopping during storms of greater than 1 in 100 year intensity and to design a rock lined bypass channel. A downstream slope of 1 vertical to 4 horizontal was found to be stable against seepage and overflow forces during the once per 100 year flood if armoured with a 3 m thick layer of 1 m diameter waste rock.



LIME CREEK DRAINAGE BASIN

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6639A-WCH am 81

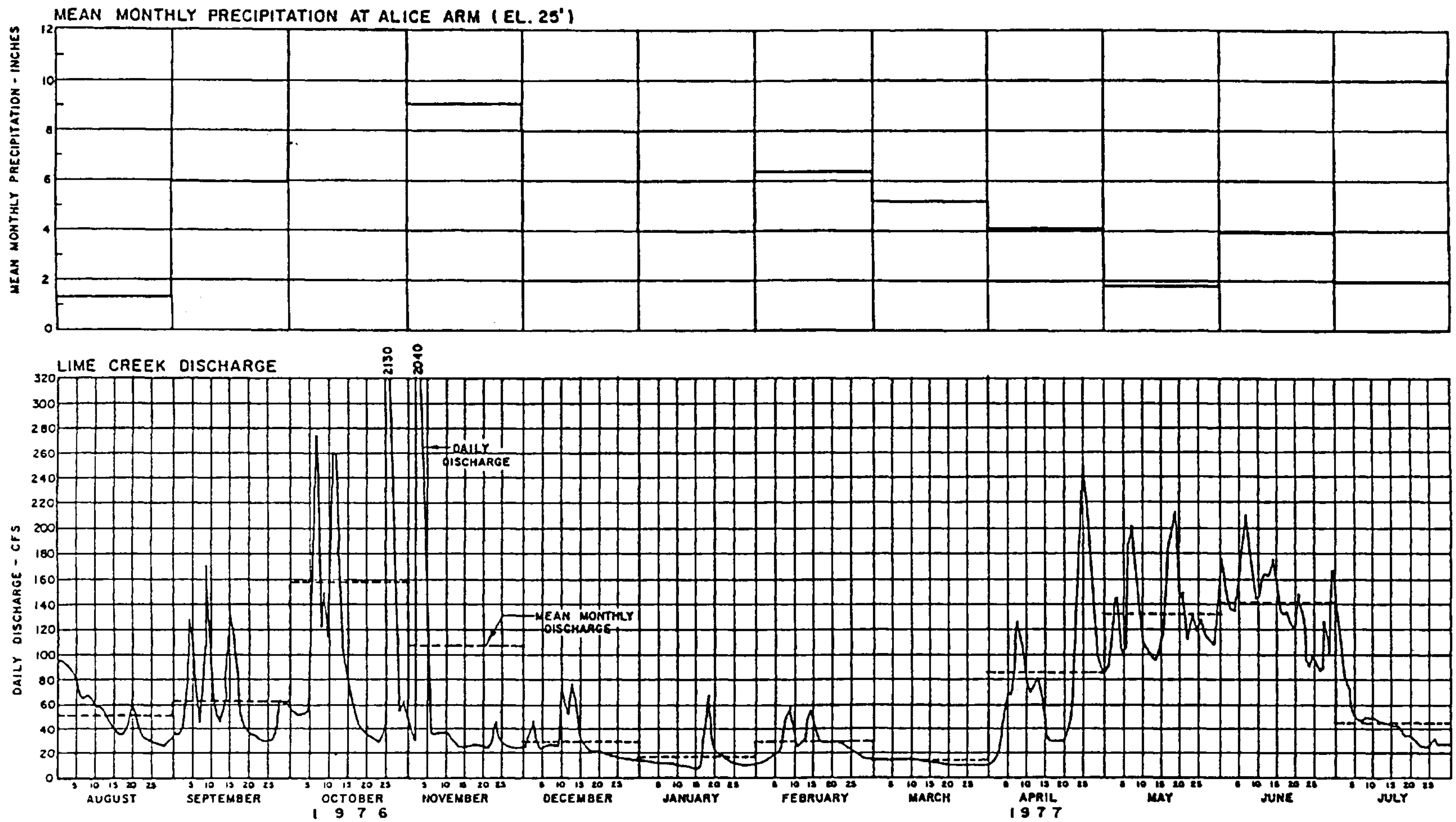
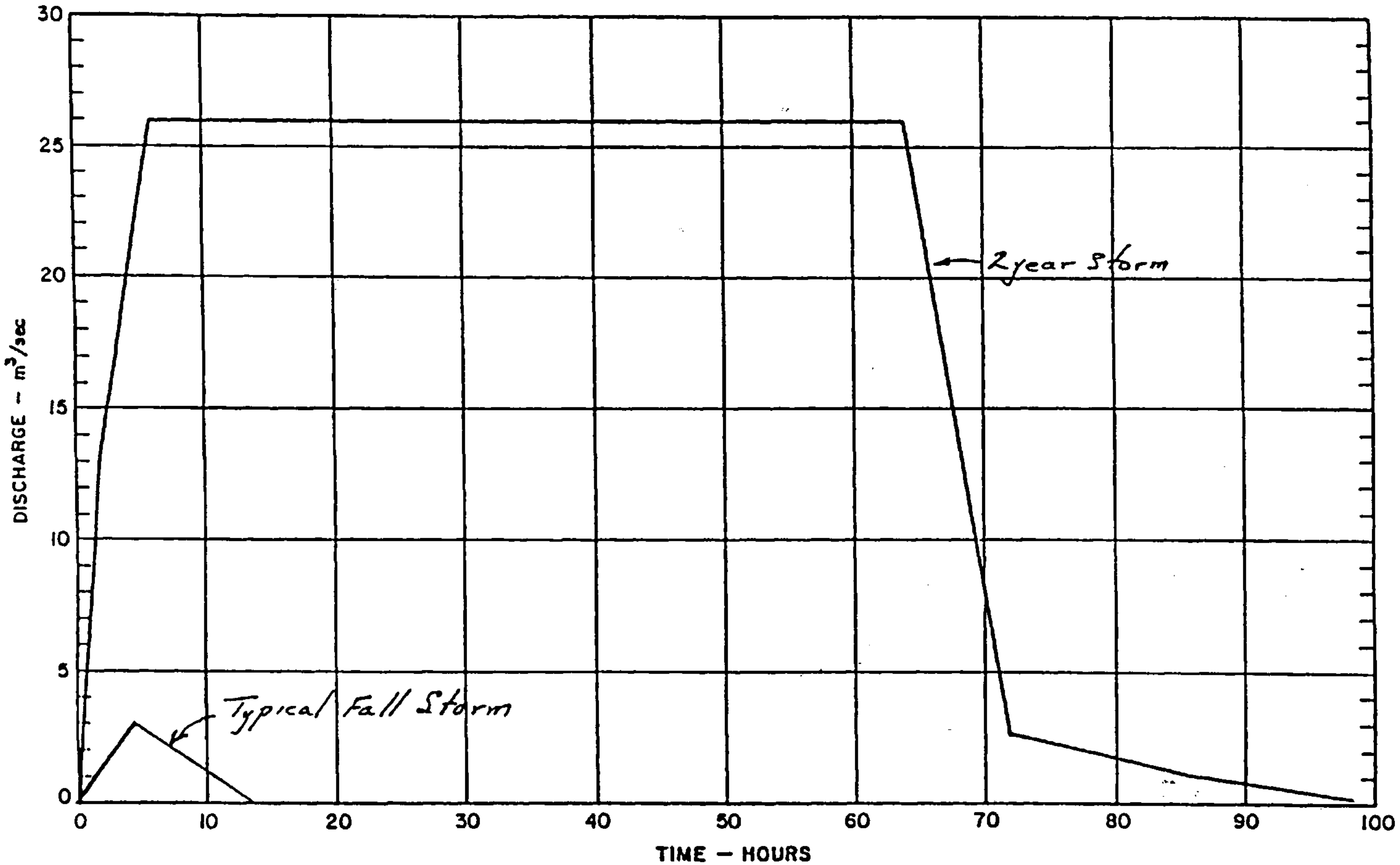


FIGURE 2

LIME CREEK HYDROLOGY
AUGUST 1976 - JULY 1977

WESTERN CANADA HYDRAULIC LABORATORIES LTD.



NOTE: Hydrograph shape based on that designed by S.I.G. (Nov. 1980)

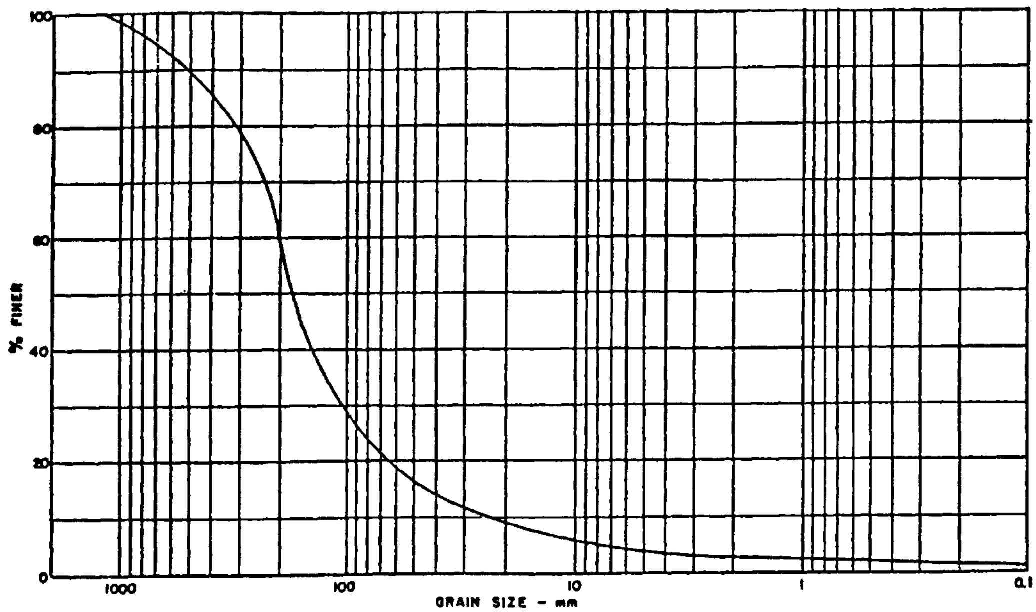
DISCHARGE AT TEST FILL SITE
2 YEAR STORM

