Research Technical Completion Report
Pacific Northwest Regional Commission
Grant # 10890704

ASSESSMENT OF MONITORING TECHNOLOGY - APPLICATIONS TO EARTH DAM SAFETY

By

R. Hays
T. R. Howard
Geological Engineering

J. N. Peterson
G. Garrabrant
Electrical Engineering

Submitted to:
Idaho Department of Water Resources

Idaho Water & Energy Resources Research Institute
University of Idaho
Moscow, Idaho

January, 1981

Research Technical Completion Report
Pacific Northwest Regional Commission
Grant # 10890704

ASSESSMENT OF MONITORING TECHNOLOGY - APPLICATIONS TO EARTH DAM SAFETY

By

R. Hays
Geological Engineering

T. R. Howard
Geological Engineering

J. N. Peterson
Electrical Engineering

G. Garrabrant
Electrical Engineering

Submitted to:
Idaho Department of Water Resources

Idaho Water & Energy Resources Research Institute
University of Idaho
Moscow, Idaho

January, 1981

ACKNOWLEDGEMENTS

The authors wish to express their sincere thanks to Dave Hollingshead Office of Dam Safety, Idaho Department of Water Resources, for his assistance and guidance throughout the course of the project; Linda Fulton of the Idaho Water Resources Research Institute, University of Idaho, for her help in the acquisition of materials; and to the Pacific Northwest Regional Commission who funded the project through Grant No. 10890704.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF GRAPHS

LIST OF FIGURES

LIST OF FIGURES (cont'd)

ABSTRACT

The need for a national inspection program for dams had been recognized for many years. On August 8, 1972, the 92nd Congress passed Public Law 92-367 creating the National Dam Inspection Program. The act authorized the Secretary of the Army to undertake a national program of inspection of dams. The Corps of Engineers inspection program, along with State cooperation, represents significant progress toward reducing the risks of high hazard non-Federal dam failures.

In order to obtain well defined causes of earth dam failures, statistical analysis of past earth dam failures was made. A comprehensive list of earth dam failures was collected and listed after Middlebrooks (1953) and others. The results of this list were analized to determine the major failure mechanism(s) and to help establish an inspection tool for earth dams. The most common failure mechanisms appear to be seepage and embankment sliding.

A survey of existing technology was made in an effort to determine which type of equipment would best detect seepage and slope stability and still be within the following guidelines:

1. Inexpensive
2. Portable
3. Ease of Operation
4. Easily Interpreted and Accurate Data.

The selection of the method was done by a process of:

1. Discussion by the project members of each method and its potential;

2. Personal interviews were conducted with representatives of various equipment manufacturing companies;
3. Personal interviews were conducted with area consulting geophysicists;
4. A final discussion of the methods, costs, and results of the interviews was conducted and a method was selected.

The method selected was the Acoustic Emission Monitoring System.

Acoustic emission is a passive method similar to the geophysical methods. A metal bar is inserted into suspect areas (areas of known or suspected deformation). An accelerometer is attached to the metal bar or waveguide. The accelerometer converts the deformation, "noise", brought to the surface by the waveguide into an electrical impulse. The impulse is then filtered to eliminate noise, and amplified. If the impulse is great enough to cross a preset threshold, (manual or automatic), a "count" is registered on a digital display. The higher the number of counts displayed in a given time, greater deformation is occurring.

In order to become familiar with the acoustic emission monitoring system, field work was initiated. The purpose of the field work was to:

1. Become familiar with setting up the equipment,
2. Become familiar with the different equipment settings to insure proper technique, and
3. Become familiar with data interpolation.

Field work was done on four dams in southern Idaho with known problems.

The acoustic emission monitoring system appears to meet the guidelines set down above. The system appears to be an excellent tool for monitoring slope stability. Acoustic emission appears as a promising tool for the detection of seepage and more work is being done in this area as a continuation of this project.

CHAPTER I

INTRODUCTION

Purpose and Objectives

The purpose of this study is to ascertain the most common mechanisms of earth dam failure and to evaluate presently available and non-available (research stage) technology that may be incorporated into a state dam inspection program to help insure the safety of the structure and life and property below.

This purpose may be expressed in terms of these general objectives:

1. Ascertain most probable mechanisms for dam failure by the compilation of case histories of dam failures and statistical analysis of the data collected;
2. Determine available and presently non-available technology suitable for detecting failure mechanisms;
3. Evaluate technology for its primary use, advantages, disadvantages, availability, and cost; and
4. Select the method(s) that would be most suitable to detect failure mechanisms and if possible do more detailed research in an attempt to find its capabilities and limitations.

Scope of Work

To ascertain the most probable mechanisms for dam failure, a two stage approach is taken. First, a detailed compilation of worldwide dam failures was undertaken. The method of ascertaining probable failure mechanisms in dams is confined to failures that are reported in journals, professional magazines, newspapers and other published sources. This covers most but not all failures since many small failures in each state never become known except to those individuals involved directly. Second, when the list of dam failures is complete, a statistical analysis of the results to define the most probable failure mechanisms is to be run.

A search was conducted to identify methods for detecting failure mechanisms. Letters were written to each of the fifty states, Puerto Rico, and the Virgin Islands to find if any other states are involved in a project of this nature or if any state has adopted already any equipment as an aid to their dam inspection program. A literature search for descriptions of equipment and methods was conducted and included reviewing professional journals, magazines and newspapers.

Research into devices suitable for detecting failure mechanisms is limited to equipment that is inexpensive, portable, with data that are interpreted easily, and are available or appear to be available in the near future. The research is also limited to equipment that could have a direct application to detection of failure mechanisms. No equipment that requires highly sophisticated techniques or is currently being developed will be considered but may be mentioned.

History of National Dam Safety Inspection Program

On August 8, 1972, the 92nd Congress, H. R. 15951 passed Public Law 92-367 creating the National dam inspection program. The act authorized the Secretary of the Army to undertake a national program of inspection of dams.

The Act defines "dam" as any artificial barrier, including appurtenant works, which impounds or diverts water and which is twenty-five feet or more in height or has an impounding capacity at maximum water storage elevation of fifty acre-feet or more or a minimum of six feet high and contain fifty acre-feet or more of storage.

The National Program for the inspection of dams is to be carried out by the Secretary of the Army, acting through the Chief of Engineers for the purpose of protecting human life and property. Exempted from the program are:

1. Bureau of Reclamation dams,
2. Tennessee Valley Authority dams,
3. International Boundary and Water Commission dams,
4. Dams licensed under the Federal Power Act,
5. Dams inspected by a state agency which the governor requests be excluded, and
6. Dams which do not pose any threat to human life or property.

As soon as possible after the inspection of any dam, the result of the inspection shall be forwarded to the Governor of the state in which the dam is located. Any hazardous findings will be reported immediately. The secretary shall provide advice on remedial measures necessary to mitigate or obviate any hazardous conditions found during inspection.

The Secretary shall report to Congress the activities under the Act, which report shall include, but not be limited to:

1. An inventory of all dams located in the United States;
2. A review of each inspection made, the recommendations furnished to the Governor of the State and information as to the implementation of such recommendation; and
3. Recommendations for a comprehensive national program for the inspection of dams and the responsibilities which should be assumed by Federal, State and local governments and by public and private interests (Public Law 92-367).

Action Taken After Public Law 92-367

Corps officials stated that they had intended originally to inspect dams, beginning with a representative sample. In December, 1972, the Undersecretary of the Army requested five million dollars to initiate a nationwide program for dam inspection. The funding proposal was rejected by the Office of Management and Budget and no appropriation request was made thereafter to the Congress to carry out Public Law 92-367 except for collecting inventory data and preparing recommendations for a national dam safety program.

In January, 1973, OMB issued a policy statement directing the Corps to perform an inventory of dams and make recommendations for a comprehensive national program to inspect and regulate dams for safety purposes. The OMB policy statement stated further that:

Inspections, to the extent they were made, were to be accomplished by the concerned States as part of their normal responsibilities.

The Corps was to develop inspection guidelines to be included in the national program.

The Department of the Army was to provide advice to the respective State Governors, upon request, for correcting or eliminating any hazardous conditions found in their States.

Earlier, in Feburary, 1973, the Corps advised the Senate and House Subcommittees on Public Works, Committee on Appropriations, that it did not intend to implement that section of the law which pertained to actual inspection of non-Federal dams. No appropriations requests were made to the Congress for such inspections.

On July 24, 1974, the Acting Secretary of the Army, by letter, advised the Congress of the Corps of Engineers progress in fulfilling the requirements prescribed by Public Law 92-367. The Acting Secreatry stated the Corps was (1) compiling an inventory of all dams in the Nation, (2) surveying Federal and State dam safety inspection programs, (3) developing guidelines for dam inspections, and (4) formulating recommendations for a national program of dam inspection and safety.

The acting Secretary of the Army also informed the Congress that while the authorizing legislation provided for the inspection of non-Federal dams, no inspections had been made and none were planned. The Acting Secretary said it was believed that the states should perform the inspection as part of their normal responsibilities.

In June, 1975, the Chief of Engineers submitted to the Secretary of the Army a five-volume document containing the report "National

Program of Inspection of Dams." This five-volume compilation included an inventory of dams, recommendations for a national program of dam safety, responses by State and Federal agencies to a aquestionnaire on dam supervision, a model law for state supervision of dams, and recommended guidelines for safety inspection of dams. Draft legislation for a dam safety program was submitted by the Chief of Engineers to the Assistant Secretary of the Army (Civil Functions) in December, 1975.

In March, 1976, the Secretary of the Army submitted to the Office of Management and Budget (OMB) the five volumes and draft legislation and recommended that the Corps' proposed national dam safety program be implemented. On April 2, 1976, four of the volumes containing the inventory of dams were released to the Congress, Federal agencies and states. On November 16, 1976, the draft legislation and the fifth volume, which contained the Corps' recommendations for a national dam safety program, were released to the Congress.

Why the National Dam Safety Program Had a Slow Beginning in Review of Corps of Engineers National Dam Safety Program

In 1976 the Office of the Comptroller General reviewed the Corps of Engineers procedures and guidelines, records, and reports applicable to implementation of Public Law 92-367. The Office of Comptroller General then interviewed Federal and State officials; and reviewed State legislation concerning dam safety. Also reviewed were various States' methods of collecting inventory data and obtained State officials' views on the Corps' proposal for a national program of dam safety and the costs of initiating the program.

The Office of the Comptroller General found that the national dam inventory developed by the Corps was both incomplete and based on data collected using inadequate definitions and procedures. Also most of the data were not verified.

The Office of Comptroller General stated that since the Corps had not determined the inventory's accuracy, more assurance should be obtained as to its accuracy before the Congress decides on a national dam safety program based on the information contained in the inventory.

The Office of the Comptroller General stated that the Corps of Engineers report and recommendations for a national dam safety program as released to Congress on November 16, 1976, was inadequate because it:

> Placed primary emphasis on voluntary participation by the States for non-Federal dams without presenting adequate information to the Congress as to the cost to the States or as to how the States could carry out the program without Federal assistance.
>
> Did not require minimum inspection criteria.
>
> Contained unreliable inventory and cost data.
>
> Recommended primarily an inspection rather than a safety program and did not present information to the Congress on safety matters, such as public information programs, possible revisions in zoning laws, and emergency warning systems.
>
> Did not provide the Congress with alternatives for carrying out a dam safety program. (Comptroller General, 1977)

Carter Administration Review of Dam Problems

Since the failure of Teton Dam in 1976, the Carter administration has attempted to improve both Federal and non-Federal dam safety for about 50,000 dams in the United States.

The Carter administration's effort has been directed along three fronts. These include a review of Federal dam safety procedures, a commitment to private dam safety and institutional reorganization.

Federal Dam Safety Procedures Review

The heads of each Federal department or agency responsible for any aspect of dam safety were asked to undertake a thorough review and evaluation of their own practices which could affect the safety and integrity of their dams.

The chairman of the Federal Coordinating Council for Science, Engineering, and Technology (FCCSET) convened an ad hoc Interagency Committee to analyze the Federal agencies' practices and procedures and to provide recommendations for improving Federal dam safety. As a result of this, on November 15, 1977, the report, Improving Federal Dam Safety was published by the FCCSET.

The director of the Office of Science and Technology Policy (OSTP) was established to organize an independent panel of experts for reviewing both agency practices and the Federal dam safety guidelines proposed by FCCSET. As a result of this, the OSTP Independent Review Panel published its report, Federal Dam Safety, on December 6, 1978.

In the first session of the 96th Congress on February 22, 1979, House of Representatives Bill 2354 and Senate Bill 504 were passed. Both amended Public Law 92-367. H. R. 2354 authorized the Secretary of the Army to restore certain hazardous dams to a safe condition. Senate 540 made provision of Federal assistance to the states for the development and implementation of effective dam safety programs in order to protect human life and property.

On June 25, 1979, the document Federal Guidelines for Dam Safety was published by OSTP and sent to President Carter with a cover letter from the Director of OSTP reporting the completion of the requested Federal dam safety review activities and summarizing the nature and intent of the guidelines.

Recommendations for Improving Federal Dam Safety

Recommendations which emerged from intensive review efforts included the following:

1. Implementation of the Proposed Federal Guidelines for Dam Safety. The guidelines apply to management practices and procedures for dam safety of all Federal agencies responsible for planning, design, construction, operation and maintenance, emergency preparedness, or regulation of dams.

   The guidelines also provided direction for management of technical activities in (1) site investigation and design; (2) construction, including quality assurance; and (3) operation and maintenance, including periodic inspection and emergency action planning.

2. Establish Federal Agency or Department Dam Safety Offices. Each Federal Department or Agency having responsibility for any aspect of dam safety should have an independent dam safety office or officer reporting directly to the head of the Department or Agency.

3. Coordinate Federal Dam Safety.

   A. General Interagency Needs. The interagency dam safety coordination established initially under the FCCSET *ad hoc* Interagency Committee and guided by the Office of Science and Technology during the Federal review process should continue.

B. Establishement of a Federal Dam Safety Coordinating Office. Previous to and during the process of this report, no single Federal agency has been charged with a continuing responsibility to plan and coordinate the dam safety programs carried out by a number of agencies.

4. Establish Formalized Programs of Periodic Inspection. It was recommended that each Department or Agency set up a formalized program of periodic inspection and re-evaluation of all dams against modern criteria in order to reduce the potential risk and consequences of Federal dam failures.

5. Initiate Efforts to Define Federal Role in Non-Federal Dam Safety. Departments and agencies should begin to resolve the safety-related ambiguities, associated with already-constructed and future private dams, which result from Federal actions (financial and technical).

Non-Federal Dam Safety Activities

The Corps of Engineers' inspection program represents significant progress toward reducing the risks of high hazard non-Federal dam failures. Although the States are cooperating in the inspection program and becoming more accountable, many complex issues remain which need to be addressed rationally and systematically and solved ultimately as the inspection program moves forward (Tschantz, 1979).

CHAPTER II

STATISTICAL ANALYSIS OF EARTH DAM FAILURES

Purpose and Objectives

The purpose of the statistical analysis of earth dam failures is to obtain well defined causes for such failures. In order to accomplish this purpose, a list of earth dams which failed was made. (See Appendix 2 for a complete list.) The list is modeled after and includes Middlebrooks (1953) analysis and the U.S.C.O.L.D. dam failures (Lessons, 1975). These data are to determine the major failure mechanism(s) and to help establish an inspection tool for earth dams.

An objective of the statistical analysis is to describe the age distribution for earth dams given various types of failures. If an age distribution for earth dams can be determined then it will be possible to predict the probability of failure of each earth dam after T years for a given type failure condition.

Major Causes of Failure

The cause of failure for each dam is classified into one of the following twelve general categories.

A - Piping	G - Poor construction
B - Overtopping	H - Blow out
C - Seepage	I - Breach
D - Sliding	J - Slope protection
E - Aperature works	K - Cracking
F - Settlement	L - Earthquake

In an effort to determine which types of failure mechanisms predominate, the data are sorted into percentages of total number of dams in each category divided by the total number of dams (i.e., % = (dams in category/total dams) x 100). Table II-1 summarizes the results. Table II-1 indicates that there are five predominate types of failure. These five types of failures occur so much more frequently than the other failure types, that most attention will be directed to them. The five most predominate types of failure are:

1. Piping - The movement of soil particles occurring when the hydraulic gradient of seepage approaches the critical hydraulic gradient. May occur at anywhere seepage occurs including foundation abutments, or along conduits.
2. Overtopping - Occurs when the spillway is blocked or inadequate or the basin capacity is exceeded by unusually heavy rain, melting snow, etc.
3. Seepage - Movement of water through a structure with or without soil particle movement. May or may not be a prelude to piping.
4. Sliding - Movement of embankment material along a plane of failure.
5. Aperature works failure - Improper function of aperature works may cause overtopping, seepage, piping, or sliding.

Two of these failure mechanisms are somewhat independent of the age of the dam. For instance, spillway failure due to obstruction can happen at any time of the life of a dam, as with overtopping. In other words aperature works and overtopping failures are independent of the

Table II-1

Percent of Total Failures in Each Category and in Each Sub-Category *

Category	Sub-category	% Failures In Category	Category	Sub-category	% Failures In Category
piping		15.7	seepage		16.4
	A-1	21.4		C-1	6.8
	A-2	31.4		C-2	4.1
	A-3	27.1		C-3	34.2
	A-4	1.4		C-4	5.5
	A-5	2.9		C-5	30.1
	A-6	1.4		C-6	0.0
	Misc.	14.3		Misc.	19.2
overtopping		17.8	sliding		15.3
	B-1	29.1		D-1	25.0
	B-2	13.9		D-2	25.0
	B-3	13.9		D-3	10.3
	B-4	0.0		D-4	1.5
	B-5	0.0		D-5	2.9
	B-6	0.0		D-6	2.9
	Misc.	43.0		Misc.	32.4
aperature		15.5	poor		2.2
works	E-1	10.1	construction	G-1	0.0
	E-2	46.4		G-2	0.0
	E-3	26.1		G-3	0.0
	E-4	4.3		G-4	0.0
	E-5	0.0		G-5	0.0
	E-6	0.0		G-6	0.0
	Misc.	13.0		Misc.	100.0
settlement		1.6	blow-out		1.6
	F-1	0.0		H-1	100.0
	F-2	0.0		H-2	0.0
	F-3	0.0		H-3	0.0
	F-4	14.3		H-4	0.0
	F-5	14.3		H-5	0.0
	F-6	0.0		H-6	0.0
	Misc.	71.4		Misc.	0.0
breach		.4	cracking		4.3
	I-1	0.0		K-1	10.5
	I-2	0.0		K-2	0.0
	I-3	0.0		K-3	0.0
	I-4	0.0		K-4	5.3
	I-5	0.0		K-5	5.3
	I-6	0.0		K-6	0.0
	Misc.	100.0		Misc.	78.9
slope		6.3	earthquakes		1.6
protection	J-1	7.1		L-1	57.1
	J-2	17.9		L-2	0.0
	J-3	7.1		L-3	42.9
	J-4	0.0		L-4	0.0
	J-5	0.0		L-5	0.0
	J-6	0.0		L-6	0.0
	Misc.	67.9		Misc.	0.0

* for explanation of sub categories see Appendix III.

internal structure of the dam. For this reason aperature works and overtopping failures will be ignored in this study.

Comparison with Middlebrooks Study

Middlebrooks (1953), as forementioned, collected data on earth dam failures. He also did some limited statistical analysis. Middlebrooks constructed two tables, one showed calendar year versus the number of failures in various categories of failure; the second was a table of the number of years after completion versus the number of failures in various categories of failure. Using Middlebrooks format, tables II-2 and II-3 show the results of the tabulation of data from this study. There is a slight variation in tables II-2 and II-3 from those of Middlebrooks, in that seepage and piping have been combined. The results shown are evident. Table II-3 should be looked at with some caution. The increments on the number of years after completion vary and have a tendency to allow more data in the same interval of later years than in earlier years (i.e., the first interval is $[0,1)$ while the third is $[5,10)$ and the fifth interval is $[20,30)$).

Statistics

In order to try to fit a distribution to the set of data points, it is necessary to look at a frequency plot of the number of dams that fail in a given category after T years old, only the increments will be constant and the smallest possible. In this case the smallest time increments are T = 1 year since the date of failure is only known up to the year. The frequency plots which follow show the number of dams that failed

Table II-2

Calendar Year Vs. Number of Dams Failed in Various Categories

Calendar Year	Piping	Overtopping	Seepage	Sliding	Aperature Works	Misc.
1850-1860	0	0	0	0	0	0
1860-1870	2	0	0	0	0	0
1870-1880	2	0	1	0	1	1
1880-1890	4	2	3	0	2	1
1890-1900	10	7	4	1	3	3
1900-1910	10	18	4	6	1	3
1910-1920	11	13	8	6	4	7
1920-1930	11	11	5	11	7	10
1930-1940	3	7	6	16	4	7
1940-1950	4	5	4	4	3	8
1950-1960	2	3	5	4	7	10
1960-1970	10	7	24	14	26	25
1970	1	6	9	6	11	11

Table II-3

Number of Years After Completion Vs. Number
Dams Failed in Various Categories

No. of Years After Completion	Piping	Overtopping	Seepage	Sliding	Aperature Works	Misc.
0,1)	14	4	23	12	6	17
1,5)	28	12	16	14	15	22
5,10)	1	1	8	5	7	8
10,20)	6	7	1	8	10	9
20,30)	0	1	7	5	5	2
30,40)	0	4	4	2	4	1
40,50)	0	2	1	1	4	2
50,100)	1	3	3	5	10	4

versus the number of years old before failure. It should be noted that these frequency plots are unconditional frequency plots in the sense that there may be many more dams which are 1 year old than dams that are 56 years old. In this case there is opportunity for more failures in 1 year after construction rather than in 56 years after construction. What is needed is a conditional probability function.