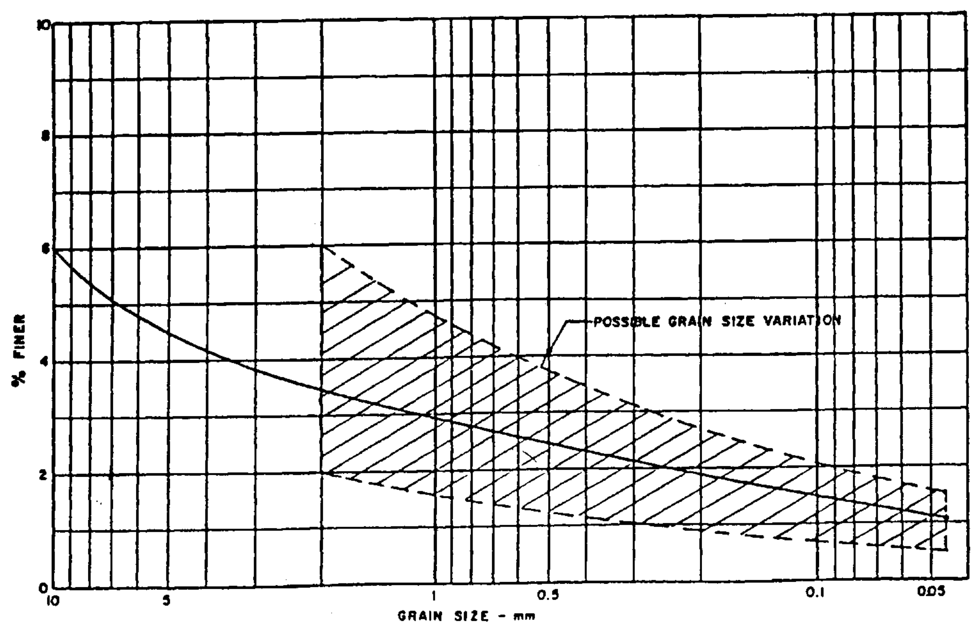
FIGURE 3

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcll 6638 A-WCH

FIGURE 4





GRAIN SIZE DISTRIBUTION

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

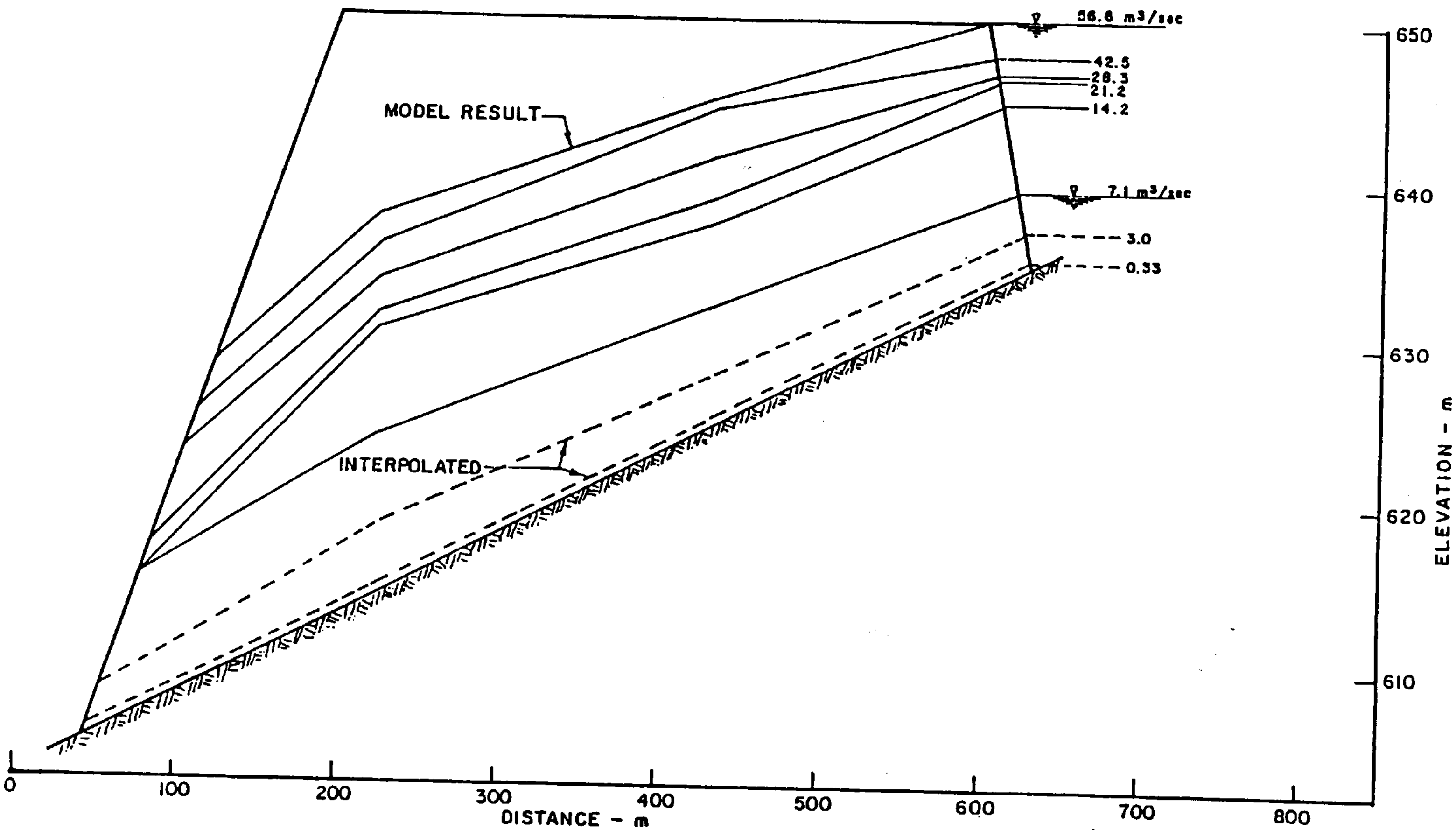
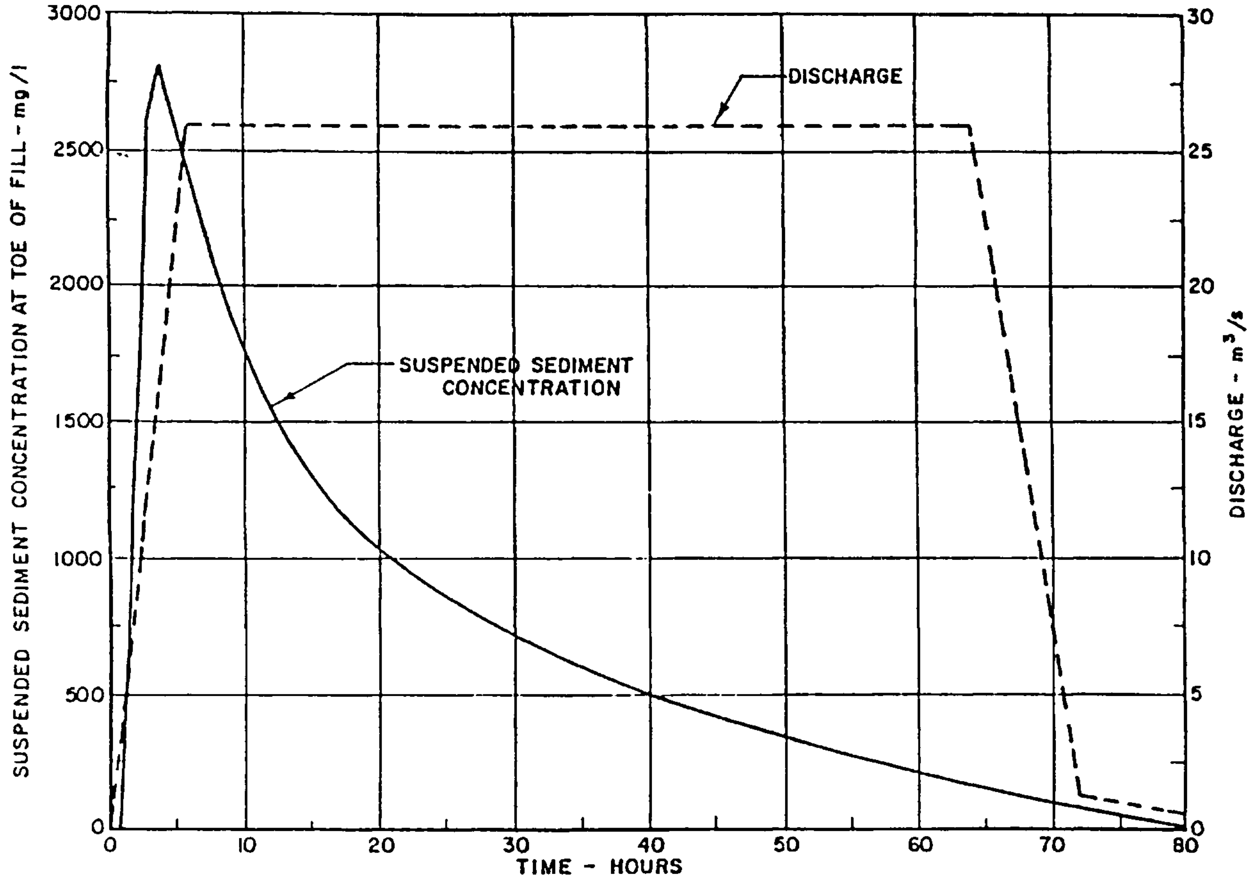


FIGURE 5

PHREATIC SURFACES IN TEST FILL

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcll 6638 A-WCH sm81



SEDIMENT PRODUCTION - TWO YEAR STORM

WESTERN CANADA HYDRAULIC LABORATORIES LTD.

bcil 6638 A-WCH

CONTRIBUTION FOR MINE WASTE STABILITY

by
Jacques Gourdou
Tecminemet Ingenierie
Trappes France

OVERVIEW

This presentation refers only to waste disposal in the thalweg of a flowing stream because in a flat site, waste disposal does not take place because of occurence of springs.

Disposal of mining waste in valleys is determined by the nature of the rejected products and by the morphology of the valley itself.

In many cases the occurence of a river or rivulet in the bottom line requires the construction of a drain. Correct working of the drain is mandatory for waste disposal system stability.

The drains generally work if correctly made but the risk of drain destruction is noticeable and the drain has to be maintained whatever the conditions of the disposal operation should be. Nevertheless some assays are necessary in order to verify if the drain is jeopardized by water accumulation inside the waste material.

In a wide range of climates vegetation regeneration on the slope is not a major problem if the slope itself is correct and additional material of appropriate nature is convenient.

PRINCIPLES

A first principle in the disposal process would be to divert surface waters coming from the upper basin via a peripherical collector built in the slopes of the valley out of the disposal area. Outflow of this collector is located downstream of the disposal site.

When dumping coarse mining waste from the height of a slope without taking any special care granulometrical segregation (size classification) is as efficient as the height of the slope and as the cubicity of the material.

Problems arise from the fact that it is not always possible to obtain these ideal conditions. Special conditions in which one could be far from the ideal case are numerous.

The present list is not exhaustive it is only our personal experience.

- products are not of cubic shape;
- bottom line is made up of fine and plastic material
- quick weathering of the rocks
- alteration in the disposal of two types of products
- slope angle more than 25°

The ideal case is shown in figure 1. Surface of repose is a gentle slope and rock is sound (no mould or loam). The height of dumping is sufficient for a good segregation. As an order of magnitude our estimate is 15 metres.

Under these conditions coarse material end boulders accumulate at the foot of the slope, fine material remains near the dumping point and permeability is continuously increasing from top to bottom. Rainfall or other water on the surface of the dump dissipates at the foot of the slope.

If a decision is made to create a dam with spoiled material crossing a valley, (side to side) as shown in figure 2 the surface of the slope angle does not provide a sufficient height of slope in the edges of the cross section to meet the preceding principle. According to a given level of permeability the limit profile is shown as line S of the figure 2.

If the objective is to create a disposal system extending from this dam across the valley, it is suggested the progression should be axially downward and not upward and a longitudinal profile as shown in figure 3 will result.

A progression upward would have insufficient height and accumulated spoiled material would not have appropriate sizing. Consequently the resulting permeability of the waste disposal system would probably be insufficient along the bottom line.

Such are the ideal conditions of coarse products disposal. Three principles are involved:

slope angle of original ground is less than the slope angle of coarse spoiled material

- surface of original ground should be cleaned of vegetation
- sufficient dumping height be maintained all along the longitudinal bottom line

Spoiled material not of cubic shape

This refers to dump material similar to slate or schist as a constituent of the walls of sedimentary deposits (coal). This material although coarse can show low permeability by shape when mixed with fines or when dumped from a significant height. Furthermore the handling of this product gives, for the same reason, a lot of fine material.

In this condition one has to sort as far as possible, materials into classes. The first type is the coarse product which is only to be dumped from a small height to avoid additional breaking and provide permeability. Second type is the fine product disposed in placed where permeability is not required.

Ground surface composed of fine or plastic materials

Two reasons are given for the ground surface to be composed of fine or plastic material. The most common one is that this is due to vegetative covering mixed with weathered rocks. The second one originates from the nature of the ground (i.e. natural limonitic soil in New Caledonia or mine tailings anywhere in the world).

1. Loam and weathering

   The whole mass of the waste may slip on the seam of plastic soil which acts as a lubricant between the disposed material and competent underlying rock. No remedy is known for this phenomenon. The solution is to get rid of the loam before the beginning of the operation (mine waste from Nickel mines in New Caledonia).

2. Old tailing

   Figure 4 explains the interesting case of Huaron mines. First a valley relatively flat has been filled with tailings. A smell creek flows along the upper level of this tailings disposal. The fine product content of these old tailings make them impervious. Mining wastes are dumped laterally, progressing from one side of the valley toward the other.

   Two phenomena occur simultaneously: coarse product is quickly embedded in the surface of the fine old tailings and the permeability at the contact of this surface is not secured. On the other hand the coarse tailing load leads to deep circular rupture. The uplifting of the fine products is shown on the figure 4.

   The miner faces the risk that the creek does not find its way among the upset fine products and impregnate the whole mass. This constitutes an unreliable dangerous and unstable dam.

   Risk involved in this disposal has been reduced by displacing the river bed on the opposite side of the valley and stopping the coarse disposal far enough back from the river.

Quick weathering of the rocks

The segregation phenomenon leads to a very high permeability which can decrease as a function of time for two reasons:

- Fine products abundant at the disposal site surface lead to a quasi clogging of water percolation through the coarse product. This happened in the case of a mine waste showing a granulometrical discontinuity, in other words a coarse product from the mine source mixed with a fine product from another.
- Quick weathering products act in the same way, for the intermediate size products weather quickly and the detritus goes down through the coarse product. To redress the loss of permeability longitudinal drains may be constructed to intercept percolating water.

Variability in the nature of material being disposed

When fine waste is produced by mining and is alternated with coarse products permeability is reduced. It is the same case as previously studied if no special measures are taken. The disposal system must be built in such a way that problems do not occur. First suggestion is to place the fine products where the height of the slope is minimal and where permeability is already low.

This works if the fine are not too abundant or if the disposal area is rather large.

We have no personal experience on the method which involves building a drain near the bottom line prior to disposal and covering this drain with intermediate sized products and with Bidim (*) and after, dumping products of any siza and condition. This system seems very economical provided the drain works perfectly in collecting both the water coming from upstream and the water coming from the disposal. This last collection is made through lateral branches.

DRAIN ALONG THE BOTTOM LINE OF THE DUMP

Necessity of this drain

If a valley is completely dammed by mine waste it is possible in many cases, to create a longitudinal drain going along the bottom line, even if the spoiled material is assumed to be permeable.

This drain can be made with concrete tubes or corrugated plate tubes or with other systems. Although the outflow of the stream should be low, it is not advisable for the internal diameter of the tube to be less than two meters. As far as possible this drain has to be in a straight line.

This drain should be equipped with a grizzly at the inlet end. This avoids the accumulation in the drain of broken trees or other pieces of vegetation.

The drain conducts the stream coming from the upstream part of the basin through the dump. In the same way, this drain is used for the waters filtering through the disposal to the upstream slope of the dam. The drain is of no use for waters filtering downward through the disposal.

Drain construction

The operation is a bit tricky and requires a touch of professionalism but the construction principle is quite simple:

- the drain must lay on sound rock;
- the drain must be laterally stable;
- open gates must be allowed to avoid unexpected external pressure.

(*) A kind of felt, unwoven material, from mineral origin.

It is self evident that the drain is always built before the stockpiling of waste materials. After erection of the drain itself a blanket of boulders covers and protects the drain from damage due to stockpiling (figure 5).

The general procedure is shown in figures 5 to 7. Preliminary work should be carefully controlled to avoid future problems.

In this operation it is assumed that the drain lies on sound rock with mechanical characteristics capable of supporting the future load of the whole disposal system.

If the hydraulic pressure increases inside the disposal system as a consequence of a low permeability on the downstream side it is mandatory to be able to release pressure on the drain structure. Therefore during construction, it is useful to locate in the walls of the drain, pressure take up and discharge gates.

LATERAL DRAINS

Surface water coming from the slopes of the valley along the final level of the disposal are generally gathered by the lateral collector which is dug out of in situ ground as described in figure 2. In the vent it is not possible to collect the water by this means (i.e. a subsurface spring) a lateral drain is installed.

This drain may also be constructed of larger diameter pipe protected by a pervious blanket of large boulders. A boulder drain can be constructed but the risk of clogging is posed.

SOILS RECLAMATION

Soils reclamation involves a control of the slope angle and knowledge of the nature of future erosion and weathering. It is well known that unplanned dumping of mining waste on a slope more than twenty meters in height, increases the difficulty of revegetation.

Soil reclamation includes two aspects:

- minimizing the slope angle;
- improving growth capability by adding surface soils to rocky dump material.

Reducing slope angle

We think that slope angle of two horizontal to one vertical is favourable to the growing of vegetation on the waste disposal. Water drains quickly but nevertheless it is possible to protect young plants with isolated boulders as shown in the figure 10. Such a method was used successfully in the south of New Caledonia (Penamax Company).

The general longitudinal section is then constructed as shown. Slopes from ten to twenty meters height are separated by berms. These berms avoid surface soils being carried away due to runoff from rainfall, and collect water to conduct it laterally to the side of the disposal.

Improving growth capability of spoil

Final slope faces may be covered with top soils which encourage revegetation. Revegetation can also be improved through a scattered pattern of anchoring boulders. These serve to protect young trees which may be planted downslope (figure 10).

TESTING AND EVALUATING

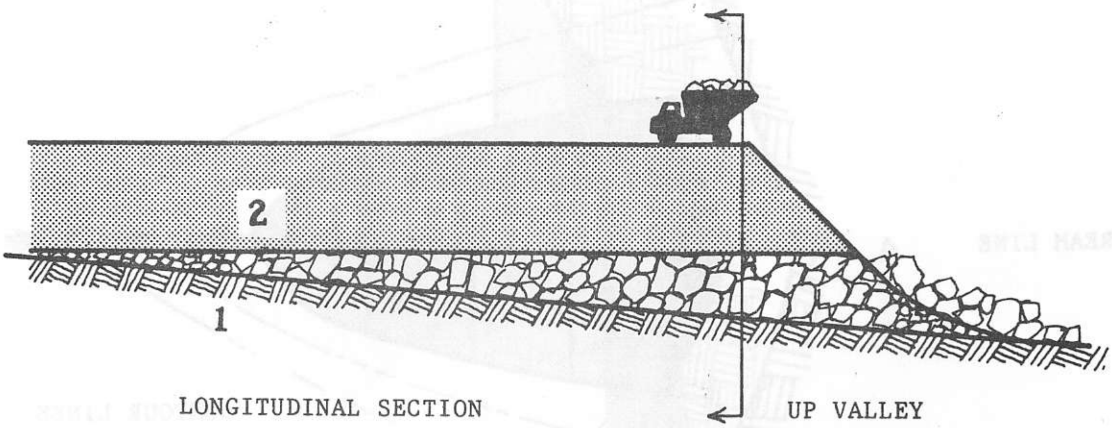
Tests on an active disposal site are generally difficult to carry out and are not very reliable in themselves. Measures are required to ensure safe all-weather operation of the disposal. A large safety margin should be maintained over theoritically calculated values.