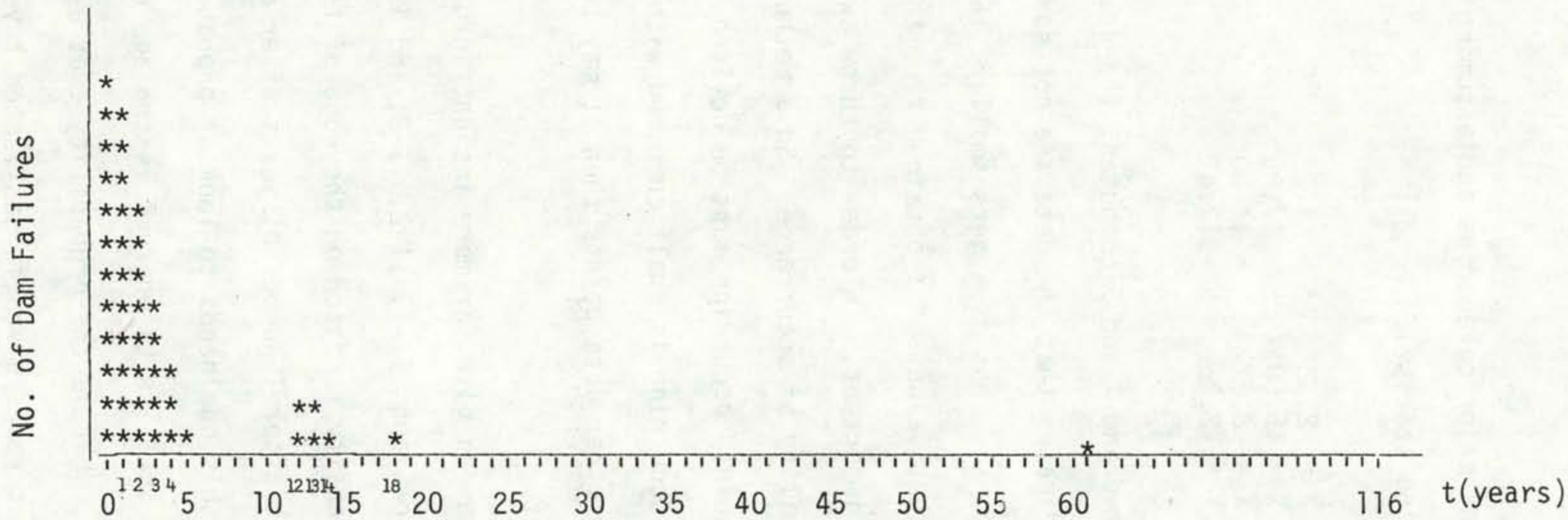
One of the questions of prime importance is to determine how long it would be necessary to monitor earth dams for a certain type of failure. From Graph II-1, if the average $\overline{X} = \sum_{1}^{n} f_i X_i$ (where f_i = number of occurrences of X_i) is taken the value of $\overline{X}$ is found to be 6.56 years. Yet if the two failures at 56 years and 116 years are thrown out (outliers) then $\overline{X} = 2.98$ years with $S^2 = 18.44$. Notice that the variance (S^2) is high indicating a spread of the data. The variance is a measure of the dispersion of the data. The variance is analogous to the moment of inertia of a mass. This information could be used, for instance, to limit the inspection of earth dams for piping failures to 3 years. The mean and standard deviation for each type of failure is listed below. The standard deviation, S, is just S^2 or var. These means and standard deviations include all data points, no outliers have been thrown out.

Piping	$\overline{X} = 6.34$	$S = 17.97$
Overtopping	$\overline{X} = 15.68$	$S = 17.85$
Seepage	$\overline{X} = 10.38$	$S = 16.69$
Sliding	$\overline{X} = 15.75$	$S = 23.49$
Aperature works	$\overline{X} = 20.57$	$S = 21.63$
*Settlement	$\overline{X} = 0.00$	$S = 0.00$ (no data)
*Poor construction	$\overline{X} = 5.43$	$S = 5.21$
*Blow out	$\overline{X} = 4.25$	$S = 5.07$
*Breach	$\overline{X} = 15.50$	$S = 13.50$
Slope protection	$\overline{X} = 6.13$	$S = 12.08$
Cracking	$\overline{X} = 7.35$	$S = 15.35$
Earthquake	$\overline{X} = 35.20$	$S = 15.05$

*Not enough data to draw valid conclusions about $\overline{X}$ and S.

Graph II-1

UNCONDITIONAL FREQUENCY PLOT OF FAILURE DUE TO PIPING.



Number of Years to Failure

After considering only the four major categories and assuming that outliers may be thrown out, the following statistics result:

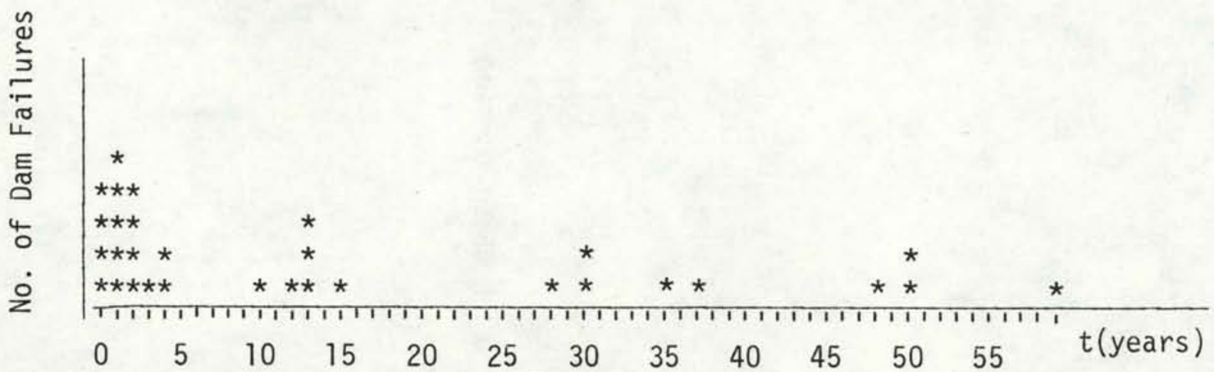
Piping	$\overline{X} = 2.98$	$S = 4.29$
Overtopping	$\overline{X} = 15.68$	$S = 17.85$
Seepage	$\overline{X} = 8.92$	$S = 14.00$
Sliding	$\overline{X} = 12.86$	$S = 17.47$

When outliers are ignored and $\overline{X}$ and S computed, the standard deviation, S, is smaller. This indicates that the data are not spread out very much and are close to the mean, $\overline{X}$. Thus as S gets smaller, the mean, $\overline{X}$, becomes more significant, so that values of $\overline{X}$ obtained from throwing away outliers start to become more important. In order to throw away outliers, one needs to know their probability of occurrence. So a method to determine the probability of data points occurring must be devised. If the probability of the data point occurring is small compared with all the other probabilities of all the other data points then it may be thrown away, otherwise it must be kept.

This type of information can give far more insight into the time of failure of each dam than can Graphs II-1, II-2, II-3, and II-4. What is required is a conditional density function on the edge of the dams so that the number of dams left after T number of years after construction is taken into account. To do this one needs to look at probabilities of the following type; $P(T > t + \Delta t \mid T > t)$ where T is the age of the dam. This is answering the question, What is the probability that a dam of age T will last more than $t + \Delta t$ years given it has lasted t years. This type of probability density function can be obtained from empirical estimates. In order to determine empirical estimates of these types of probabilities it is necessary to have the age distribution of the dams.

Graph II-2.