Nevertheless, in heavy rainfall situations, the permeability of the whole disposal system may be measured with relative accuracy. To do so the upstream and downstream flows are gauged and evaluated. The comparison of the discharge curves (flow as a function of time) may indicate a possible accumulation of water inside the disposal.

CONCLUSION

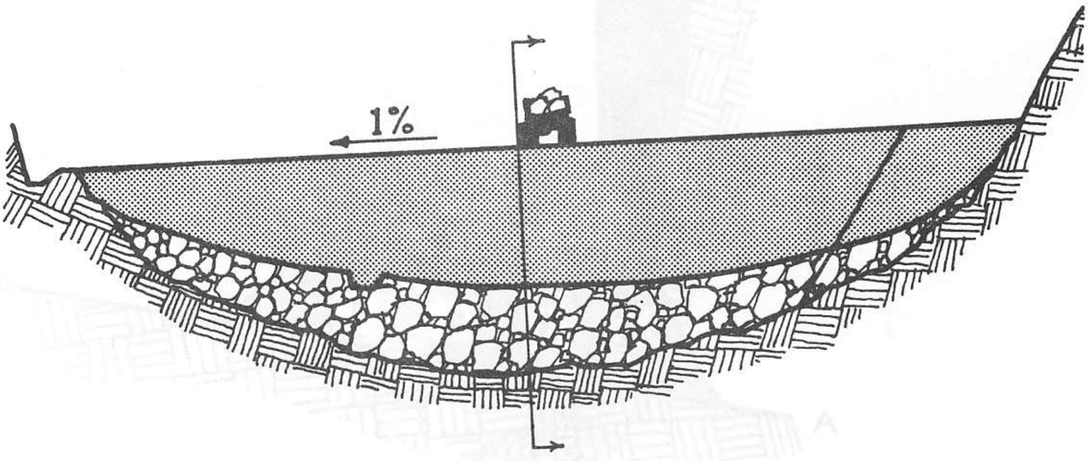
Stockpiling of large boulders required in construction is dependent on proper field execution of the work. Good field control simply consists of avoiding common mistakes. Uncontrolled dumping practices will result in problems being realized only when maximum dumping heights are reached.

FIG. 1



LONGITUDINAL SECTION

FIG. 2



CROSS SECTION ACROSS VALLEY

FIG. 3

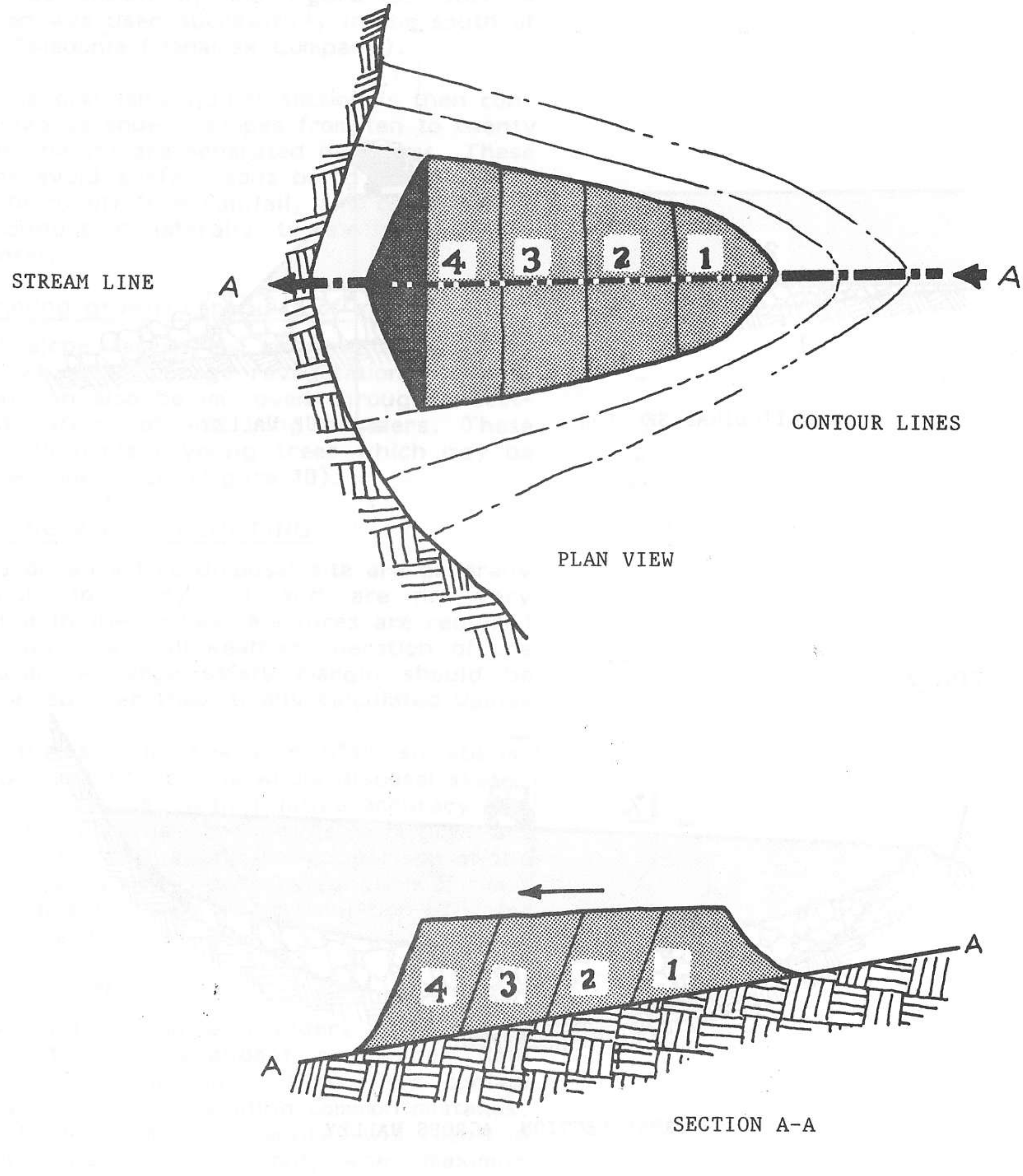
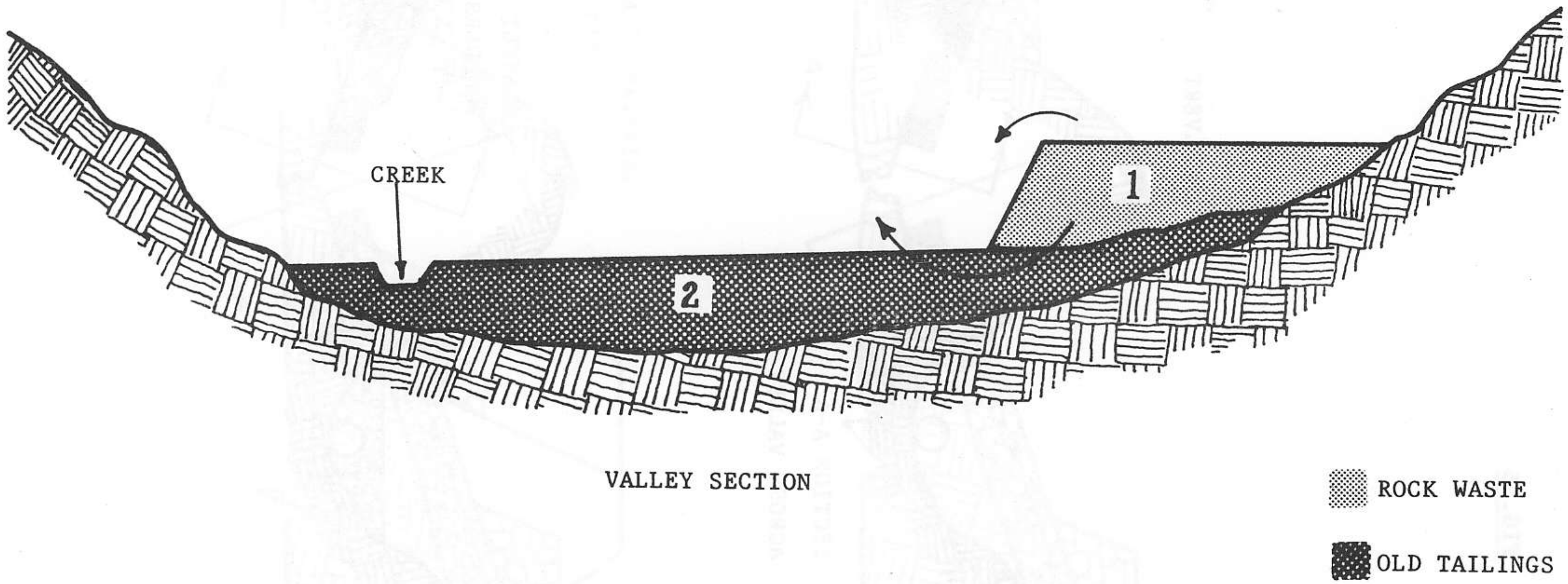

STREAM LINE
A
4
3
2
1
A
CONTOUR LINES
PLAN VIEW
4
3
2
1
A
A
SECTION A-A

FIG. 4



VALLEY SECTION

FIG. 5

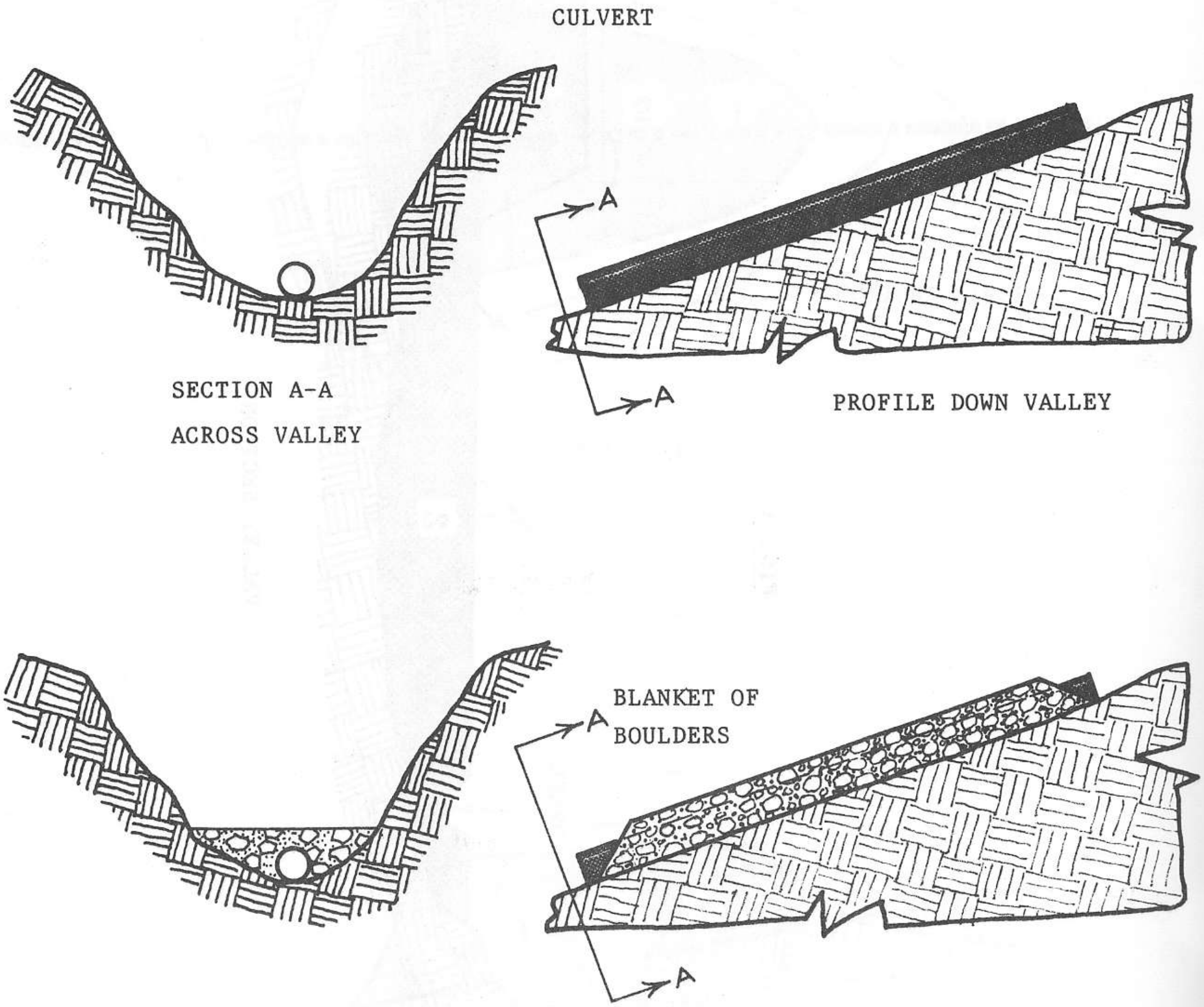

CULVERT
A
A
SECTION A-A
ACROSS VALLEY
PROFILE DOWN VALLEY
BLANKET OF
BOULDERS
A
A

FIG. 6

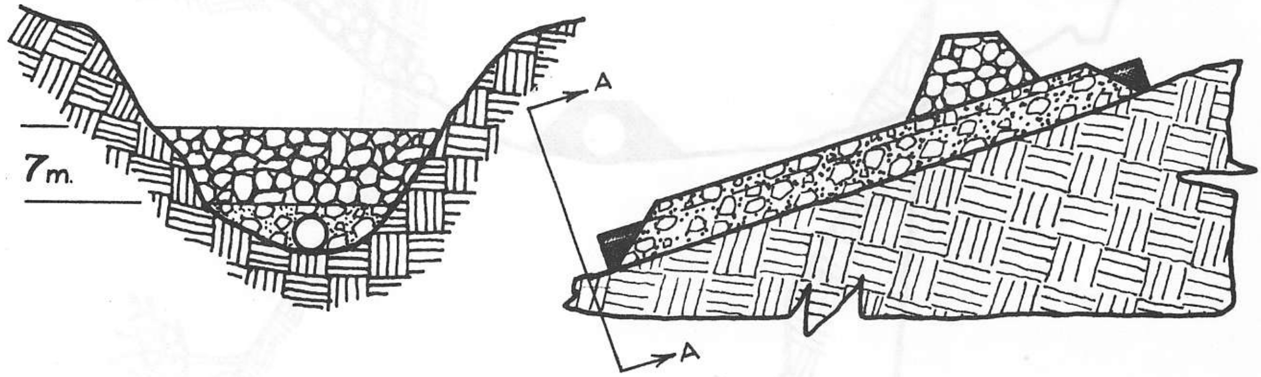


FIG. 7

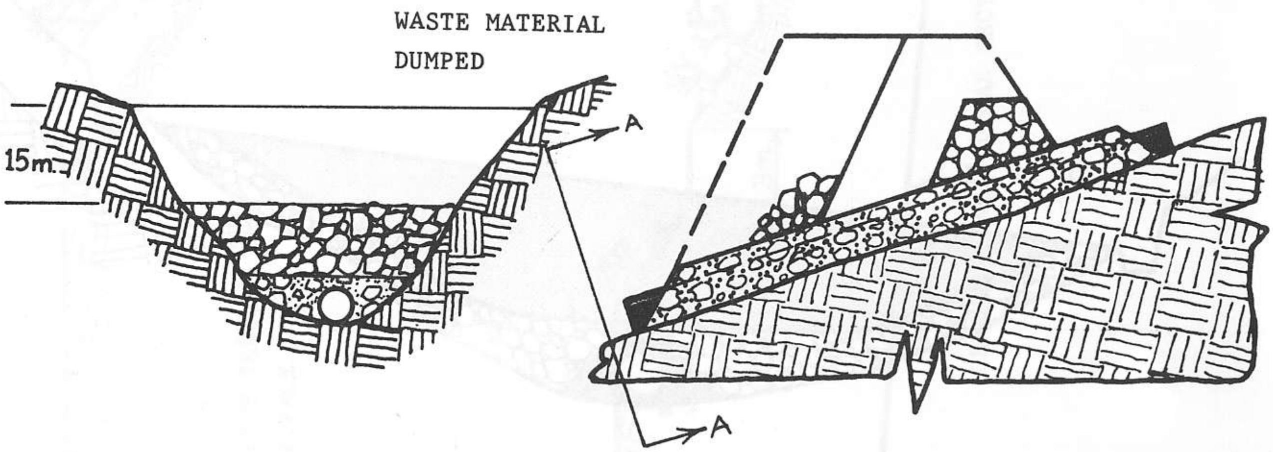


FIG. 8

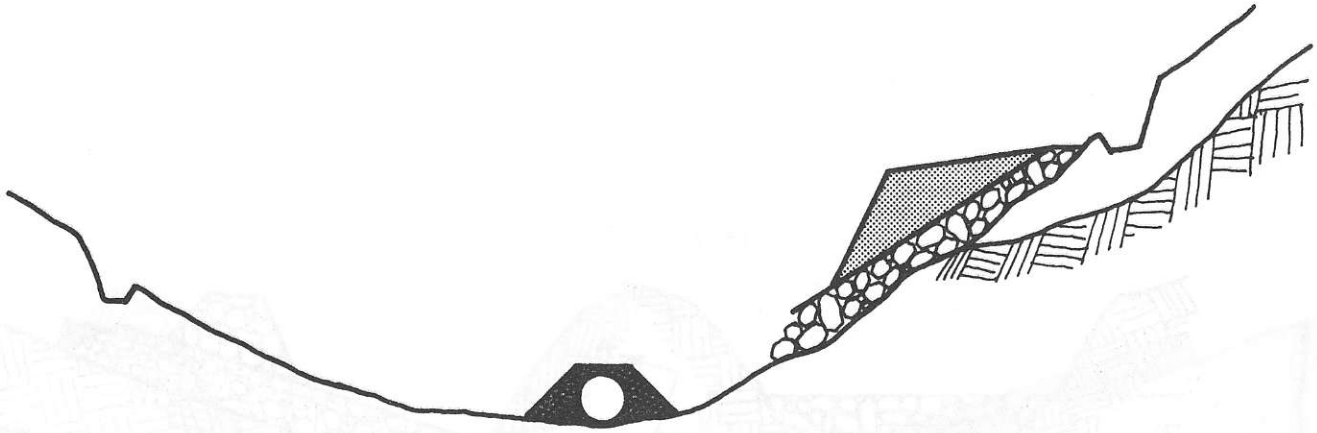


FIG. 9

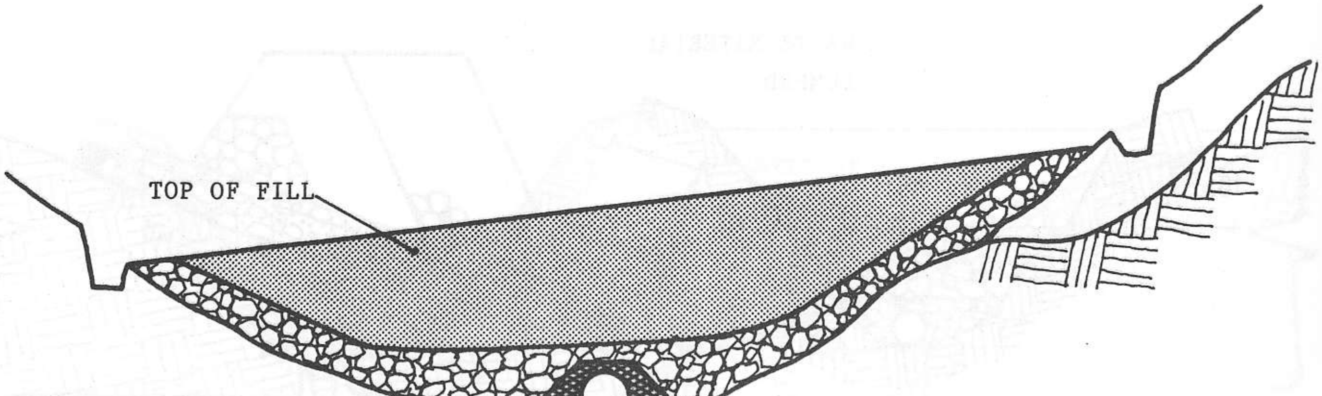
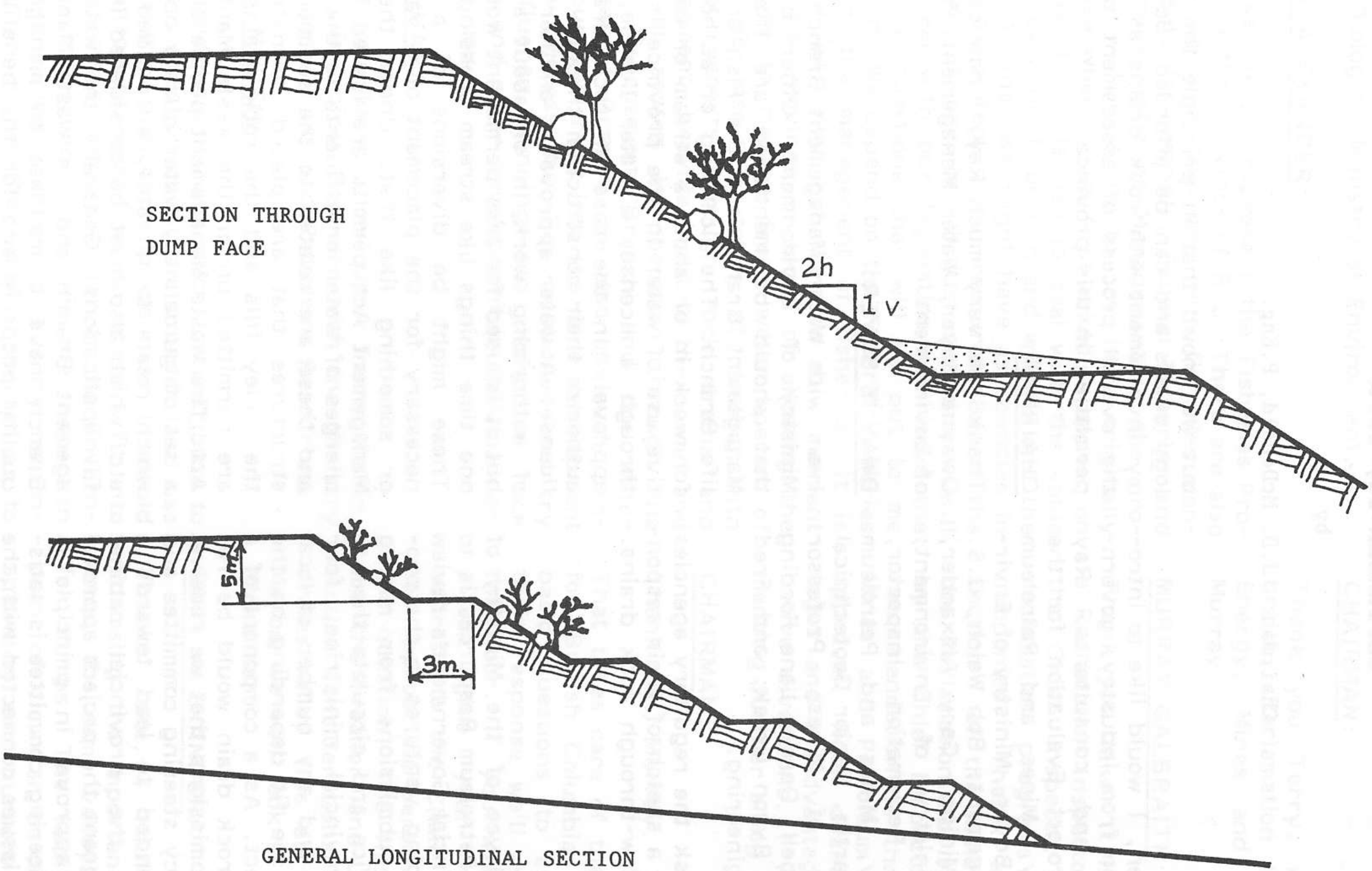


FIG. 10



SECTION THROUGH
DUMP FACE
2h
1 v
5m.
3m.
GENERAL LONGITUDINAL SECTION

PANEL PRESENTATION
ROCK DRAIN SYMPOSIUM
SEPTEMBER 11, 1986

by

Chairman: J.D. McDonald, P.Eng.

CHAIRMAN:

Ladies and Gentlemen, I would like to introduce the panel drawn from industry, government, universities and consultants. Ray Crook, Manager Project Evaluation for the Ministry of Energy, Mines and Petroleum Resources; Dwayne Boyer, Ministry of Environment, Water Management; Bob Welch, U.S. Office of Surface Mining; Gary Alexander, Fish Biologist, Ministry of Environment; Murray Galbraith, Reclamation Inspector, Ministry of Energy, Mines and Petroleum Resources; Terry Martin, Senior Geotechnical Engineer, Ministry of Mines; Professor Lawson; Dave Campbell, Dermot Lane Fording Coal; Rob Nichols, Byron Creek; and Fred Claridge, Piteau Engineering.

First of all I will ask the regultory agencies for B.C. to give us a sketch of their responsibilities towards flow-through rock drains. Firstly Rey Crook.

RAY CROOK

My job as an employee of the Ministry of Energy, Mines and Petroleum Resources is to coordinate the Provincial Government's review process for new mining ventures in the province. We receive submissions from mining companies in my office and circulate them to up to fourteen provincial ministries, four federal departments and any number of local governments as we see fit depending on the nature of the project. As a component of a mining proposal a rock drain would be reported on in the submissions that we receive. I chair a multi-agency steering committee and our review is intended to lead towards a major decision by the provincial cabinet whether or not to grant the project approval in principle. And approval in principle is granted when the steering committee is satisfied that all policy issues connected with the project have been resolved, and all significant technical issues are known to be amenable to economically affordable resolution. We may not have all those details worked out but we known that in principle the technology exists and can be afforded. So that is my involvement wth rock drains as part of the overall process of assessment prior to permitting in this province.

CHAIRMAN:

Thank you very much Ray, I now call upon Dwayne Boyer, Water Management, Ministry of Environment.

DWAYNE BOYER:

I am with Water Management Branch in the Ministry of Environment. Other branches that should be mentioned are the Waste Management Branch and the Fish and Wildlife Branch. The licensing or authorization for work in or about a stream or consumptive use of water in the province is issued through a license, a water license, or on approval. In the case of rock drains we authorize their construction through a water license. A water approval is another method of authorizing work in or about a stream but it is used for less permanent works and one time things like stream crossings, etc. These might be diversions of a stream necessary for the placement of a valley fill or something like that. Under the Waste Management Act permits are issued for discharges of water or effluents to the ground and these are related to the sediment pond structures that are placed downstream of the valley fills and the rock drains. They are permitted under the Waste Management Act. The Waste Management people also have a set of guidelines, water quality objectives which relate to nitrates, suspended solids, etc. which also must be considered in valley fill applications. Generally the water Management Branch and the Waste Management Branch have a mandate to manage water quality properly and for the benefit of the people of the Province of B.C.

CHAIRMAN:

Thank you Dwayne, I ask Gary Alexander Fish Biologist, Ministry of Environment.

GARY ALEXANDER:

Thanks Jake, I represent the Fisheries Program in the Province of B.C. There are also other agencies that deal with fisheries management, for example, Federal Fisheries and Adromous Fisheries Management in the province. I had a lot to say here originally because when I was first asked to appear on the panel, I planned to deal with the value of valley habitat for fish and wildlife and the concerns that we might have as agencies in terms of protecting our resource. But in the interest of brevity I'll just say that I will just deal with our legislation and leave the rest for questions that will be put to me later on. We depend on the Federal Fisheries Act in the management of fisheries. It is federal legislation but there is an agreement with the Province in terms of allowing the Province to manage fresh water species of fish. We also, in terms of dealing with habitat impacts through rock drains and development, provide input to the permit and approval system in operation through the Waste Management Branch and the Water Management Branch. We also obviously prdvide our own input into the mine development review process that Ray mentioned and try to influence planning and design of rock drains and other development in that manner. Thank you.

CHAIRMAN:

Thank you Gary, I call upon Terry Martin, Senior Geological Engineer with the Ministry of Energy, Mines and Petroleum Resources.

TERRY MARTIN:

I'm afraid I don't have much to contribute to the first question about the broader land usa of resource protection guidelines. I should just introduce myself as to where I fit into the approvals process as far as the Government is concerned. I work in the Inspection and Engineering Branch and it is my responsibility to advise the Chief Inspector as to the approval of mining systems which would include dumps and there is no specific approval for rock drains as far as I'm concerned, it's just simply part of the dumps, so there is no specific points that have to be addressed from my point of view to make the recommendation for approval of a dump.

CHAIRMAN:

Thank you Terry, now for Murray Galbraith, Reclamation Inspector, Ministry of Energy, Mines and Petroleum Resources. Murray

MURRAY GALBRAITH:

Thank you Jake. It is part of my duties as a Reclamation Inspector to provide advice to the Chief Inspector on the approval of reclamation programs submitted by mining companies. These are submitted under Section 7 of the Mines Act, for review by the Reclamation Advisory Committee chaired by tho Chief Inspector, who then makes a recommendation to the Minister of Energy, Mines and Petroleum Resources, on their approval. Rock drains have their advantages and disadvantages as does the alternative diversions. I will probably be commenting further along in the panel on this.

CHAIRMAN:

Thank you, Murray.

That takes care of the Regulatory Agencies for British Columbia. We now have three basic questions to ask the panel. After their response we'll ask for questions from the floor. If you wish to ask a question please give your name, employer and address to the Chair and I will give it to the appropriate person I feel should answer the question. The first question is: "What are the broader land use and resource protection considerations in the choice of rock drains/valley fills as a mining technique?" Maybe we will look at fisheries first, Gary Alexander.