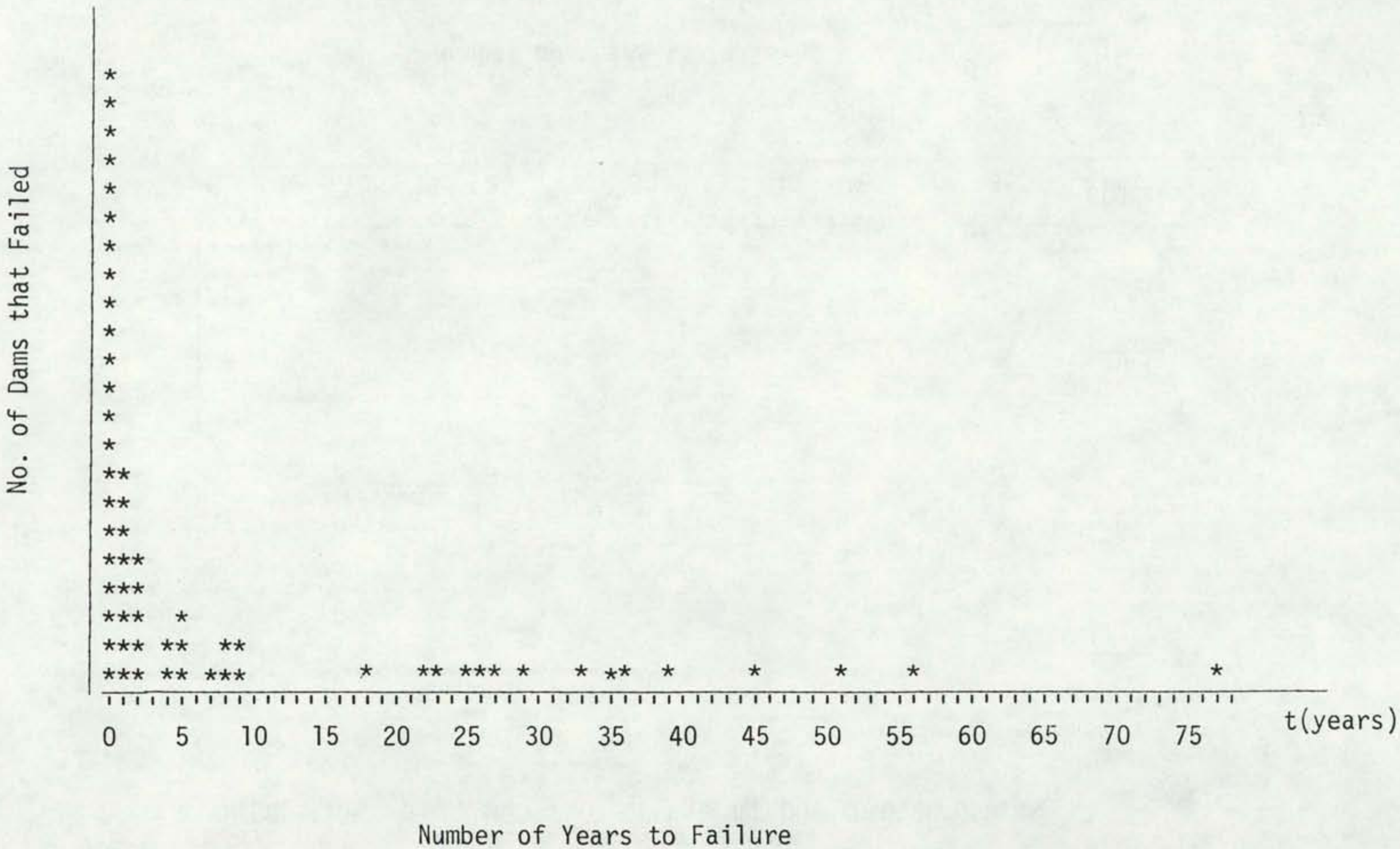
UNCONDITIONAL FREQUENCY PLOT OF FAILURE DUE TO OVERTOPPING



Number od Years to Failure

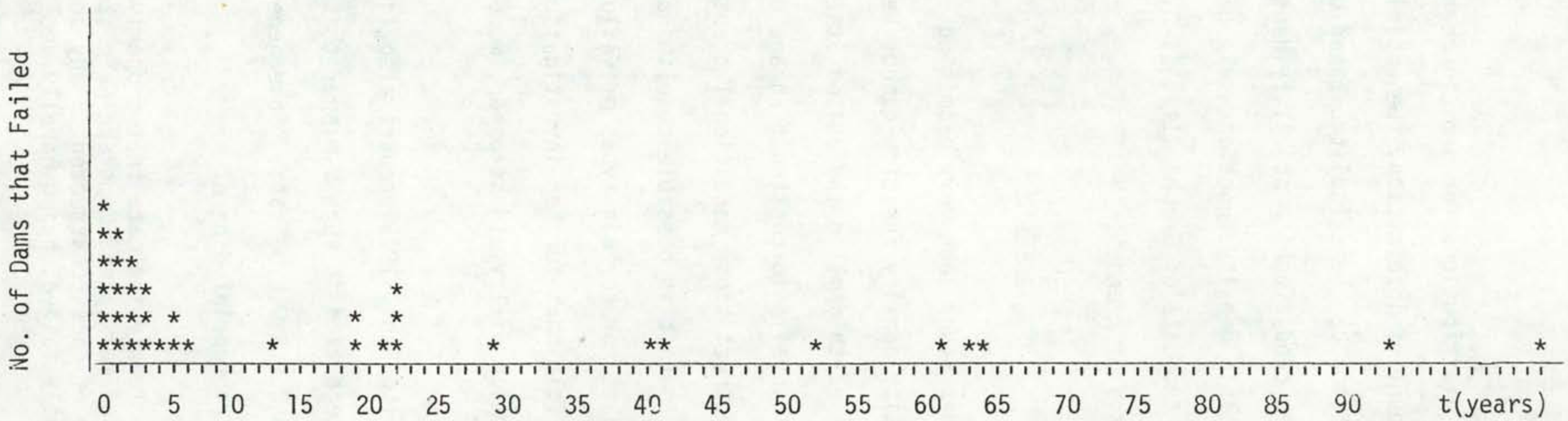
Graph II-3.

UNCONDITIONAL FREQUENCY PLOT OF FAILURE DUE TO SEEPAGE



Graph II-4.

UNCONDITIONAL FREQUENCY PLOT OF FAILURE DUE TO SLIDING



Number of Years to Failure

After obtaining the age distributions for dams in certain states which would yield the most amount of information, the empirical conditional probability density function can be calculated based on the data that have been collected. This conditional probability density function would be the desired result. This density function would give knowledge that could be used to weight the data of Graphs II-1, II-2, II-3, and II-4 and allow better estimates of $\overline{X}$ and S.

Summary

The results of this study look very promising. The completion of the conditional probability density function cannot be completed until a data tape containing all earth dams in the United States is obtained. Without these data it is invalid to construct an age distribution for earth dams and hence construct those conditional probabilities. One aspect of this study is that it is possible to eliminate the frequency of inspection of earth dams for certain types of failures after a given number of years. It is felt that further investigation into the conditional probability density function will yield stronger, more usable results than presented here.

The U.S.C.O.L.D. (Lessons, 1975) report also included statistics. The dam failures are categorized in such a manner that data in this report could not correlate with U.S.C.O.L.D. data. Some general conclusions from the U.S.C.O.L.D. study are reprinted here:

> The data clearly indicate that properly designed, constructed, and maintained dams are safe structures. The percentage of dams constructed in any decade, which have subsequently failed, has generally declined decade

by decade; (this agrees with the results of this study), the greatest decline in this percentage was made after 1930 and has remained below 0.3% for each decade since 1930. However, the reported failures have occurred anywhere from initial filling phase to after more than 100 years of satisfactory service. Failures caused by overtopping have been practically eliminated in dams completed since 1930, however most of these modern dams have not experienced their maximum reservoir elevations. (Lessons, 1975)

General Conclusions

Upon inspection of Graphs II-1 through II-4, a general trend appears. Failure due to piping along foundations, abutments and conduits, seepage through foundations, abutments and interior embankment and sliding upstream slope or downstream slope appear to be the predominant modes of failure in earth dams.

These results then provide a clear definition of what failure mechanism a chosen piece of equipment must detect. In summary the equipment chosen for this project must be able to measure seepage and slope instability with some warning time.

CHAPTER III

EQUIPMENT SURVEY: METHODS FOR DETECTION OF FAILURE MECHANISMS

Introduction

The results of the previous chapter indicate that there are two major structural failure mechanisms for earth dams, piping and slope stability. A survey of existing technology was made in an effort to determine which type of equipment would best detect both types of failure and still be within the following guidelines:

1. Inexpensive,
2. Portable (light weight and easily handled),
3. Ease of operation (equipment that does not require a technician), and
4. Easily interpreted data (results that anyone could interpret without extensive background knowledge).

This section and Appendix I are the results of this equipment survey. This section deals with some general types of equipment and gives general background information on each type. Appendix I lists the method, primary use, advantages, disadvantages, stage of development, and approximate cost, (these prices are 1978-79 and are minimum prices).

Flow Measurements

Water Balance

The presence of leakage through a dam or reservoir foundation is often detected by a water balance. A water balance compares the inflow and outflow of water to the reservoir, after allowing for evaporation and other losses, and can be evaluated as follows:

$$Z = P + S - K - V - O$$

in which Z = loss of water from the reservoir; P = the inflow of surface and ground water into the reservoir; S = the rainfall on the reservoir area; K = the change in water contents of the reservoir; V = the evaporation from the reservoir area; and O = the outflow of water from the reservoir.

Under good conditions an error of measurement of about 3% in water inflow, P, can be anticipated. With a large reservoir area, an error in evaporation measurement, V, of about ± 5 liters/sec/km^2 to 10 liters/sec/km^2 can be anticipated. Total seepage through the dam and foundation is measured at a suitable point in the run-off channel, several hundred meters from the dam, with outlets closed. Also, one needs to know the natural, ground-water inflows in the respective section and must estimate with sufficient accuracy the water flows through the top layers of the valley bottom along the channel.

Some authors believe a water balance to be extremely rough and inaccurate and only capable of determining very large leaks exceeding 10% of the whole reservoirs' water discharge. Also, results from the balance do not locate the actual site of the leakage.

<u>Water Velocity</u>

Near leakage sites in a reservoir, water velocities are high due to concentrated flow. As a result, sensitive current meters have been used in an attempt to locate leakage sites.

Other current meters which include microrotators, ultrasonic, and electrothermal devices have also been used successfully to measure flow velocities. Thermal meters make use of the decrease in temperature of a heated body (i.e., a resistor) with increasing water velocity circulating around it. As the temperature of the resistor decreases its resistance changes. By using graphs of resistance as a function of temperature, the resistor's temperature in the moving water, near the leakage site, and in still water is proportional directly to the water velocity.

Water movement caused by leakage is detectable only when flow rates exceed by two or three times the flow rates due to convection currents caused by temperature differences, local currents, wind influence, and other factors in the water. In general, the lower limit of detectable water velocity is 3 mm/sec. to 5 mm/sec. Any measuring device therefore should be sensitive in the low velocity range and also have a range from one to two hundred millimeters per second.

Benthonic Water Velocity

Rather than exploring for leakage sites in a reservoir by measuring the increase in clear water velocity, the velocity of the denser layer of water containing sediment on the bottom of the reservoir (the benthonic layer) can be measured. Benthonic refers to the bottom of a body of standing water. Besides giving the location of the infiltration sites, these velocity measurements also give the relative flow intensities.

Tracers

A widely used method of detecting seepage paths utilizes some form of tracer. A tracer may be injected in the water at the point of seepage origin and sampling the water at suspected areas of emergence can verify a seepage path. There are many types of tracer materials each yielding good results under certain conditions. Some classes of materials that have been used include dyes, chemicals, suspended particles, dissolved gases, bacteria, radioactive isotopes and even some naturally occurring parameter unique to the reservoir water.

Dye and Other Nonradioactive Materials

The movement of water through soil often may be traced by the use of dyes. Although dyes do not move as rapidly as water through soil, they are particularly suited for obtaining flow paths. Two general classes of non-radioactive dyes are fluorescent and nonfluorescent. Fluorescent dyes have been preferred for measuring water movement between wells and in canals because they are detectable at low concentrations. One of the major disadvantages of fluorescent dyes is that acid soils cause most of them to fade. Adsorption or absorption is a problem, particularly in clay soils. Many nonfluorescent dyes do not suffer from this limitation.

Nonfluorescent dyes are available in various forms, e.g. direct, disperse, acid, and basic. Of these, basic dyes are able to satisfy cation exchange sites of the soil and are likely to be poor water tracers. An experimental evaluation of the suitability of the different types of nonfluorescent dyes found that the direct form was absorbed rather strongly. The most suitable dyes and thus the least absorbed by soil, were found to be acid and disperse dyes.

Radioactive Tracers

As mentioned previously, another method of tracing seepage flow involves the use of radioactive isotopes. Basically, the disintegration of unstable isotopes in order to attain a stable nuclear configuration is a radioactive process during which nuclear particles or photons of energy, or both, are emitted. The process of radioactive disintegration or decay is spontaneous and cannot be influenced by external factors. Radioactives emitted include alpha, beta, and gamma particles. The utilization of this technique depends upon the detection of the radiations that radioactive isotopes emit.

Radiotracers will produce ionization in gases. This means that the number of positively charged ions and electrons formed in a given volume of gas is related to the amount of relevant radiation.

The most popular radiotracers are bromine-82 and Iodine-131. Radiotracers have certain advantages over other tracers because they can be utilized in the anionic form that reduces the possibility of absorption and they can be introduced in minute quantities because of the very high detection sensitivity of the present measuring instruments. One disadvantage of the radiotracers is the potential health hazard. However, if strict maximum permissible concentrations are specified, and, if like the tracers mentioned previously, the half-lives are short, there is only a short-term environmental contamination.

Temperature Sensing

Circulating water is known to affect the temperature of the soil or rock through which it flows. Analysis of existing theoretical equations suggest that ground water flow will also affect the surface soil temperature. Therefore soil temperatures may be used to delineate small, shallow ground-water flow systems. The horizontal movement of ground-water in shallow aquifers has also been shown to affect soil temperatures. The aquifer acts as a heat sink or source, depending upon the season of the year, and the heat is exchanged between the aquifer and the land surface.

The greater the velocity of water movement through a porous medium and the resulting larger mass movement in the system, the greater the effect of the fluid movement on the aquifer temperature and thus on the soil or rock temperature. In shallow flow systems in which the rate of movement of water is low, only the effect of vertical movement will be reflected in the soil temperatures.

During the summer months, warm water enters the recharge zone, moves downward, and warms the soil. As the water moves horizontally, it comes into equilibrium with the general thermal regime, and as the water moves upward toward the point of discharge, the soil is cooled. In the winter, the water entering the system cools the soil, and water at the discharge point warms it.