GARY ALEXANDER:

Thanks Jake, I would like to begin by stressing the importance of the valley bottoms, the same valleys that are candidates for dumps. The habitat from a Fisheries point of view can be critical in terms of the spawning and nursery areas for fish. It's the kind of thing that is unique and the conditions that make up the valley, the stream habitat in a valley, may not be found elsewhere in the stream. It's not simply a question of the fish being displaced to another part of the stream if that habitat is lost. It may be a question of that population being lost entirely if the stream filling goes ahead. Now when I'm concentrating on fish here obviously I'm representing the Fishery Agency. It should also be mentioned that wildlife also have critical habitat needs in the valley bottoms and particularly the riparian vegetation that exists in valley bottoms may be critical to maintaining populations of moose, and elk. Here again the over wintering habitat may be critical in terms of the maintenance of that populetion. Now, specifically in terms of the impacts that I see from the Fisheries perspective I think you can class them in terms of the inundation effect of infilling and loss, the complete loss of fisheries habitat, and then there is also the downstream impacts and I think the downstream impacts are considerable and should be stressed. There is the impacts of suspended solids and the potential for acid drainage and toxicity in fish. And to an extent you have indicated during this conference that you can control some of these problems through design and construction of rock drains. But it is other things that I have heard during this conference that concern me and these deal with for example; The attenuation affect on freshet flows and what that might mean in terms of the channel configuration downstream. If you have a large catchment area and you are influencing freshet flows what does that mean to channel characteristics downstream. What does it mean too, in terms of scouring, replenishing spawning gravels for fisheries? Also, other problems we might like to address would be the temperature affect: is there a reduction of temperature, water temperature downstream of the rock drain and what does that mean in terms of fisheries?

The question of abandonment I gather we are addressing in questions later on. That is a serious concern to fisheries. I guess that the point is as an agency we'd be looking at the impact of rock drains and the affects in terms of: 1) the value of the fisheries lost; and 2) the ability to mitigate or replace in kind that habitat that is lost. Both those considerations would determine our approval input into the process. I should just mention in conclusion that if we are dealing with critical habitat, habitat that can't easily be replaced, then there are provisions in the Federal Fisheries Act, habitat provisions, that would control work going on in terms of disruption of fisheries habitats in the stream. We would use that if necessary but we would prefere again to influence changes early on in the planning process in terms of design and of rock drains so that the impact on fisheries habitat would be minimal. That's the resource protection considerations from a fisheries point of view. The whole question of land use impacts of the rock drain in terms of wildlife is substantial and shouldn't be ignored.

CHAIRMAN:

Thank you Gary.

DWAYNE BOYER:

Our concerns would also revolve around water quality for consumptive use, domestic use, and irrigation. So in our relationships with the Wildlife Branch, we look at the water quality for both the fish and people use. The water quantity is also affected as we've seen by the rock drains in that in this area there is a couple of positive factors. If you have the damming affect, the attenuation of a flood, will help to preserve flows in the creeks which tend to dry up. This is a problem in irrigation. There may be some positive benefits there and also with flooding and erosion of property, if you can attenuate the peaks, it reduces flood damage. The other major problem or concern we have is the establishment of permanent water courses after the mine is complete (although during is a consideration) and we are looking positively at the rock drain concept if we can leave the stream at the valley bottom and not have to hang it off the valley wall in a diversion ditch or something. It allows the stream to get it's basic energy down and keep it down, and that's positive for us.

CHAIRMAN:

Thank you Dwayne

RAY CROOK:

In terms of real technical reaction the two Ministry of Environment people have made the main points. I would like to think about it in a more general way in terms of the way the government makes decisions about projects. I said that the review process prior to permitting leads to the approval in principle decision and approval in principle is granted when there are no outstanding policy issues and all the technical issues are known to be resolvable. And I think it's fair to say that four years ago, anyway, rock drains were a policy issue. The way the review process works, each agency that participates in it makes it's own decisions and so really we were talking about primarily Ministry of Environmental agency input as woll. And four-five years ago there wasn't the experience in the province, it was considered quite strange technology. And it was a general policy question about whether the government agencies would want to allow rock drains. Now there had been a few operating in a more or less unofficial way on some dovelopments prior to that, I do not want to suggest that they have never been here before. I would like to think that rock drains are moving out of the realm of being a policy question into the realm of being a technical question and it becomes a question of whether we can come up with a satisfactory design. We've started perhaps in a small way. We have somo significant examples that you've seen on the fieid trips recently that have been approved in recent years. We're starting perhaps slowly but definitely beginning to move towards a feeling of greater comfort with them and certainly I get the feeling of much less "a priori" concern from Ministry of Environment. They are more inclined now to treat it as a technical issue that they can analyze in some reasonably readily available technical framework. As far as broader land use and resource protection issues it still remains a fact that rock drains are incompatible with fisheries. If we are going to have a major fishery there is going to be an incompatibility that is going to lead to some sort of a winner/loser situation. You can't have both, certainly, at the drain, probably also upstream from the drain. You could have a problem downstream as well. Listening to some of the papers maybe you don't get as much of a problem downstream as we thought but certainly there is a generally incompatibility at it and upstream. Another issue which I'm inclined to think as still a policy issue is any suggestion that they are not permanent structures. One of the reasons that we accept rock drains in the mine development review process is because they are a logical answer to a difficult waste disposal problem. If you can use them, if the stream situation is suitable, if thero is low to nil fishery values and other foundation conditions are OK and whatever, then it often makes sense to have valley fills rather than try awkward valley site dumps and stream diversions and this sort of thing. And we are going ahead with the ones we've gone ahead with on the basis that they are permanent structures. If there is any question that within a reasonably short period of time their performance is going to start to deteriorate then I would have to be concerned with that. I know at least one speaker made a major pitch in that direction during this conference. I also heard the opposite argument. But to me that remains a critical outstanding question mark.

CHAIRMAN:

Thank you Ray.

I'd just like to comment that there is no alternative to valley fills. If you can't have them you're not going to have mines, because we're looking at disposal of over 150 million bank cubic metres per year of waste material in the mines in the Kootenay area. We'll now go on to the second question which is: "What Geotechnical considerations are necessary in rock drain design and construction?" I'd like to ask Terry to comment first of all.

TERRY MARTIN:

Well as I indicated to you before hand there are really none. That's the answer in brief. They are treated just as part of the dump and we've prepared a document known as the Mine Dump Guidelines or something to that effect I'm not sure of the exact title we've given it. These were put out in 1985 I believe and there is some evidence to indicate that there may be a few mines that have seen them and recognized the requirements that are embodied in them. That sums it up.

CHAIRMAN:

Thanks Terry. Dave gave us a pretty good talk this afteroon on what is necessary in rock drain design end construction so I'm going to turn this over to Dave and let him comment on the question.

DAVE CAMPBELL:

I think if you are considering development of a rock drain one of the things that you have to look at initially is is there a source of rock that is durable, is it going to be chemically compatible and if you can't find those two things then the situation gets pretty tough because those are necessary to the design of a rock drain. Assuming that there is durable rock available it's chemically inert you're going to be concerned with rock strength depending on the height of fill that you may have over the rock drain. You're going to be concerned with rock size and sometimes this is not an easy thing to assess prior to either some quarrying or some mining operation. The rock size that's produced by the normal blasting is going to be controlled at least to some degree and in some cases a significant degree by the joint spacing in the rock mass and the thickness of the sedimentry units. Even if you make a survey of joint spacing and thickness of sedimentary units you may not have the answer because there may be some micro-fractures that will open up and reduce rock size on blasting and these micro-fractures may reduce rock size still further in the course of transit of the large blocks down the face of the dump to the position where the rock drain is going to be. Other factors that you are going to consider are gradient which in most cases, is going to be equivalent to the stream gradient but there may be cases where you're going to be considering gradients that are somewhat steeper than stream gradients. You're going to have to make an estimate of void ratio because void ratio and rock size together with the hydraulic gradient are going to govern the through-flow capacity of the drain. Rock size and void ratio are something that's very difficult to get. I think the design, the approach that you take is that you have to be conservative, you have to be conservative with size and I think that you should assume a size somewhat smaller than you see at either end of the drain. Rock sizes are going to be somewhat smaller if you cover this rock drain up with any thickness of fill, rock sizes are going to be somewhat smaller than you see go into the drain before it gets covered. At the same time void ratio is probably going to be somewhat smaller, so I think that you have to be conservative. And there is a degree of conservatism in the type of rock drain thats formed by end dumping from the crest. The prime example we saw the other day is the West Line Creek rock drain. The cross section is very generous. There is I think, a great deal of conservatism in that rock drain and that's not by design. That happens to be the type of rock that Crow's Nest are blessed with, that's the way it separates, the dump height is such that there is a large amount of material that accumulates at the toe of the slope and that's a situation where there is a large capacity relative to what we think is going to be required and I might say that I mentioned that I think that rock drain has capacity of something like 9 to 12 times the anticipated 200-year flood. I'd also like to make the point that in the event of the 200-year flood, what will pass through that rock drain is going to be less than the 200-year flood because there will be some ponding on the upstream side and the ponding is going to attenuate the curve so even if you wanted to get the 200-year event through the rock drain, if you wanted, if you designed the drain with a capacity equal to the instantaneous peak for the 200-year event, that peak would never go through the drain. What would go through the drain is somewhat less than. So having said that about the type of rock drain that is developed by end dumping from the crest, the other type of rock drain is the one that is formed and I think that there is a great deal less conservatism in the rock drain that is formed. If there is not then the drain is going to be very expensive. In the formed rock drain and I am taking for example the formed rock drain that parallels the real rock drain at Swift Creek. It has an upper boundary on it and it's somewhat analagous to putting a flow of water through a culvert when you get water up to the top of the culvert you aren't going to get much more through it. That's it. Whereas the capacity of these rock drains that are formed by gravity separation, the water can rise metres if it needs to. The rock is so pervious there is a lot more safety I think in the type of drain that's born by gravity separation or end dumping from the crest, than there is in the formed rock drain.

CHAIRMAN:

Thanks very much Dave. Maybe Fred you'd like to comment on that question.

FRED CLARIDGE:

I just wanted to add perhaps a little on hydrology in discussing the geotechnical considerations. As a Geotechnical Engineer I get carried away a little bit myself but we shouldn't forget the hydrologist. First of all, I think that we're all becoming more and more comfortable with the particular rock drain applications that have been discussed in this symposium. Particularly at Line Creek, at Fording and at Byron Creek, and I think we should remember one thing and that is the rock types are quite similar. There is a lot of exchange of knowledge and data from mine to mine. What I would caution people is that if they go too far afield, particularly into new rock situations to be careful. The other caution I would like to mention is that we are dealing with fairly small catchments in all of these rock drains and technically they are, I think all less than 15 square kilometres, obviously there is an upper limit that one would probably would not want to have a valley fill and a rock drain on. I don't know what that limit is, it may be a lot higher than we're talking about, but we shouldn't extrapolate the knowledge from the Elk Valley without some cautions being applied. One other aspect just following up from Dave's comment that the instantaneous flow is probably excessive, what we normally have indicated is a mean daily flow in a rock drain design which we feel fairly well accommodates the peaking and you're talking about a reduction to somewhere probably in the 50% of peak flow when you do that and that helps eliminate some of the conservatism. Just another point that I thought I'd make is, and I mentioned that this morning, is that many drainage systems that a designer looks at have to be workable from the mining point of view and it is absolutely imperative that the miner be involved with the geotechnical engineer in the system design because whatever the geotechnical engineer does if the mining engineer does not believe in it or does not plan to follow it then it may not work.

CHAIRMAN:

Thank you Fred.

DERMOT LANE:

I guess my only comment from indstry's standpoint is carrying on from what Fred said, it is important to us when we are constructing these that we have a good monitoring system to watch the material that's going in to make sure that as we are constructing the rock drains that the quality control is there and we're building them as specified and I think we all recognize the dangers of putting poor material in there. I think it is important from our standpoint when building them that we can't afford to have an accident happen. Being in Water Management, Dwayne would you like to comment.

DWAYNE BOYER:

Just maybe a word on the licenses that we have issued so far, and it pertains to the question. Why we would have issued these licenses if we had any concerns about the longevity of the rock drain? I think the discussions of the previous gentlemen have just hit on the fact that the justification for licensing is in the generousity in the cross sections. The ones we've issued you stand and look down 100-150 metres to the bottom of the valley and there is a little creek yeu can hardly see. So these conservatisms allow us to proceed and issue these approvals. If the whole dam was to break down all the rock would break down and you don't have the hydraulic capacity. Fred Clariage has just mentioned on the Byron Creek one you have enough volume upstream to contain five years of discharge that's just a point on that. But also as a caution on extending this information, there is, has been talk or discussions about creating formed drains beneath coarse whole refuse. Now we're talking about a drain that has to function as a drain, it doesn't have a multiple or generous cross section, so we have to depend on its longevity and it's flow-through capability into the far future so the questions are still not answered because it's not a large end dumped fill. And also I would just allude to a question that probably I can't answer. It is about discharge of the equivalent to the 200-year flood. Its common design practice for spillways for dams but we also consider the dam has to be safe under the probably maximum flood so I think that deserves consideration here. Are we talking about an equivalent design? Should we be designing for the probable maximum flood as well?

CHAIRMAN:

Thank you Wayne

BOB WELSH:

We are addressing Question #3, is that right? What is your opinion on the question of long-term permanency of rock drains in relation to the migration of fines and the durability of rock used? Well, this may not be the appropriate place to bring some of these concerns up. There are some things

I've jotted down I've been wondering throughout the process of the conference. Obviously as a foreigner, a flat-lander in essence, I've seen some different conditions here than I usually see in the applications. In a way this is a good test case for us to look at because we have built in factors in our design which I see in varying amounts here but its not a locked-in system in the regulatory apparatus as we have in the States as I see it. There are some considerations in our regulations that are included in some fills I've seen here and others that I've not. One of the issues is grading of the surface of the fill to lower angles below the angle of repose and that is to allow revegetation. Again terracing the surface of the fills, particulary larger fills for water drainage control on the surface. Another issue is the use of diversion ditches around the perimeter of the fill again with the idea of controlling the flow of surface water over and into the fill. And another issue which appears to be dealt with is foundation preparation in the underdrain phase itself. And one concern I have personally regardless of geographic area is the use of a high enough percentage of durable rock within the fill that not only covers underdrain materials but also throughout the fill. I guess being in a federal agency we tend to think in terms of the worst case, but I can see situations where you can possibly have a higher level slope failure which can come down and actually smother your underdrain inlet and that's why we have the concern of not only good rock down low in the fill but also a good quality rock at higher levels in the fill as well. And also controlling the amount of water that's infiltrating into the fill, particularly along potential failure surfaces. I think Dave Campbell who's site box model of the fill showed you could develop these layers of fines which are parallel to the slope and to me you happen to wet those surfaces it appears to be a fairly good failure plane. Another concern I've had just in listening throughout the past few days is the effect of humidity levels within the fill with the constant or intermittent flow of water throughout the fills. Now what is that doing to the structure of the rock and the fill. Perhaps the alternating of wetting and drying, what's that doing to the integrity of the rock. Just some thoughts, like I say in some cases they appear to be addressed, other cases not. I haven't reached any solutions in my own mind, I think in the States with the regulatory situation we have, these factors are filled in. Here it appears to be a new enough practice where we're still in a state of flux. And now we are going to be watching this situation very closely indeed, and I think it's very interesting.