The temperature of the soil within the flow system depends on the velocity of ground water movement, the thermal properties of the materials, heat gained or lost in the atmosphere, and geothermal heat added to the system. The measured soil temperatures of the recharge and discharge areas are functions of the vertical velocity, while the temperatures in between are functions of the horizontal velocity and the distance of travel. In general the soil temperature decreases with increasing horizontal distance between the recharge and discharge zones.

The equipment used generally for temperature sensing consists of a thermistor at the top of an insulated aluminum tipped probe. The probe is inserted into the soil and the temperature is read after coming to equilibrium. At locations where discharge is occurring an increase in soil temperatures of 0.75^{o} C in clays to 5^{o} C in sands has been monitored.

Locations where recharge occurs are not reflected generally in the soil temperature changes; this fact is due probably in part to the intermittent nature of the recharge events.

Infrared Sensing

Remote sensing at different wave bands of the electromagnetic spectrum has been used frequently for the evaluation of soil types and general ground conditions. One of the most useful regions of the spectrum for a remote sensing, in terms of engineering and agricultural applications, is the infrared. Infrared is an electromagnetic radiation whose spectrum band falls between that of the microwave region and visible light, i.e., from 3×10^{11} HZ to 4.3×10^{14} HZ. This energy can be reflected from, absorbed by, transmitted, or emitted by objects. Infrared sensing systems and detection techniques developed in the past few years have made it possible for infrared radiometry or infrared thermal imaging airbourne techniques to be used to detect and record temperature differences, at close range, of less than 0.01^{o} C.

At temperatures above absolute zero (-273^{o} C) all objects radiate electromagnetic energy as a result of their atomic and molecular motion. The amount of emitted radiation increases with the object's absolute temperature and reaches a peak at a certain wavelength. Each object in nature has its own unique property of reflected, transmitted, absorbed, or emitted radiation. These properties, once identified, can be used to distinguish one object from another or obtain information about shape, size, and other characteristics.

The airborne sensing equipment is flown over the terrain in straight parallel lines. A scanning device, such as a rotating mirror, scans the terrain in continuous strips perpendicular to the line of flight. The image from the mirror strikes an element sensitive to infrared radiation and the signal from the sensitive element is electronically amplified and produces a visual image on a cathode ray tube or by means of a glow tube. A final photographic record is made of the glow tube or cathode ray tube. The scanning mirror sweeps an angle on either side of the vertical and an image is recorded, which for the most part is an oblique view of the terrain. The photographic tone of an object sensed by the imaging system is a function primarily of the intensity of the objects infrared radiant intensity, which depends on the object's emissivity and temperature.

The infrared radiance of soils is dependent upon many factors that influence their emissivity and temperature. Such factors can be divided into two general groups, intrinsic and external. The intrinsic factors are those contained in the soil, namely, specific heat, radiating power, heat absorption, nature of surface, moisture content, organic matter content, etc. The external factors consist of the meteorological elements, chief of which are air temperature, sunshine, barametric pressure, wind velocity, humidity, precipitation, etc. Each of these general groups may be subdivided into two parts, one part tending to impact energy to the soil and thereby raising its emittance and the other part tending to take away energy from the soil thereby lowering its emittance. These opposing factors are in operation all of the time, but some predominate over the others at different seasons of the year or even different time of the day. In field thermal imagery, an image taken at any time may be considered the resultant or summation of the effects of these opposing or contrasting factors.

Controlled experiments have been performed in which all intrinsic and external variables were kept constant except for the depth to the ground-water table and time. These experiments indicated clearly that differences in depth to ground-water table within the range used in the experiment (0 to 2 m) were detectable measuring the radiant intensities given off by the soil surfaces in the 8μ to 14μ spectral range.

Microwave Sensing

Microwaves are electromagnetic waves that occupy that portion of the electromagnetic spectrum between shortwave radio waves and infrared radiation, i.e., from 2×10^8 HZ to 3×10^{11} HZ. They propagate readily through dry materials like plastics, ceramics, and wood, but are attenuated to varying degrees in passing through wet materials. The reason for the attenuation is that the microwave induced rotation of the water molecule is a "lossy" process. At relatively high power, this loss is converted into heat that is the basis of microwave heating and cooking.

The original research and development in the utilization of microwaves was performed by the military defense establishment during the 1940's and 1950's. With the formulation of fundamentals and the growth of a large microwave equipment and supply industry, it was only natural that the method should be used to attempt to detect subsurface geotechnical anomalies. Using pulsed systems, a number of investigators, (see table III-1) have had reasonable success in locating rock faults, joints, cavities, and tunnels at depths of hundreds of meters. The systems use generally monocycle pulses in the frequency range from a few megahertz to about 1GHZ and require only the travel time of the wave to the reflecting layer and back and the propagation velocity of the wave in the material through which it is passing to calculate the unknown depth. The method can be

Table III-1

Current Microwave Research and Development in Geotechnical Applications. (Koerner et al, 1979).

Investigator (1)	Affiliation (2)	Status (3)	Application area and equipment details (4)
Cook	Teledyne Geotech Dallas, Tex.	Available	Detection of rock faults, walls, holes, etc. Pulsed monocycle system with propagation depths up to 225 m in rock salt. Frequencies in 1 MHz-100 MHz range.
Rubin	Calspan Co. Buffalo, N. Y.	Available	Close-in detection system, probably pulsed, used to detect land mines.
Moffatt, et al.	Ohio State University and Microwave Assoc. Burlington, Mass.	Available	Detection of rock faults, joints, cavities, and lithologic contrasts. Pulsed monocycle system using orthogonal antennas with 1/2 nsec pulse times for close measurments.
Unterberger	Geophysical Survey Systems, Inc. (GSSI)	Available	Detection of rock faults, caverns, utilities, strata, and other anomalies. Pulsed repetition rate of 50 KHz using a single antenna transceiver in low conductivity soils depths up to 30 m can be monitored.
Dolphin, et al.	Stanford Research Institute	Research	Detected chambers in dolomite rock mine at 30 m-40 m beneath ground surface using a pulsed monocycle system at 16 MHz-50 MHz with peak power of 0.2 MW.
Rubin, et al.	ENSCO, Inc. Springfield, Va.	Research	Used GSSI system with modifications to locate a subsurface tunnel, i.e., Washington Metro where microwaves reflect off heavily reinforced roof of structure which is 6 m-10 m beneath ground surface. Also monitored grout flow and effectiveness.
Unterberger	Texas A&M University	Research	Radar (and sonar) probing in salt to locate tunnels beneath salt floor and fractures in salt. Depths of up to 1,000 m have been successfully probed. Sonar is used to improve probing direction resolution.
Lundien	Corps of Engineers Vicksburg, Miss.	Research	Detection of single and multiple subsurface layers using a continuous wave microwave system.
Ellerbruch	National Bureau Standards	Research	Determined coal seam thickness as small as 19 cm-40 cm thick using continuous wave microwave in the 0.5 GHz-4.0 GHz region.
El-Said	Cairo University	Research	Used continuous wave radio waves from 77 KHz-940 KHz to determine water levels in Egyptian desert. Predicted water at 861 m which agreed with nearby boring data of 790 m and 875 m.
Koerner, et al.	Drexel University	Research	Continuous microwave system at L-Band frequencies used to locate ground water, top of rock and utilities to depths of 4 m.

classified as a microwave reflection method.

Research is also being done with continuous wave microwave methods whereby the receiving antennae is monitoring continuously the returning microwave signals from various subsurface reflecting layers. This results in a series of constructive and destructive interference patterns when viewed on an oscilloscope or recorder. From theoretical analyses of the electromagnetic interference patterns produced as a function of depth to a reflecting interface, an equation can be developed which requires the index of refraction of the medium, the angle of incidence, the frequency of two peaks of the interface pattern, and the number of peaks between these two frequencies.

Geophysical Methods

Geophysical methods of subsurface seepage detection include the seismic techniques of refraction and reflection and the electrical methods of resistivity and streaming potential. While seismic techniques can be used to locate the water table, electrical prospecting also can give flow velocity and rates. Interpreting seismic data, though, is much simpler than for the electrical results. Dobrin's (1976) text is an excellent introduction to any of the geophysical techniques.

Seismic Methods

Seismic refraction or reflection can be used to locate the water table because of the difference in elastic properties of dry versus wet soil. Both techniques measure the travel time of a seismic wave, usually a blast, transmitted through the soil and back to the surface where it is picked up by the geophone. The longitudinal or P-wave component of the blast wave is timed because it is the first wave to arrive at the geophone and is not attenuated by water. Both types of seismic surveys use several geophones spaced at equal intervals to obtain as much information as possible per blast.

Refraction surveys measure, see figure III-1, the time it takes for a blast wave to return to the surface after refraction by a higher velocity, subsurface layer such as soil containing the water table. Blast waves will travel through the overlying soil until they hit the surface of the saturated soil where some are reflected back to the surface while others are transmitted through the wet soil. Only those waves striking the saturated soil's surface at a critical angle equal to the arcsine of the ratio of the P-wave velocities of the dry versus saturated soil will be refracted and, thus, travel along the wet-dry interface. These refracted waves return also to the ground surface at the same critical angle described previously. By knowing the various distances and travel times from the blast to several geophones at increasing distances from the blast, the depth to the saturated-dry interface may be determined.

Refraction data are analyzed normally by drawing a plot of first arrival time versus distance for each geophone. The plot is linear for small distances with a slope equal to the inverse of the upper soil layer velocity. As the distance between the geophone and the blast increases, the refracted blast wave travelling in the higher velocity, i.e., the lower soil layer, will overtake eventually the upper layer wave and be the first to arrive at the geophone. The curve will show a break where the refracted wave overtakes the surface blast wave. The time-distance curve will continue its linear trend except now at a new slope equal to

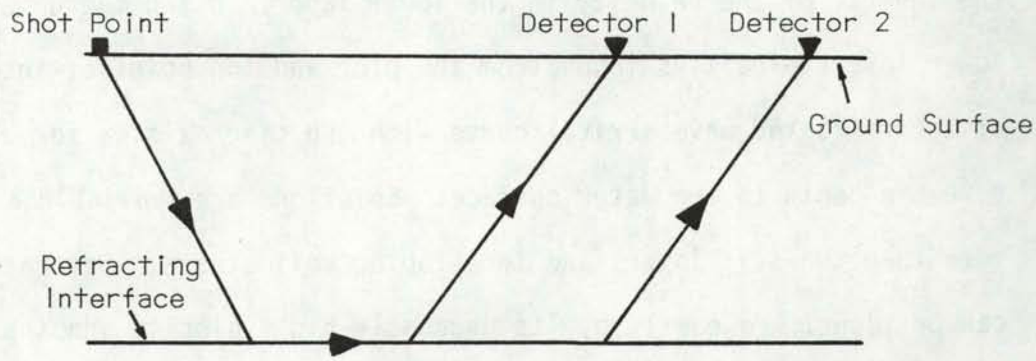


Figure III-1. Wave Path for Seismic Refraction Method.