<u>CHAIRMAN</u>:

Thank you Bob

<u>DAVE CAMPBELL</u>:

I'd like to comment on the suggestion that the fine material, the layered material that's parallel to the dump face is a potential shear surface. Material comprising those fine layers, mineralogically is about the same as the material in the rest of the dump. Getting it wet does not change it's friction angle and when you get any significant depth below the surface of the dump, the water is not under positive pressure it is under negative pressure and that adds apparent cohesion which does contribute to stability. Your point's well taken but I don't think that its applicable in this case.

<u>CHAIRMAN</u>:

Thanks Dave

The Panel is open for questions from the floor.

Ralph McGinn, Ministry of Energy, Mines and Petroleum Resources, Victoria. I jotted down a few things which I think somebody on the panel could indicate the importance of or the need for during design and construction and one of those things had to do with the cooperation between the mining crew and the planner in relation to the blasting pattern that's used which determines the size of the material and the powder factor that's used. Another thing was I didn't hear anybody talk about foundation investigation prior to design and construction. I'd like to see how important you consider that is. The crest height, how important is the crest height for construction of an end dumped drain, is it important to have it 30 metres up or 60 metres up or 150 metres. The upstream face of the valley fill, should there be any special precaution taken there, should we assure that the coarser material at that upstream face has a little higher elevation than it does throughout the rest of the dump. And snow on the dump surface which is dumped over and possibly scoured down and areas where we have a high level of snow fall how important is that in its effects in the drain? Monitoring of the dump itself, If you have a failure and significant amount of fina material from the top runs down into that drain you may have a problem of where you're going to place the drain. What level of monitoring would you have on the dump, at the dump itself? At what movement rates would you consider shutting the dump down because you felt the movement might lead to failure and jeopardize your drain? Just one more point, and that has to do with dumping rates per metre or crest length, how important is that on the stability of the dump?

CHAIRMAN:

Is that all Ralph? Well, I guess we'll start off by asking Dwayne, Dermot or Rob to comment on the first one, which is cooperation of the mine crew and the blasting and what is the powder factor? What's your experience.

ROB NICHOLS:

Regarding the powder factor and the cooperation. Typically of the mines around here the engineering department is intimately involved with the design of the blast patterns so that they do have most of the control over what is drilled and blasted and the size of material produced. Now the size of material produced is not going to be governed by a rock drain typically. You are going to govern it by the equipment you are mining it with and whatever suits that equipment is what you're going to get. What you have to consider when you are looking at forming a rock drain from natural segregation you've got to know a size gradation of the typical blasted material.

DERMOT LANE:

I don't think I can add much more to what Rob just said. I can just say from my experience that we've done at the operation where I'm employed, we've used the normal operating procedures and we've taken advantage of those to build rock drain structures. We haven't had to get into custom blasting or such things and it's fairly obvious that we minimize fracture to the level as Rob said, where we can handle it with the equipment. We have no intention of making dust out of that stuff. It just costs us money. So again, the mining equipment really limits what we can do in terms of making the blocks large and we obviously don't make them any smaller than we have to. Blasting is optimized as much as possible.

CHAIRMAN:

The next items -- foundation, crest height, upstream face of the valley fill, snow on dump surfaces. Dave could you respond?

DAVE CAMPBELL:

As far as foundation investigations go, it maybe wasn't mentioned because it's so common. Certainly if you're going to build a rock fill over a drainage course, that rock fill is a dump, the toe of it will become the rock drain, the foundation investigations have to be carried out, stability analysis made and you want to be reasonably confident that the dump is going to remain stable. I would be the first to admit that you could be caught out, sometimes things can develop and failures do occur that you don't expect to occur and they occur under conditions that you may not have anticipated.

These dumps have lower factors of safety than are common in engineering practice, sometimes you get a failure. Now I don't know if anyone pointed it out when we were looking at the West Line Creek dump, there is a failure there that has occurred recently. There was also some slide debris in the bottom of the drainage course, as a result of a failure that occured on the 4th of July, 1982. I don't think that that previous failure in any way detracts from the function of that drain. The failure occurred near the upstream limit and it is probably going to be responsible for raising the water level in the drain in the upstream region. But the water surface in the West Line Creek rock drain is going to be controlled below that point primarily by the gradient along the drainage channel and the debris flow that went down the drainage channel as a slope just about the same as the original drainage course. And when the water gets down to the downstream toe conditions there are not going to relate to what happened in the upstream portion because it is so far removed. I would venture to say that you could put in some fairly precise instrumentation and you couldn't detect the difference between one and the other. Now the failure that had occurred recently certainly it's going to do something to the water level in the drain. The dump is going to go further downstream but again I think within the toe region of the dump, we don't want to have another one of those. But it's not going to have a significant impact on the function of the drain.

As far as dump height goes, the dump height has to be such that you get segregation and the volume of material at the toe that's going to do the job that you need. I think by selective dumping that, you might use the best quality rock you've got in the upstream region and increase the height at which the permeable zone extends above the inlet of the drain. As far as winter construction goes, it might not be appropriate to incorporate a lot of snow at the bottom of the drain. In the case for example at the Swift Creek Rock drain, it was constructed in the winter time. But there is no stability problem because the toe advanced on to the other side and the toe had a high degree of support. There's no possibility that the face of the dump was going to fail. I don't think there is anymore snow left there, its disappeared. It's possible that snow in the rock drain may have contributed a small amount to the deformations that have occurred on the surface of the cross-over fill. I don't think they have increased the deformations significantly. Most of the deformations that have occurred there are the result of compaction of the waste rock under self weight. I'll let it go at that.

DAVE CAMPBELL:

I think I know the mine which might have inspired some of Mr. McGinn's questions but I don't say anymore. As far as blasting effects there's no doubt that increased blasting can increase the fragmentation but I don't feel that is a serious problem. I feel the serious problem can be where rock just naturally degrades into fines and it's doubly important where a drainage course must serve as a rock drain that only high quality rock be placed in that area of a dump and I think just about every mine has enough flexibility such that it can arrange to put high quality rock into the key areas. And in that respect it is important to keep a number of dump faces open at one time to give the miners the flexibility to dump poor rock where it's not going to do any damage if it should fail and keep the good rock where it is really needed. On the subject of crest height, I think the feeling that we developed this morning Rob might want to comment on but the preferred height of dump for maximum segregation is in the order of 30 metres for the materials in the Elk River. I just want to finish up what I had to say as far as snow over a dump surface, there is no question, if any area of poor rock with high fines has snow in it we have seen cases where snow has been trapped over a summer without melting and then we feel it has rapidly thawed, usually after a rain period, and in that case you have really an excess of water and you can have a sudden and catastrophic failure. So again the advice I would give if you're worried about that situation developing, you can't necessarily prevent the snow from blowing ovar a crest and accumulating. If you are worried about the effects next summer then it's absolutely imperative to keep the rock as coarse as possible and minimize the possibility of excess pressures developing. The other aspect, poor material and the limiting rate in terms of rate of dumping per metre of crest length per 24 hours, some numbers have been suggested, 100-150 banked cubic metres per metre per 24 hours as being a limiting rate, but I think that's just a very general guideline and in some cases that maybe far too high when other factors are present. And in other situations it may be too low. That's just a feel that we've developed. Rob you may want to comment yourself on the limiting height of dump because I think that's very important for maximum segregation.

ROB NICHOLS:

OK that's something I really haven't measured but in my little truck and shovel experiment when you look at the potential energy, the particles in the gravel study and compare them with the potential energy of actual waste dump fragments I estimated for my long slope an equivalent height of an actual dump would be about thirty metres. And for my short slope it would be about 15 metres and as you saw there was a significant, somewhat significant reduction in segregation in the short slope from the long slope and just from what I've seen from around the valley I would say that in a 30 metre high dump you would be guaranteed of good segregation. Anything less than that you would get segregation, yes, but it probably wouldn't be as good as in excess of 30 metres.

CHAIRMAN:

I think Ralph, all dumps are monitored, they all, I guess have various practices. It might take a while to go into the detail on each one, I would suggest that you could get that information off them later. The dumping rates, I think we all know that if your dumping rate is increased the chances of failure at your crest increases end I think all companies are aware that the dumping rates have to be looked at and any signs of movement from your monitoring you move elsewhere to dump. Are there any other questions from the floor.

QUESTION FROM THE FLOOR:

My name is Mark Strosher, with the Ministry of Environment in the Waste Management Branch in Cranbrook. The design considerations that you have been speaking of, as I see it, come mainly from the industry and consultants and the "what if" questions are posed mainly from the regulatory people. As Mr. Campbell has pointed out, some of these design considerations are conservative and are designer built in to the system and some are by luck. On the other hand on the regulatory side from what I'm hearing, and from what I know of the system, the design considerations are minimal and cause a lot of uneasiness on the government side. I guess that's part of the reason for this symposium of course. What I'd like to get some feedback on, is where do we go from here and what might be needed for the efficient review, approval and construction of rock drains and this is from the point of view of industry especially and I'd be interested in some of the feedback that might sort of get this in a more standardized light and make it more of a science as opposed to an art and sort of consolidate the various points of view that we have here.

CHAIRMAN:

Thank you Mark, I think you're getting into probably the next question. But I should tell you that there is a committee getting together tomorrow morning, a small committee, to review the proceedings that have gone on here, and make recommendations on where we go from here and hopefully those recommendations can be put in the proceedings. Is that OK?

Charlie you have a question?

My name is Charlie Ripley, I'm a private citizen and I have come here as a retired civil engineer with extensive experience in the storage dams field and I would like to make a couple of observations if I may, Jake. Firstly, it's been a wonderful experience for me. I have a question that relates to geotechnical considerations. I'd like to make the observation that Chuck Brawner yesterday gave an excellent outline of geotechnical considerations as they would relate to these dumps and the drains as I would see them. My comments, the reason I got up to make a few comments, is that I think that, as Dr. Chappell has indicated, that there is considerable experience in the storage dams field that relates to this and I think it's being utilized. I would like to draw attention to this, from my observations. One is that there is great comfort in a new area as far as the engineering goes, as long as the observational method is used. Now all the geotechnical engineers present understand the observational method. It requires proceeding with a design on the basis of the best assessment of the information conforming to accepted engineering practice. Where you're on the leading edge of technology, you can step beyond the bounds of precedent as long as you, according to the observational method, anticipate some of the "what ifs", and if they happen, you have developed some concept of dealing with them. Now for those who are interested and aren't geotechnical engineers, there is an excellent paper prepared by Ralph Peck, in which he outlines the observational method and step by step the requirements to have to use it. Now it relys largely in taking a step forward on monitoring. Now with regard to these slides that may come down, supposing a slide comes down in one of these rock drains developed by segregation rather than the placed one. By the theory of the observation method that would require undertaking a "what if it happens" procedure and carrying on with the drain. The observational method requires then a pre-thought assessment of what you will do, and what the consequences might be, and how you could make corrections. From what I have seen here, I have the feeling that if that is followed, that if some anticipatory thought is given, eventualities can certainly be dealt with. After the "what if" has occurred there are ways of getting around and ensuring the continuity of the drain in the continuing construction. That's just an opinion that I have gathered from this briefly. The one thing that I feel that there may be some digression from this observational method, which is essentially being followed, is with respect to the monitoring in the early stages and through the work, where it can be conveniently done with respect to use of piezometers or observation wells into the drain to know better what the water levels are. And I think that that is something that should be considered. Thank you.

CHAIRMAN:

Thank you very much Charlie, appreciate your comments and I think that we all should take a look at what our monitoring is going to be.

BRIAN CHAPPELL:

This question is basically to Dave and the genetleman on the end there and possibly the others, we've noticed in some of the dams that we have built and again noticed on the trips that we've been on that the dam toe is the problem, the toe drain, and to stabilize you'd take a lot of care of that, it is basically your stiffest part of your whole structure. Now I'm looking at this as a structure. The structure as a whole is on an angle, 5, 9 degrees, whatever, we've had failures at 5 and 9 degrees, so it's not out of the realms of slip and things like that. The thing is that you're putting the loose dumped rock, it's got a lot of deformability, it starts to deform and it shifts its load through arching and then it starts to throw its load onto the toe. And when the toe then starts receiving this load it starts to show distress and then the observational characteristics come in. So you've got a beautiful technique but there is this problem.

DAVE CAMPBELL:

That's interesting that you should mention that the load shifts to the toe. I've been observing dumps for quite some time, looking at evidence of deformation on a face and often that's not easy to see. Light conditions have to be right to see patterns and that's generally what you see are patterns. I've come to the conclusion that one of the common modes of deformation is just what you have indicated in that the stability of the whole dump face depends on the stability of the toe region and if you can hold the toe region stable the whole dump's face will remain stable, at least you will not have a failure. As far as the stability of the toe where these rock drains exit, I think that what we have to do is to do flood routing analyses. We have to make the best estimate that we can as to where the phreatic surface might break out on the downstream toe. I think that the method of controlling it, making sure that that remains stable, is placement of coarse rock fillets and by the way, I think that these fillets should consist of rocks that are picked up individually. That's not an easy thing to do. The commonest method of going to pick rock, you're going to find it at the dump toe someplace, the commonest method is with a front-end loader, but it is difficult to pick up a large piece of rock with a front-end loader and npt get some small stuff with it and I think this fillet, and this is my feeling, I think the fillet should be as devoid of small sizes as it is practical to produce.

NO NAME:

My comment would be there is a different element of risk during construction of a dump downstream. There is a higher possibility of a failure occurring because you cannot control what goes into the dump and I think the point that was made is excellent as many failures do occur, bulging at the toe, water pressures, poor materials, are the causes, however, when it comes to forming the ultimate downstream slope, the mine has a lot of control how that is going to be done. It can often shape the dump, it can put in a number of berms at an overall slope, whatever that is that's considered to be stable, and it also has the control over the rock that goes in there. So there is flexibility to construct something that will remain stable for hopefully, as long as anyone cares.

CHAIRMAN:

So the question of permanency -- what is permanent, nothing, I guess to define permanent as something that remains constantly the same. We all know that nature is such that it keeps working and nothing is permanent. But the degree of permanence I guess is what we are looking at. So the third question is "What is your opinion on the question of long term permanency of rock drains in relation to migration of fines and the durability of rock used?" I'm going to ask members of the panel for their opinions on this question. I don't know if everybody wants to comment but I'll start off with Professor Lawson here.