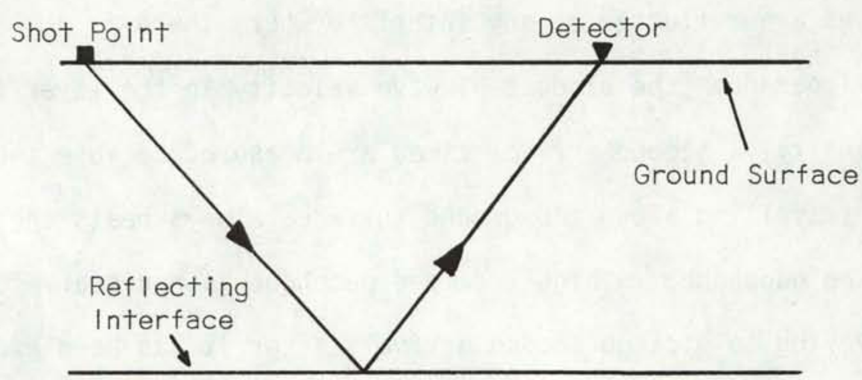


Figure III-2. Wave Path for Seismic Reflection Method.

the inverse of the velocity in the lower layer. Using the upper and lower layer velocities found from the plot and the point of intersection of the refracted wave arrival curve with the time axis, a formula will give the depth to the water surface. Solutions are available also for more than two soil layers and for sloping soil strata. Saturated soil can be identified easily by its unusually high velocity and the differing wave form of the refracted wave.

Reflection surveys, see figure III-2, measure the time for a blast wave (P-component) to return to the surface after reflection from the water table surface due to a difference in soil properties. Although reflection surveys are used most extensively for oil they also have been used for engineering purposes. Low velocity below high velocity strata cannot be detected by refraction and, thus, give false data while reflection surveys can detect them (although with some difficulty). Blast waves are reflected at any interface where there is a change in acoustic impedance (the product of wave velocity in the layer times the layers density). Second arrival times are measured because the first arrival, travelling along the ground surface, always beats the reflected wave to the geophone. A highly damped geophone is necessary for reflection surveying to pick up second arrivals after it has been excited, and is usually still vibrating, due to the first arrival.

To determine the depth to a reflecting layer, the travel time and distance from the blast to a geophone, for the second arrival, must be known. Also, the velocity in the upper soil layer must be determined either from separate measurements in a borehole or by the inverse square root of the slope of a curve of the square of the second arrival time versus the distance squared to the geophone. Once these parameters have been determined, a formula is used to determine the depth to the water table. Techniques are available also to analyze more than two soil layers and for dipping strata.

Regarding seismic methods in general, there are several controls necessary for accurate results:

1. Elevations of the blast and pickup geophones must be accounted for.
2. The influence of weathered or loose, near surface soil must be accounted for since its elastic properties differ from the soil below.
3. A correction for the depth of capillary zone in fine grained soils is necessary; and
4. Background noise can limit the depth capable of being measured for a given blast energy.

Electric Methods

Electric techniques have been used to give depth to the water table and can also give information on flow rate and velocity. The first technique described, resistivity, involves generating an artificial current in the soil. The second, streaming potential method, measures natural currents in the soil caused by seepage.

The resistivity method, see figure III-3, involves generating a current in the ground by two electrodes and measuring the potential drop with several electrodes placed in between. The goal of a resistivity survey is to determine resistivity variations in the ground or rather the ground's apparent resistivity. Apparent resistivity is proportional

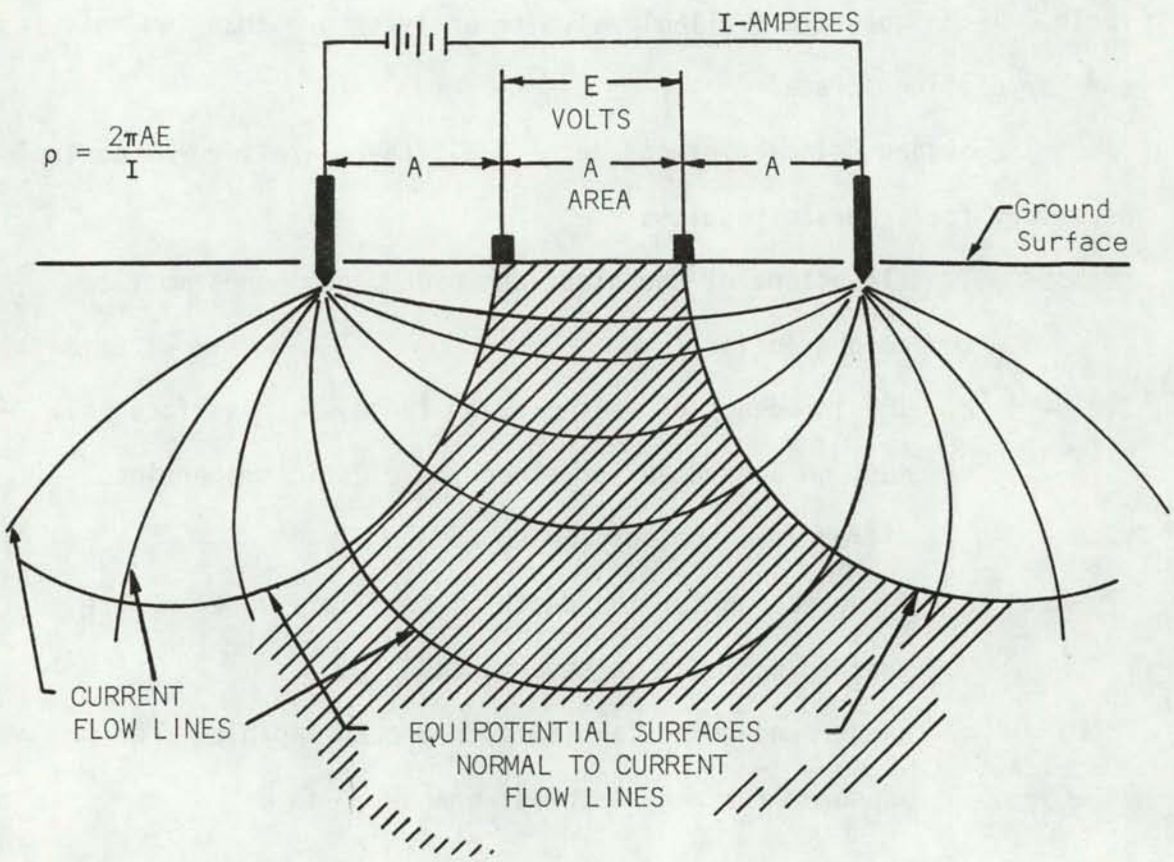


Figure III-3. Resistivity Method. Electrode Configuration for the Wenner Arrangement, in Common Use in the United States.

to the measured potential drop and inversely proportional to the generated current. Electrode configurations are varied during the survey and apparent resistivities are calculated for each configuration, thereby, yielding subsurface stratigraphic information. A common electrode configuration used in the United States is the Wenner arrangement where equal distances are maintained between all electrodes.

Two types of resistivity surveys are used: horizontal and vertical profiling. When horizontal profiling is used, the electrode separation is kept constant and the entire set of electrodes is moved along a survey line. Since a wider electrode spacing allows deeper probing but less sensitivity (a closer spacing does the reverse), a critical spacing is chosen for the depths desired to be investigated in the field. As the electrodes are moved along a survey line, any change in apparent resistivity indicates an anomaly. A plot of apparent resistivity versus distance can be compared with theoretical anomaly curves in an attempt to identify the subsurface anomaly. Ground water is located easily since even in a nonhomogeneous soil, the electric properties do not show as much contrast as for saturated soil having five to ten times less resistivity. Vertical profiling consists of increasing the electrode separation of a single survey station in order to probe deeper continuously. A sharp decrease in apparent resistivity may indicate a saturated soil surface interface. Combinations of horizontal and vertical profiling are used commonly for reconnaissance site investigations.

To interpret vertical profiling results, a set of empirical curves called Wetzel-Mooney Curves is used to determine the depth to an interface and it resistivity. These curves are log-log plots of apparent resistivity versus electrode separation and are matched to the similar curves determined from field investigations. Approximately 488 curves exist for various resistivities and depths for both two and three layer cases. Other simplified methods also exist to interpret resistivity data.

When conducting a resistivity survey, profiles must be run parallel to elevation contours and corrections made for the cpaillary zone above the water table in fine grained soils (i.e., by correlation with boreholes). Depth of penetration for the Wenner electrode configuration is of the same order as the spacing between adjacent electrodes.

Streaming potential (SP) measurements have been used in the U.S.S.R. not only to find leakage sites in reservoirs but also to monitor the evolution of seepage and soil piping in dams. The SP method is based on a well known phenomenon of electric fields (natural) being set up when water flows through any porous medium such as soil.

Streaming potentials are casued by interactions at the boundary of the solid and liquid phases where a double, diffuse layer is formed. The part of the double layer in the liquid phase can be moved out of its equilibrium position by the latter's movement thereby creating a potential difference or SP anomaly. Equations have been developed to relate the magnitude of the SP anomaly to the hydrostatic head causing it but they only hold for perfect tubes and not for complicated case of a real soil. These equations show that the potentials increase in the direction of water flow and that their gradient is proportional to the seepage intensity (for laminar flow). When the flow is turbulent as in large open fissures, the law is violated. Laws governing the origin of SP occurring in fissures have not yet been established. Sands with medium grain sizes develop the greatest potential fields. If the pore water has a salinity corresponding to a resistivity of 10 ohm/cm or less, no noticeable SP anomalies will be observed.

Maps of equal SP values may therefore reflect the water table surface. Increases in flow can be noted by contour changes in time on a SP contour map. Since it is similar to the resistivity method, one method can be used to check the other. When runing an SP survey, corrections must be made for the capillary zone above the water table.

In general, the electrical methods are used to find depths to the water table or flow rate but the interpretation of these data are complicated and many natural obstacles must be avoided when performing the surveys (i.e., natural currents in the earth and steel fences, to name a few). Some researchers have used stationary electrodes to monitor the changes in seepage and, therefore, the ground's electric properties, with time. Efficiency of grout curtains at reducing seepage and soil piping progress in dams have been observed using the simplified stationary electrode method.

Acoustic Emission Monitoring

Acoustic emissions are the noises generated when a material deforms thereby mobilizing some form of stress wave. The monitoring technique that quantifies these stress waves is well advanced in metal and rock testing and has been extended lately into the soils area. The instrumentation, see figure III-4, consists of:

1. A wave guide inserted into the general soil zone being monitored. A wave guide can be any material that is

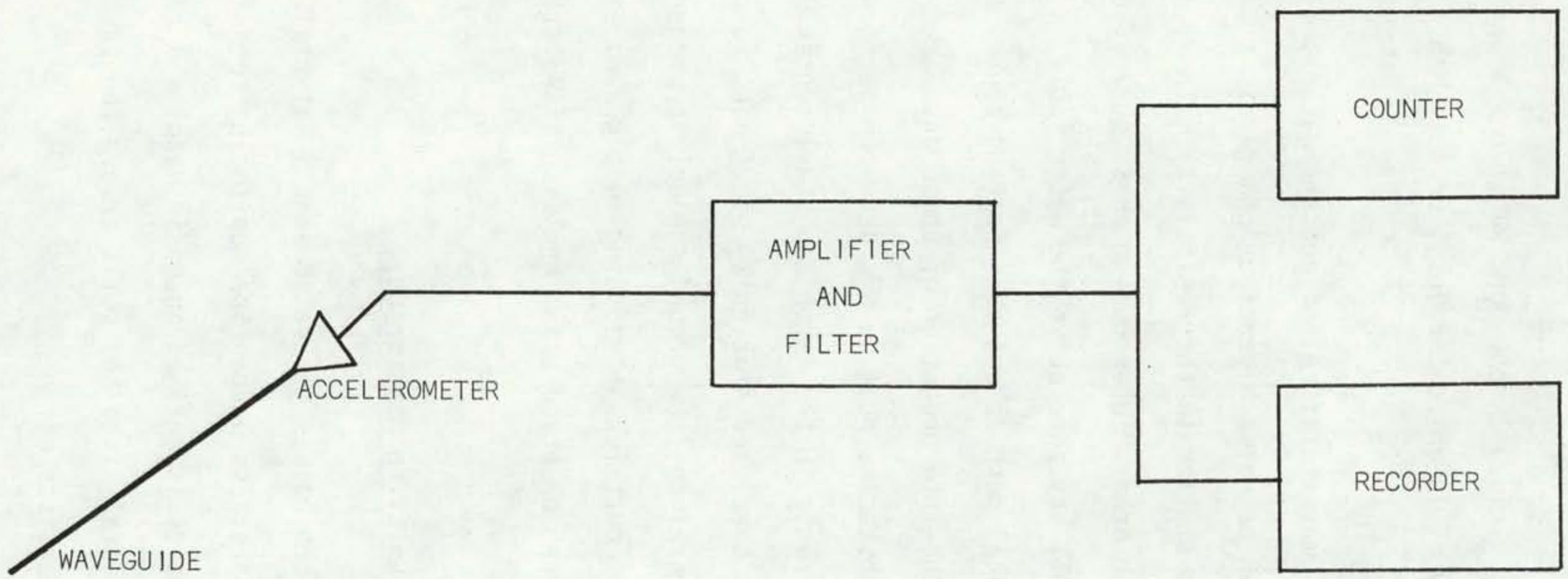


Figure III-4. Schematic Diagram Showing Acoustic Emission Monitoring System.

capable of carrying acoustic emissions from depth to the surface, (i.e., concrete drains, metal pipes, conduits, metal rebar, or metal wire).

2. A pickup transducer, usually a piezoelectric transducer or commonly an accelerometer.
3. An amplifier, usually equiped with a filter.
4. A readout system which is usually a Light Emitting Diode and if a permanent record is desired a tape recorder and/or strip-chart recorder.

The readout system is often a frequency counter but also can be based on amplitude or root-mean-square for continuous emission monitoring. The frequency response of the transducer is of major importance and can vary over wide limits (e.g., in metals, transducers are responsive typically in the Kilohertz or even Megahertz range. In soils accelerometers or hydrophones are used with a frequency response from 100 HZ to 10,000 HZ, and rock, geophones are used with a frequency response from 0.25 HZ to 100 HZ.) The most suitable frequency range for monitoring seepage flow depends greatly on the manner of pickup. If the pickup is made directly at the source of emissions, i.e., downhole or in water monitoring, the pickup transducer can be of high frequency, which will eliminate background noise. Conversely, the farther away the transducer pickup is from the source of the emissions, the lower the frequencies that must be utilized. This allows for longer transmission of the emission along the wave guide without complete attenuation of the signal.

The technique has been used recently to detct seepage beneath a small earth dam of 13 feet (4 m) in height and approximately 1312 feet

(400 m) in length. While grouting was an obvious solution to the problem, the cost of grouting the entire length of the dam was prohibitive to the site developer. Thus a series of borings was made along the axis of the dam and seepage tests were conducted.

Since the borings were available, acoustic emission monitoring was attempted. A heavy steel wire was inserted down the borehole to the bottom where seepage was occurring presumably. Acoustic emission count rates were recorded using accelerometers responsive in the 500 HZ to 8,000 HZ range. There was a general agreement between seepage monitoring (in gallons/minute) and acoustic emission activity (counts/minute). The actual mechanism causing the emissions is not known (perhaps it was the turbulent flow of the seepage against and around the casing), but the use of acoustic emission technique in monitoring for seepage seems to hold great promise.

In another application (Coxon and Crook, 1976), a simple microphone from a telephone coil ear piece was connected to 656 feet (200 m) of coaxial cable, sealed to make it water tight, and lowered from a boat to locate seepage through an asphalt covered upstream surface of a 197 foot (60 m) high dam. The noise was recoded using a portable AC micro-range voltmeter and a set of earphones. Leakage noises were unmistakable and produced noise levels of 15 dB to 20 dB above background levels. Subsequently, divers examined the suspect areas and found cracks more than 1 cm wide leaking reservoir water (Koerner, Reif, and Burlingame, 1979).

Piezometer Tube

Piezometers measure water level in an embankment or dam. Installation can be during or after construction. Piezometer tubes consist of a metal or plastic pipe, capped usually at the bottom, with holes or slits in the sides to allow water to enter. As the water level changes in a dam or embankment, the corresponding water level will change within the tube. Pressure cell piezometers have the advantage of no wait response time as with low permeability soils.

Boring Logs

One of the better ways to determine the subsurface conditions at a specific location is by boring an exploration hole. With such a boring it is possible to inspect visually soils representing the exact situation at a given depth. Also, since the borehole exists there are some downhole detection techniques that can be used.

For most small projects, borings would probably be cost prohibitive. Other disadvantages include:

1. Difficulty in taking borings in some terrain, e.g., over water or on steep slopes; and
2. Interpolation is required for information between borings (Koerner, Reif and Burlingame, 1979).

A summary of log applications, Table III-2 (Keys and MacCary, 1971), is a simplified table of parameters that can be measured or interpreted from commonly available geophysical logs.

Table III-2

Summary of Log Applications. (Keys and MacCary, 1971).

Required information on the properties of rocks, fluid, wells, or the ground-water system	Widely available logging techniques which might be utilized
Lithology and stratigraphic correlation of aquifers and associated rocks.	Electric, sonic, or caliper logs made in open holes. Nuclear logs made in open or cased holes.
Total porosity or bulk density	Calibrated sonic logs in open holes, calibrated neutron or gamma-gamma logs in open or cased holes.
Effective porosity or true resistivity	Calibrated long-normal resistivity logs.
Clay or shale content	Gamma logs.
Permeability	No direct measurement by logging. May be related to porosity, injectivity, sonic amplitude.
Secondary permeability - fractures, solution openings	Caliper, sonic, or borehole televiewer or television logs.
Specific yield of unconfined aquifers	Calibrated neutron logs.
Grain size	Possible relation to formation factor derived from electric logs.
Location of water level or saturated zones	Electric, temperature or fluid conductivity in open hole or inside casing. Neutron or gamma-gamma logs in open hole or outside casing.
Moisture content	Calibrated neutron logs.
Infiltration	Time-interval neutron logs under special circumstances or radioactive tracers.
Direction, velocity, and path of ground-water flow.	Single-well tracer techniques - point dilution and single-well pulse. Multiwell tracer techniques.
Dispersion, dilution, and movement of waste	Fluid conductivity and temperature logs, gamma logs for some radioactive wastes, fluid sampler.
Source and movement of water in a well	Injectivity profile. Flowmeter or tracer logging during pumping or injection. Temperature logs.
Chemical and physical characteristics of water, including salinity, temperature, density, and viscosity.	Calibrated fluid conductivity and temperature in the well. Neutron chloride logging outside casing. Multielectrode resistivity.
Determining construction of existing wells, diameter and position of casing, perforations, screens.	Gamma-gamma, caliper, collar, and perforation locator, borehole television.
Guide to screen setting	All logs providing data on the lithology, water-bearing characteristics, and correlation and thickness of aquifers.
Cementing	Caliper, temperature, gamma-gamma. Acoustic for cement bond.
Casing corrosion	Under some conditions caliper, or collar locator.
Casing leaks and (or) plugged screen	Tracer and flowmeter.

Magnetic Method

Magnetic surveys map variations in the magnetic field of the earth which are attributable to changes of structure or magnetic susceptbility in certain near-surface rocks. Sedimentary rocks generally have a very small susceptability compared to igneous or metamorphic rocks and most magnetic surveys are designed to map structure on or within the basement, or to detect magnetic minerals directly (Dorbin, 1976).

Gravity Method

The gravity method measures minute variations in the pull of gravity from rocks within the first few miles of the earth's surface. Different types of rocks have different densities and the denser rocks have the greater gravitational attraction. If the denser rocks are arched upward in a structural high, such as an anticline, the earth's gravitational field will be greater over the axis of the structure than along its flanks. A salt dome, on the other hand, which is less dense than the rock into which it has intruded, may be detected from the low value of gravity normally recorded above it.

Anomalies in gravity which are sought in oil or mineral exploration may represent only one millionth or even one ten-millionth of the earth's total gravitational field. For this reason, gravity instruments are designed to be extremely sensitive, and modern gravimeters can detect variations in gravity to within one hundred-millionth of the earth's field (Dobrin, 1976).

CHAPTER IV

SELECTION OF METHOD

Introduction

The method of selection for dam analysis was done by the process of:

1. The project geotechnical and electrical engineers discussed each method;
2. Personal interviews were conducted with representatives of the various manufacturing companies;
3. Personal interviews were conducted with consulting geophysicists in the area; and
4. A final discussion of the methods, costs and results of the interviews was conducted and a method was selected.

The method selected was acoustic emission. The reason for its selection was, acoustic emission met the initial requirements set, which again are:

1. Inexpensive. The Acoustic Emission Technology unit sells for approximately $4,000.00. This includes all accessories to begin a monitoring program. Usually a permanent recording device, strip chart recorder or magnetic tape, is used to keep accurate records, price on these may vary.

2. Portable. The Acoustic Emission Technology (A.E.T.) unit weights six pounds. Total weight of A.E.T. unit, accessories, and strip chart recorder is under fifteen pounds.
3. Ease of operation. The A.E.T. unit is very simple to use, does not require a technical background to operate and requires only a few minutes to set up and become operational.
4. Ease of data interpretation. Using a strip chart recorder to keep a permanent record makes for ease of operation and interpretation of data. A glance over the chart will reveal much of the desired information.

Acoustic Emission

A general review of acoustic emission was given in Chapter III under detection methods but this section describes the technique in more detail. As stated in Chapter III, acoustic emissions are the noises generated when a material deforms thereby mobilizing some form of the stress wave.

History of Acoustic Emission Monitoring

The first experiment to detect and record acoustic emission was conducted by Forster and Scheil (1936). They recorded the "noises" caused by the formation of martensite in 29% nickel steel. The specimen was supported between 1 mm diameter molybdenum wires. The vibrations caused by martensitic transformation in the specimen were transmitted by the supporting wire, which acted as a waveguide, to a receiving transducer, which transformed the noise into electric signals. The electrical signals passed through an amplifier, were rectified, further amplified, and recorded with a galvanometric light-beam recorder.