PROF. LAWSON:

Thank you Jake. Depsite the fact that I'm the only one present who can neither speak english or french I've got three comments to make on this question. The first relates to the presentation yesterday afternoon by my fellow academic Professor Brawner. It's a great pity that he's not present at this session this afternoon so that we could engage in some of the discussions that he made. He made, yesterday, several very generalized adverse comments about the use of rock drains, as you will recall. His statement about the high velocities of through drains made it clear to me that he had not had significant experience with flow-through rock fill. For example, Dave Campbell and others today, have shown quite oonvincingly I think, that the velocities that one gets through a rock drain are somewhere between point 15 and point 4 metres per second. Those are about the norm. And that in fact if you have increased flows as Dave Campbell indicated, you simply increase your cross-sectional area. Your velocities don't change too much. That's just a comment I think that needs clearing up and it's a pity, as I said, that Professor Brawner wasn't here to either refute that or make comment on it. My second comment is to do with monitoring and bears out what Charlie said a short time ago and this is simply a plea that I would make. A plea to install some piezometer wells or piezometers of any description but it seemed to me that the wells that we saw the other day, are pretty simple and pretty robust, to answer any potential critics of the future in relation to rock drains. Simply an observational method and it seems to me that as and when a rock fill becomes stabilized and there is no chance of any other rocks hurtling into it, it wouldn't be too difficult, once every few months perhaps, to install a piezometer. Anyway, that's just a plea that I would make. My third comment relates more particularly to the question that's been posed and relates particularly I guess to migration of fines, something I've gone on about all week. I'd like to refer to Dave Campbell's absolutely excellent presentation before coffee where he dealt with this and other issues. And I'd like to say for a start, that in relation to the sediment that might come into the rock drain from upstream, his suggestion, I completely agree with, that even if the worst comes to the worst and there is a sediment build up you might almost regard it as a build up behind an inbuilt spillway dam, a thing I was talking about at the beginning of my presentation. And what would happen if there was a build up is that the flow would simply go over the top, fall freely through the rock fill and carry on as usual. So that I don't believe it is of any real great concern unless you have tremendous sediment loads which of course in the small catchments you're dealing with usually isn't the case I would suggest. The other question about migration of fines relates to the grading curve which is on the overhead. I thought that Dave excellently expressed and satisfied one of my queries about the migration of fines down through the bank by illustrating what happens in relation to curves A and B where he said they were reelly acting like a filter and in that respect I am quite satisfied that you have got an effective filter if that's the sort of grading you have on your bank. My only query now end I can now put it quite firmly to Dave I suppose, would be the tail end of curve F what happens to that? The bottom right hand side of curve F.

DAVE CAMPBELL:

I didn't investigate what happens to the end of curve F in the model. Curve F I think is somewhat finer than curve F would be if you went about sampling the toe region of the dump before it was covered. What we have to go on I think is, for example, what we see in the Swift Creek Rock Drain, very coarse material. It does contain some fines, when I say fines, they are smaller than the normal 1/2 metre to metre size rock that makes up the predominant size of that drain. Some of those pieces, the smaller pieces, are shatter fragments. Large blocks come down with considerable energy, end part of that energy is used up in smashing themselves, and smashing the blocks that they impact on. After we cover up the drain, I think we have got a further reduction in size. I think that what we see at the base of #1 spoil, there is an opportunity where a spoil is being taken apart, it's not easy to get information, because it's a mining operation, the excavation's done with a shovel, it excavates a nearly vertical face. While production is going on material falls down, you don't have the opportunity with every shovel full to see what the base of a waste rock fill is like. We are not yet to the point where that excavation is crossing the drainage course which forms the main drain but nevertheless, the single photograph that I did show I think indicates that in the field, curve F doesn't have a lot of fine material in it.

CHAIRMAN:

Thanks Dave, I'll just toss this microphone down the table and if anyone wants to comment please do.

DWAYNE BOYER:

Some of the answers have been provided on this, especially the migration of fines, but durability of rock is still a question. And I'm just brain storming, talking with people around here, I think investigating the natural occurrences of rock drains may be a way of going but that may be something impractical. I'm not a geologist but there's a lot of talus slopes around that may have the order of magnitude of loading of rocks that could be carbon dated and examined. It's just a suggestion. And I've talked to a couple of people who know of naturally formed rock drains. Mountains which gave way and covered valley floors and the rivers are going through. That might also answer some questions. I guess one other question or concern that we have as Ministry of Environment, seeing how we're talking about dumps, is we're seeing a relatively large amount of sliver failures in the early stages of the dump. I hope that these won't happen until later stages of the dumps where you're progressed to the point where one of these failures will innundate the settling ponds and the structures of the front to retain the mud flow. And maybe at that point the dumping can be controlled to a point where the dump itself is progressed so that these sliver failures will be less frequent. I wonder if someone can comment to that if they want please.

CHAIRMAN:

Didn't get much action at that end of the table I'll try this. I'll send it down to you Fred and work back from there.

FRED CLARIDGE:

I agree with Dwayne's comment as the dump reaches its ultimate geometry that more care has to be taken in what goes into the dump and providing the planning is done well in advance, the mine should not have a great deal of difficulty in ensuring a favourable geometry and favourable rock goes into the final downstream face and if there is a sediment pond immediately below the dump and clearly there has to be special effort put to that end. What effort and expense is put into the end of construction has a great deal to do with the risk element attached to a failure. A small sediment pond, such as the ones at Line Creek, probably would not be a catastrophy even if a failure did reach one of the ponds. Whereas in our view, failure, a major failure, reaching the Byron Creek sedimentation pond could cause an over-topping and a sudden release of water which is not acceptable. So I would advocate in as precise a fashion as possible a risk analysis be done at various stages of dump construction and sensible judgments be made based on that as to what degree of cost and planning should go into dump construction.

ROB NICHOLS:

I guess my only comment from the Industry point of view is that we want the rock drain to last as long as we are going to be there and beyond that we want to play by the rules, so whatever the rules are we want to know, them.

DERMOT LANE:

I think my comments are really Rob's thoughts in some respect. I believe that the goalposts have not yet been clearly established for rock drains. I suspect a large reason for that is that none of us have even been forced to deal with these time frames before this type of structure. I think In particular some of the concerns should relate to the location of the structure. For example if we look at Swift Creek, there we have a small embankment fill across the lower end of a creek channel. I would suggest to you that the concerns in that dump for longevity may be different from a dump which is built higher up in the drainage and, for example, covers the upper two thirds of the drainage basin. I think that the considerations may be different for that type of rock drain. So again, goalposts have to be clearly established and I would suggest that we don't look at things in terms of 10,000 years, a million years, I think we're getting beyond the realm of what we can possibly extrapolate. I think we have to look at more reasonable limits and I have my own thoughts on those limits but I'm sure others have different thoughts. I have some other things to offer on this Question 3 in relation to migration of fines. Not being an engineer I have to offer some practical considerations from observations I have made. I have been very involved in rock drains and engineering. I've learned a bit and I guess migration of fines I usually look around, being a biologist I guess I always look at nature and I have observed, and I think you've all observed some natural situations which can lead us to some pretty general conclusions about the migrations of fines. I don't know if many of you have seen gravel operations but I've seen quite a few and one of the things I've seen fairly often is fine silt layer deposited by the alluvial process, lying over a layer of coarse clean gravel and that has obviously been there for a long period of time and its been subject to precipitation which is really the major factor we are looking at here in rock drains and the possible vibration of fines down through this, well, into the rock drain, I believe, water is the vector that we are talking about. So there is a personal observation I have made and I think it offers some practical conclusions to that potential for migration of fines. Some other observations on the long term nature of rock drains. I don't think we can take credit for inventing rock drains. I believe that if anyone's taken a hard look around we'll see that they're natural phenomenon. They exist around us just that we really haven't looked closely at them. I personally know of several examples of water flowing through broken rock and undoubtedly those structures have been in place for thousands of years. Some of them from the glaciation which is a fair period past. There are three local examples that I know of that I've been to in the area, and some of them have significant flows moving through them. And so I guess my personal observation based on that is, that I suspect that they have some good potential for being long term structures and I would suggest that maybe we could look at some of those things or maybe there is some things we can learn from looking at some natural situations to give us an idea of time frame because none of the structures we've built have been around that long. That's all I've got to say.

DAVE CAMPBELL:

I feel that I've kind of made my pitch on the migration of fines and couldn't add anything better I think than what Dermot has just added. Regarding durability of rock, we heard a paper yesterday which I think the authors admit that tests to determine the durability of rock needs some more research. What we have now is not that satisfactory. We talk about the observational method that Charlie was talking about refers to what do you do, what are the contingencies and you monitor what's going on so that you know whether you're alright or whether you should go to the contingency that you have in mind. I think another observational method is what does the field have to say. Often these rocks that we're planning to incorporate within rock drains are cliff forming members and often you can find evidence of where the cliff may have failed not through some inadequacy in the strength of the rock but simply blocks have peeled off on joint surfaces and they've tumbled down and they may have been in place for a long time. Simply by going and looking at those blocks can tell you something quite significant about durability of the rock that you plan to use.

CHAIRMAN:

Thank you gentlemen. I might make one comment I think we should also look at some of those areas which have been recleimed and resloped. Some of them at Westar have been there for close to 15 years. Those resloped areas have been revegetated. There has been no erosion there, no problems with water percolating through, and I think that tells us something about the material which we are dealing with and we should probably be looking at those areas too.

MURRAY GALBRAITH:

One of the other speakors mentioned that in the event of the drain plugging and the water coming up and over the top of the dam it would still function adequately, but the thought of all that water coming over the top of the dam and messing up all that nice reclamation and working it's way through the dam and carrying the fines further down and downstream, is kind of distasteful to a reclamation inspector. I was thinking that with Charlie Ripley here, who has a great deal of experience with dams and spillways could he comment on the practice of fill being stopped short of the bank opposite the dump, creating a notch which would act as a spillway section?

CHARLIE RIPLEY:

No comment

PROF. LAWSON:

I'd made a very brief comment on passing, and that is that if in fact there is an accumulation of sediment on the upstream side, next time the sediment becomes exposed you'd dredge it out.

CHAIRMAN:

Thanks Jack. We have a few minutes and a few comments from the audience. What is your opinion on the question of long term permanency of rock drains in relation to migration of fines and the durability of rock used. Anyone want to make comments. Dave.

DAVE SELLARS:

Thank you Jake, my name is Dave Sellars from Klohn Leonoff, the question that I think really has to be addressed is how long is the design life of these structures? And if we are considering rock drain to be a permanent structure, and that is the opinion I understand of the Ministry of Environment, then really you're talking about an infinite design life. And, you know, as engineers we don't usually have much experience with designing structures with an infinite design life, but there has been some thought given to it particularly in the United States with abandonment plans for mines. And one of the considerations that's given is to the design flood. Now when you are looking at a design flood for a particular structure, you look at the probability of that flood occurring within the design life of that stucture. Now if your structure has a design life of say, 50 years, then you can probably get away with the 200-year flood as a reasonable flood that's only got a very low probability of occurrence during the design life. But if you have infinite design life then the probability of a probable maximum flood occurring is actually a certainty. So really, philosophically, if you are going to consider it a permanent structure, you're going to consider it has an infinite design life and you have to really at least look at what the consequences of a probable maximum flood occuring are. Not necessarily design the actual drain to convey the probable maximum flood but at least look at the consequences. Now just to carry on from that I think there is no question that there is going to be sediment accumulation at the inlet of any rock drain that is constructed. And this has been demonstrated at the Swift Creek rock drain and it is not just going to be bed load but it will also be suspended load because the velocities in the pond upstream of the drain are lower than the velocities in the drain. And I agree with Dave Campbell that only very fine material will actually pass into the drain. This creates two problems for a permanent structure one is there is a potential for sediment starvation downstream with consequent erosion. Now in most of the cases we've looked at on small catchments that's probably not going to be a problem but if we start building rock drains in larger catchments then I think that is something that has to be considered. But a more serious consideration relates to an accumulation of sediment in the long term. Now all the water engineers know that the ultimate fate of any reservoir is that it will ultimately fill completely with sediment and that the water supply dam is going to be abandoned, it's normally breached, and then nature is allowed to take its course. Now I know there is going to be accumulation of sediment in the very, very long term. That accumulation of sediment could be quite extreme and I think we should be looking at, for abandonment design, at what will happen if that valley fill is over topped. Now it could be that as Professor Lawson points out, that we can get away with the fact that it will just act like a flow-through rock spillway. But that's not necessarily true because near the top of the dump we're getting into the finer material because the coarse segregation occurs at the bottom of the dump. Now, this isn't necessarily going to be a problem for design but I think its just something that has to be looked at. Just one last thing I'd like to say is that these rock drains and valley fills do occur in nature as Dermot was pointing out. There's one that I'm aware of that is very, very large. It's in northwestern B.C. It's a massive slide about the size of the Hope Slide and it has very, very coarse material at the toe with large boulders up to about 5 metres in diamete. And it has formed a pond behind the slide which extends about 1-2 kilometres, it's really a lake. Now the flow out of this lake and the catchment area is about 20 square kilometres, so it's quite a considerable catchment area. In the winter time you get about 1-2 cubic metres/second of flow through the rock fill but then during spring runoff with higher flows, the pond rises and in fact the rock fill is overtopped because the capacity of the clow-through rock fill portion it cannot convey more than about 1-2 cubic metres/ second. In looking at this kind of phenomenon one wonders whether maybe that is the ultimate fate of a valley fill. Thank you.

CHAIRMAN:

Thanks very much Dave. It is now 4:45, I think Roger has a summary to do. I will make a short summary and say that we have covered off a great deal during these two days of technical sessions. We've asked and answered a number of questions, there are still questions that are unanswered but I think we can all go out of here feeling that we know a lot more about rock drains than we did when we came in. And I think we could ask a number of questions and look at a number of things which we did not maybe think about before and I think we've gathered good knowledge and we feel that, when you consider permanency you know nothing is really permanent when you look at the physiography of the earth. We can look at, for example, the tailings dam at Lornex which would be over 500 feet high at the final analysis. Is that permanent? Who's to say. It was designed with the best knowledge and technical expertise which we have available at this point in time. So I think we'll be looking at rock drains in years to come and saying "well we know a lot more about them but we can't say with certainty, that it'll be safe in perpetuity". Roger, would you care to comment and sum up here. You got some words of wisdom to say. Before I leave I'd like to thank all the panel members for participating in this panel and I appreciate their comments and the comments from the audience which didn't get much chance to comment.

ROGER BERDUSCO

Thanks very much Jake. I think Jake did an excellent job. I think anyone in the audience will recognize this was difficult for many reasons. We tried to condense several days worth of observations both out in the field and technical presentations. I think we probably tried to condense the experience of a truly international group of experts on a rather new technology and I think we have done it very well, considering the time that we had to do it in. I'd like to thank all of the people on the panel, particularly this afternoon, and the speakers earlier. Again I think the technical sessions have been absolutely first class throughout and also, the participation from the group has been excellent as well and that of course, makes things even better as we go along. We've heard many pros and cons on flow-through rock drains and as Jake mentioned I believe we've answered many more questions then we've left unanswered. We therefore, can consider ourselves to be successful, very much so. We also can consider ourselves successful in that we have left ourselves some job security, we've left a big job to be done, and that's not being highly unintelligent as well. We've heard some pessimism, we've heard some optimism and I think those of us that worked on the pulling together of the symposium designed a substantial amount of that into the system because we tried to give as much of a balance as we could. At the same time no one has been led but we recognize again that people are individuals with varying opinions. When I think of the pessimism and try to put it into perspective, someone here earlier today reminded me of something that is very relevent, very, very, relevent and I'm sure many of you in crowd can relate to this. I'm reminded of the pessimism not so many years ago in the late 60s that we heard about land reclamation. I remember very, very distinctly sitting and listening to so called experts, say that we couldn't grow vegetation on mine slopes. It couldn't be done over 6,000 feet. We couldn't do this, we couldn't do that. Tremendous pessimism and I think all of you that have anything to do with that particular field of expertise recognize the gains that we've made in that area and again try to put that into perspective and consider what happened there. I think we're at that stage with this particular technology as well. I'm reminded also that the definition of an expert, as I've heard it, is that X is a has been, and a spurt is a drip under pressure. So you have to keep that in mind as well sometimes. We must consider the concerns and problems that we have heard here today as challenges, not as excuses to grind things to a halt and stop, throw our hands up in the air. They are challenges. Everyone of us in this room has a very good reason for being here and really in fact flow-through rock drains really do represent a substantial challenge to us. We must proceed with caution, with expertise as it can be gained from many different sources, with research-research that is directed from a group such is collected here today. I can't think of any other way to put together a better group of people that are qualified to do what we have to do. They're right here, we don't have to go looking, they're right here. The people represented here are from industry, from government, the academic world and that really again, the cross section is what we have to have. So we have been handed the challenge and let's solve it collectively. We have many opportunities to do that. Let's do it right. We have the opportunity.

SYMPOSIUM PARTICIPANTS

Mr. Charles Ripley
Consulting Eng.
5011 Hilarie Place
Victoria, B.C.
(604) 658-8007

Mr. Len Skakum
Pit Gen. Foreman
Afton Operating Corp.
Box 937
Kamloops, B.C.
(604) 374-5022

Mr. P. Ziemkiewicz
Dir. Recl. Research
Alberta Energy
10909 Jasper Avenue
Edmonton, Alberta T5J 3M8
(403) 427-8042

Mr. Larry Brocke
Chairman
Alberta Environment
9820-106th St.
Edmonton, Alberta
(403) 422-2636

Mr. D. Beddome
Recl. Officer
Alberta Environment
P.O. Box 2924
Edson, Alberta T0E 0P0
(403) 723-8231

Mr. R.G. Killam
Amax of Canada Limited
1600-1066 W Hastings St
Vancouver, B.C. V6E 3X1
(604) 689-0451

Mr. Russ Crouse
Atlanta Gold Corporation
16-3001 S. Roosevelt
Boise, Idaho, USA 83750
(208) 384-9203

Mr. Greg McKillop
Mgr. Min. Dev.
B.C. Ministry of Energy Mines
1114 Labrador Place
Victoria, B.C. V8X 4G7
(604) 727-7641

Mr. C.O. Brawner
President
Brawner Engineering Ltd.
780 Greenwood Road
W Vancouver, B.C. V7S 1X7
(604) 922-3717

Mr. R.R. Brown
Environ. Supt.
Brenda Mines Ltd.
P.O. Box 420
Peachland, B.C. V0H 1X0
(604) 763-3220

Mr. J. Paterson
Bullmoose Operating Corp.
P.O. Box 500
Tumbler Ridge, B.C.
(604) 242-5221

Mr. Arquimides Perez
Mine Engineer
Byron Creek Collieries
Box 2625
Fernie, B.C. V0B 1M0
(604) 423-6657

Mr. Bill Kovach
Env. Technologist
Byron Creek Collieries
Box 1960
Sparwood, B.C. V0B 2G0
(604) 562-2837

Mr. Rob Nichols
Engineering Sup.
Byron Creek Collieries
Box 1960
Sparwood, B.C. V0B 2G0
(604) 562-2837

Mr. F.N. Agnew
Manager
Coal Association of Canada
301, 100-8th Ave SW
Calgary, Alberta T2P 3M7
(403) 262-1544

Mr. Jim Lant
Crows Nest Resources
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Malcolm Ross
Supervisor
Crows Nest Resources Limited
525-3rd Avenue SW
Calgary, Alberta T2P 3Y9
(403) 232-2509

Mr. Brent Densmore
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Gerry Reeves
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Ken Glinz
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Roger Williams
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Ted Hanna
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Don Townsend
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Mark West
Crows Nest Resources Ltd.
P.O. Box 2003
Sparwood, B.C. V0B 2G0
(604) 425-2555

Mr. Barry Lawley
Bio. Tech.
Dept. of Fisheries & Oceans
229 Pr. John Way
Nanaimo, B.C. V9T 4L4
(604) 756-7266

Mr. Ernest Mahadeo
Energy Res. Conservation Board
640 Fifth Ave SW
Calgary, Alberta T2P 3G4
(403) 297-8311

Mr. Peri Mehling
Environmental Eng.
Environment Canada
3rd Floor, Kapilano 100
W. Vancouver, B.C. V7T 1A2
(604) 666-2199

Mr. Mark W. Fisher
Mine Engineer
Equity Silver Mines Ltd.
Box 1450
Houston, B.C. V0J 1Z0
(604) 845-7799

Mr. R.D. McCosh
F.D. McCosh Resource Consult.
15 Templemont Dr. NW
Calgary, Alberta T1Y 4Z5
(413)

Mr. Deabi A. Hlady
Wildlife Tech.
Ministry of Environment & Parks
106-5th Avenue S
Cranbrook, B.C. V1C 2G2
(604) 426-1450

Mr. Roger Berdusco
Fording Coal Ltd.
P.O. Box 100
Elkford, B.C. V0B 1H0
(604) 865-2271

Mr. Ron Jones
Fording Coal Ltd.
P.O. Box 100
Elkford, B.C. V0B 1H0
(604) 865-2271

Mr. Don Guglielmin
Mine Engineer
Fording Coal Ltd.
P.O. Box 100
Elkford, B.C. V0B 1H0
(604) 865-2271

Mr. Robin Gold
Mine Engineer
Fording Coal Ltd.
P.O. Box 100
Elkford, B.C. V0B 1H0
(604) 865-2271

Mr. Doug Kennedy
Engineer, Planning
Fording Coal Ltd.
P.O. Box 100
Elkford, B.C. V0B 1H0
(604) 865-2271

Mr. Milos Stepanek
Princ. Consultant
Geo-Engineering Ltd.
8903-33rd Ave. NW
Calgary, Alberta
T3B 1M2
(403) 250-8850

Mr. D.B. Campbell
Principal
Golder Associates
224 W 8th Avenue
Vancouver, B.C.
(604) 879-9266

Mr. Alistair Kent
Senior Engineer
Golder Associates
224 W 8th Avenue
Vancouver, B.C.
(604) 879-9366

Mr. Gary B. Gould
Sen. Mine Engineer
Gregg River Resources Ltd.
Bag Service 5000
Hinton, Alberta T0E 1B0
(403) 692-3967

Mr. Marlin Murphy
Environmentalist
Gregg River Resources Ltd.
Bag Service 5000
Hinton, Alberta T0E 1B0
(403) 692-3967

Mr. David Sellars
Manager
Klohn Leonoff Ltd.
10180 Shellbridge Way
Richmond, B.C. V6X 2W7
(604) 273-0311

Mr. Andrew M. Whale
Insp. of Mines
M.E.M.P.R., Insp. Branch
P.O. Box 1290
Fernie, B.C. V0B 1M0
(604) 423-6884

Mr. E.J. Hall
Reclamation Inspector
M.E.M.P.R., Insp. Branch
Box 7438
Fort St. John, B.C. V1J 4M9
(604) 787-3450

Mr. J.W. Robinson
Insp. of Mines
M.E.M.P.R., Insp. Branch
2569A Kenworth Road
Nanaimo, B.C. V9T 4P7
(604) 755-2486

Mr. D.L. Flynn
Insp. of Mines
M.E.M.P.R., Insp. Branch
Bag 5000, 3793 Alfred Ave
Smithers, B.C. V0J 2N0
(604) 847-7386

Mr. Bryan H. Good
Mines Inspector
M.E.M.P.R., Insp. Branch
Box 5000, 3793 Alfred Ave.
Smithers, B.C. V0J 2N0
(604) 847-7390

Mr. Ralphn W. McGinn
Mgr. Insp. Services
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St.
Victoria, B.C. V8V 1T7
(604) 387-3781

Mr. Raymond Crook
Mgr. Project Eval.
M.E.M.P.R., Min. Policy Branch
Rm. 121, 525 Superior St
Victoria, B.C. V8V 1X4
(604) 387-3787

Mr. R.A. Fyles
Mgr. Inspc. Ser.
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St
Victoria, B.C. V8V 1X4
(604) 387-3781

Mr. V.E. Dawson
Chief Insp., Mines
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St
Victoria, B.C. V8V 1X4
(604) 387-3781

Mr. R.T. Martin
Sr. Geotech. Insp.
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St
Victoria, B.C. V8V 1X4
(604) 387-3781

Mr. D.M. Galbraith
Reclamation Insp.
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St
Victoria, B.C. V8V 1X4
(604) 387-3781

Mr. David Parsons
MOE Coord., MDRP
Min. of Environment
777 Broughton St
Victoria, B.C. V8V 1X5
(604) 387-9674

Mr. Terry Johnson
Safety Director
Mining Association of B.C.
860-1066 W Hastings St
Vancouver, B.C. V6E 3X1
(604) 681-4321

Mr. John E. Brenner
Roads Supervisor
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St
Victoria, B.C V8V 1X4
(604) 387-3781

Mr. Mark Strosher
P.Eng.
Ministry of Environment & Parks
1617 Baker St
Cranbrook, B.C. V1C 1B4
(604) 426-1475

Mr. Roland W. Grimm
Technician
Ministry of Environment & Parks
1617 Baker St
Cranbrook, B.C. V1C 1B4
(604) 426-1475

Mr. John H. Dyck
Reg. Water Manager
Ministry of Environment & Parks
310 Ward St
Nelson, B.C. V1L 5S4
(604) 354-6370

Mr. Dwain Boyer
Eng. Section Head
Ministry of Environment & Parks
310 Ward Street
Nelson, B.C. V1L 5S4
(604) 354-6372

Mr. Uwe Finerg
Head, Alloc. Sec.
Ministry of Environment & Parks
1011-4th Ave
Prince George, B.C. V2L 3H9
(604) 565-6435

Mr. Larry Garinger
Engineer
Ministry of Environment & Parks
Bag 5000, 3726 Alfred Ave
Smithers, B.C. V0J 2N0
(604) 847-7289

Mr. Norman Ringstand
Coor. Land Use
Ministry of Environment & Parks
777 Broughton St
Victoria, B.C. V8V 1X5
(604) 387-9673

Mr. Yenon Fellman
Hydraulic Engineer
Ministry of Environment & Parks
765 Broughton St
Victoria, B.C. V8V 1X5
(604) 387-9453

Mr. Brian Anderson
Noranda Mines Ltd.--Bell Mine
P.O. Box 1000
Granisle, B.C. V0J 1W0
(604) 697-2201

Mr. Alan Stewart
Vice President
Piteau Associates Eng. Ltd.
408-Kapilano 100
W. Vancouver, B.C. V7T 1A2
(604) 926-8551

Mr. F.B. Claridge
Director
Piteau Engineering Ltd.
206, 1615-10th Ave SW
Calgary, Alberta T3C 0J7
(403) 244-6415

Mr. Bo Elgby
Quintette Coal Ltd.
Box 1500
Tumbler Ridge, B.C. V0C 2W0
(604) 242-3221

Mr. J.H.H. Chamberlin
Quintette Coal Ltd.
Box 1500
Tumbler Ridge, B.C. V0C 2W0
(604) 242-3221

Mr. Pete Sheehan
Sr. Mine Engineer
Smokey River Coal Limited
P.O. Box 2000
Grande Cache, Alberta T0E 0Y0
(403) 827-3711

Mr. Dave Smith
Principal
Thurber Consultants Ltd.
200-1445 W Georgia St
Vancouver, B.C. V6G 2T3
(604) 684-4384

Mr. R.A. Welsh, Jr.
Geologist
U.S. Office of Surface Mining
10 Parkway Center
Pittsburgh, PA, USA 15220
(412) 937-2877

Mr. Dave Cline
Rec. Hydrologist
Utah Div. of Oil, Gas & Mining
3 Triad Center, Suite 350
Salt Lake City, Utah 84180
(801) 538-5340

Mr. Rick Summers
Rec. Hydrologist
Utah Div. of Oil, Gas & Mining
3 Triad Center, Suite 350
Salt Lake City, Utah 84180
(801) 538-5340

Mr. R.J. Hillis
Chief, Env. Con.
Utah Mines Ltd.
Box 370
Port Hardy, B.C. V0N 2P0
(604) 949-6326

Mr. W.A. Hogan
Productions Supt.
Utah Mines Ltd.
Box 370
Port Hardy, B.C. V0N 2P0
(604) 949-6326

Mr. Ken Reipas
Geotechnical Eng.
Westar Mining Ltd.
P.O. Box 2000
Sparwood, B.C. V0B 2G0
(604) 425-8134

Mr. J. Nuffield
Dir. Gen. Engin.
Westar Mining Ltd.
P.O. Box 2000
Sparwood, B.C. V0B 2G0
(604) 425-8300

Mr. Steve Little
Westar Mining Ltd.
P.O. Box 2000
Sparwood, B.C. V0B 2G0
(604) 425-8334

Mr. Don Parsons
Westar Mining Ltd.
P.O. Box 2000
Sparwood, B.C. V0B 2G0
(604) 425-8300

Mr. Pyara S. Lotay
Principal Engineer
Westar Mining Ltd.
P.O. Box 2000
Sparwood, B.C. V0B 2G0
(604) 425-8334

Mr. W.A. McLaren
President
Western Canada Hydraulic Lab.
1186 Pipeline Rd.
Port Coquitlam, B.C. V3B 4S1
(604) 464-7277

Mr. Brian Chappell
Bureau of Mineral Resources
13 Collingridge St
Weston, Canberra, Australia

Mr. Roberts Thompson
President
CTL Thompson Inc.
1971 West 12th Ave
Denver, CO, USA 80204

Mr. Dermot Lane
Reclamation Officer
Fording Coal Ltd.
P.O. Box 100
Elkford, B.C. V0B 1H0

Mr. Jack Lawson
Professor of CNIA
University of Melbourne
41393 Barkers Road
Melbourne, Australia 3101

Mr. J.D. Ventura
Civil Engineer
US Office of Surface Mining
Ten Parkway Center
Pittsburgh, PA, USA 15220

Mr. Louis Hamm
Mining Engineer
US Office of Surface Mining
1020-15th Street
Denver, CO, USA, 80202

Mr. Tom Waterland
President
Mining Association of B.C.
860-1066 W Hastings St
Vancouver, B.C. V6E 3X1

Mr. Richard Booth
Insp. of Mines
M.E.M.P.R., Insp. Branch
P.O. Box 1290
Fernie, B.C. V0B 1M0

Dr. John Errington
Sr. Reclamation Insp.
M.E.M.P.R., Insp. Branch
Rm. 105, 525 Superior St
Victoria, B.C. V8V 1X4

Dr. Dick Lewis
Inspector of Mines
M.E.M.P.R., Insp. Branch
1652 Quinn Street
Prince George, B.C. V2N 1X3

Mr. Art O'Bryan
Reclam. Inspector
M.E.M.P.R., Insp. Branch
310 Ward Street
Nelson, B.C. V1L 5S4

Mr. Gene Mickleson
Westar Mining Ltd.
P.O. Box 2000
Sparwood, B.C. V0B 2G0

Mr. John Dick
Ministry of Environment & Parks
777 Broughton St
Victoria, B.C. V8V 1X5

Mr. Jacques Gourdou
Techminemet Ingenierie
1, Avenue Albert Einstein
B.P. 106
78191 Trappes Cedex, France