E. A. Hodgson in Canada was the first modern day scientist to propose a practical application of sub-sudible rock noise (acoustic emission). He proposed their use to predict rock burst and earthquakes in 1923.

In the 1930's Obert (1941) was measuring seismic velocity in a mine pillar, when he discovered that his equipment was being triggered by noises generated by the rock. Obert and Duval (1941) investigated eventually the possibility of predicting rock bursts utilizing these signals and found an increased acoustic emission rate preceeding rock burst. Their work, in the Ahmeek Copper Mine in Michigan, proved successful. The predicted 14 rock bursts; 9 were followed by rock bursts. There were 5 predictions not followed by bursts, and two unpredicted bursts.

The father of acoustic emission, Josif Kaiser, published his Ph.D. thesis in 1940, which was the first study of the acoustic emission phenomenon itself. Kaiser's research objectives were to determine from tensile tests of conventional engineering materials what noises are generated from within the specimen. The acoustic process involved, the frequency levels found, and the relation between the stress-strain curve and the frequencies noted for the various stresses to which the specimens were subjected. His most significant discovery is the phenomena of irreversibility which is now known as the Kaiser effect. The Kaiser effect is stated as the absence of detectable acoustic emission until previously applied stress levels are exceeded (Leaird, 1980).

Tatro and Liptai (1962) used the technique as a yield detector in metals and also did pioneering work in analyzing the fundamental characteristics of acoustic emissions in metals. Recently, the most active acoustic emission work has been in the area of nuclear pressure vessel proof testing (Green, 1969). A large number of transducers are placed on the pressurized vessel. Any flaws that may be present are detected and evaluated by their acoustic emission response. These flaws can be located to within inches of their actual locations.

While the materials mentioned previously, rocks and metals, have been subjects of major acoustic emission research, other materials have been evaluated also. These include composites, concrete, ceramics, ice, and wood.

Information regarding the acoustic emission response to soils is relatively new. Early soils reference was addressed in a preliminary manner by Caddman and Goodman (1967). Subsequent work has been done at Drexel University and forms a large part of acoustic emission response of soils to date (Koerner, Lord, McCabe and Curran, 1976).

Characteristics of Acoustic Emission from Soil

The mechanisms responsible for the shear strength of soils appear to be the basic generators of acoustic emission in soils. These mechanisms in granular soils are the fundamental components of the angle of shearing resistance, including sliding friction, rolling friction, degradation, and dilatation. Work by Koerner (1977) has shown that conditions producing the greatest number of interparticle and therefore frictional contacts, i.e. well-graded soils, also produce the greatest amount of acoustic emission activity. The tendency of a granular soil to generate more emissions with higher confining pressures and consequently higher frictional forces, is further evidence of a friction based emission source.

It has also been shown that frictional characteristics of soil particles vary with mineral type (Horn and Deere, 1962). From this, one would conclude that mineral type will also affect acoustic emission, although this hypothesis has not as yet been tested.

Instrumentation

See figure III-4 for schematic diagram of acoustic emission test setup.

Waveguide

A waveguide transmits the acoustic emissions from depth to the surface. Waveguides are necessary because acoustic emissions are attenuated rapidly in soils; from 1 dB/ft to 200 dB/ft (0.33 dB/cm to 0.67 dB/cm). Attenuation rate is affected by types of soil and moisture content. A waveguide may be a section of low carbon steel rod (e.g., bar stock), reinforcing bar (rebar), baling wire, metal instrument pipe, metal drain, outlet pipe, etc. Attenuation rates in steel rods have been measured from 0.005 dB/ft to 0.1 dB/ft (1.7×10^{-5} dB/cm to 3.3×10^{-3} dB/cm). The waveguide must be located in, or near, the highly stressed zone in the soil being monitored (Koerner, Lord, and McCabe, 1978).

Accelerometer

As a soil mass deforms, the resulting acoustic emissions are received by the metal waveguide and transmitted to the accelerometer attached to the waveguide at the ground surface.

The laboratory testing of a number of soils, and subsequent analysis, has shown the frequency of typical soil emissions to be in the 1-8 KHz range (Koerner and Lord, 1974). Therefore the range of the accelerometer must cover this. Koerner, Lord and McCabe (1978) used a piezoelectric transducer with a relatively flat frequency response from 500 Hz to 5,000 KHz, a resonance at 5,700 Hz and voltage sensitivity of approximately 100 mr/peak g, which is in the range of greatest interest in acoustic emission monitoring of soils.

Filter

To eliminate extraneous noises, the signal is usually filtered.

Amplifier

The signal is usually amplified by .01 g then filtered.

Counter

When working with acoustic emission in the field, the count mode is used. What takes place follows this sequence; deforming soil creates acoustic emissions which are transmitted up the waveguide to the accelerometer which is then vibrated changing the "noise" to an electrical impulse. This impulse is then amplified and filtered. Before monitoring begins a threshold voltage is set up. If the impulse from the accelerometer is higher than the set threshold, a count is registered. Since each acoustic emission is actually a rapid series of sound waves, the electrical signal produced by the accelerometer takes the form of a logarithmically decaying transient wave train. Consequently, a single emission could produce a large number of counts. The number of counts, not the number of emissions, is referred to as "acoustic emission count" (Koerner, Lord, and McCabe, 1977).

Strip Chart Recorder

A strip chart recorder may be used along with the acoustic emission instruments to keep a permanent record of counts at a site. The line trace on the strip chart paper also makes for easier interpretation of the data.

Acoustic Emission Response of Dry Soils

To determine the acoustic emission response in dry soils, Lord and Koerner (1974) investigated the response of a series of axially stressed dry soil samples.

Two different types of soils were used, a decomposed mica schist entirely in the sand range, and a finer alluvial clayey silt. The soils were ovendried, sieved, and blended in the following proportions by weight:

Soil No.	Sand %	Clayey Silt %
1	100	0
2	67	33
3	50	50
4	33	67
5	0	100

After blending, the soils were mixed with water to their optimum moisture content (about 12%) and compacted to maximum density. The specimens were then ovendried at 105°C for 24 hours. These cylinders were 1.3 inches (3.3 cm) in diameter and 2.8 inches (7.1 cm) high with average density of about 113 lb/ft^3 or 1.8 g/cm^3.

The soil samples were tested in unconfined compression using a hand-operated press in a stress controlled manner. A dial deflection gage was mounted on top of the specimen so that strains could be computed. The waveguide, a ¼ inch steel drill rod, was placed against and slightly into the soil specimen under test.

Soil Response

The stress-strain and stress-acoustic emission responses of the five soils tested are presented in figure IV-1. In comparing one set of curves to another, the following observations can be made:

1. The maximum stress reached in each of the five different soil types was high (from 185 to 345 psi) due to the method of sample preparation. This assured good acoustic emission response without the necessity of large amplification.
2. The axial strain at failure was about four percent for all samples.
3. At low stress levels, the acoustic emission output is much more pronounced than strain output. This difference, however, depends on the soil type, the difference becoming less as the amount of fines in the soil increases.
4. The acoustic emissions are greater in sandy soil than in fine-grained soil. This suggests that sliding friction is

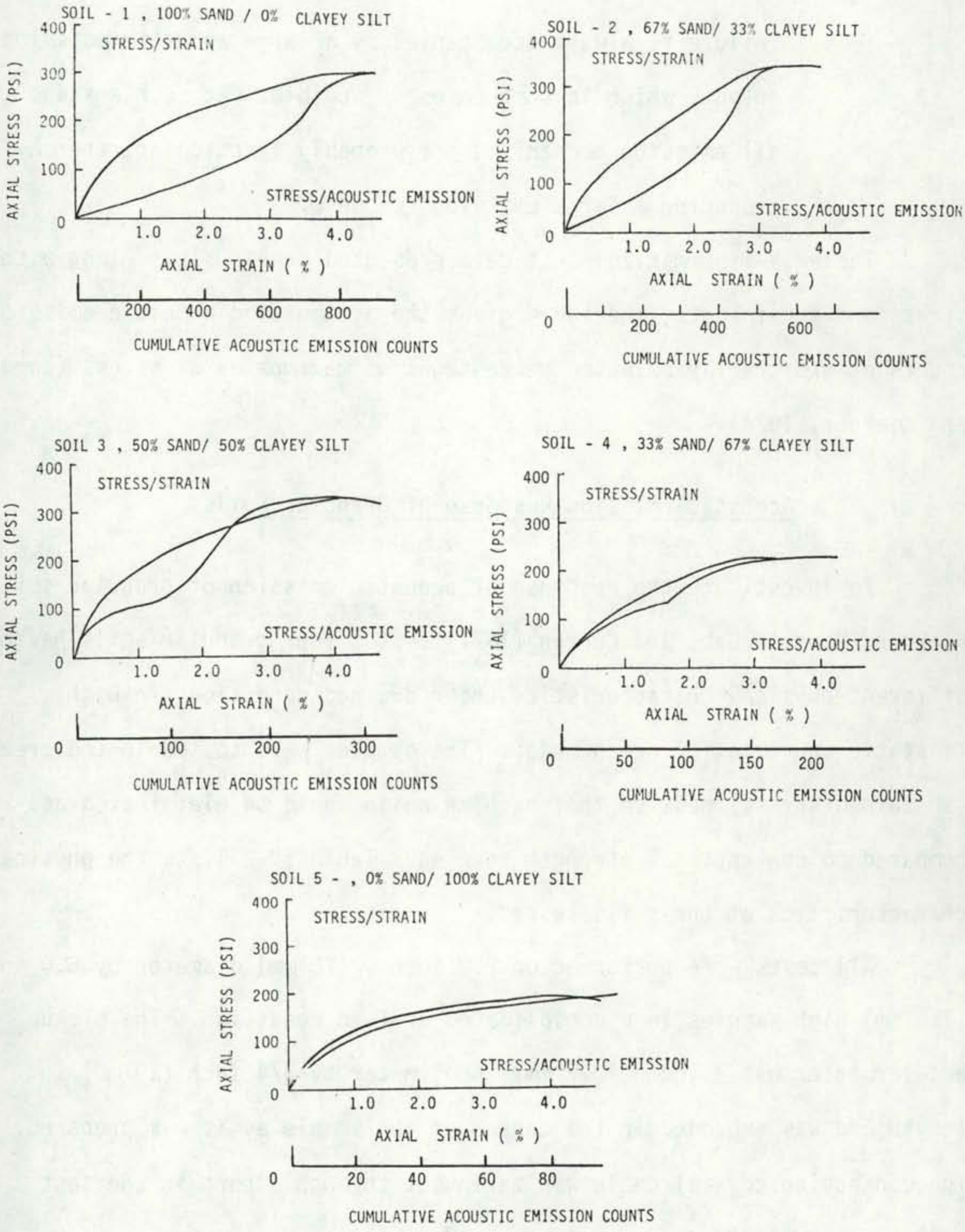


Figure IV-1. Typical Stress/strain and Stress/acoustic Emission Response to Soils 1-5. (Lord and Koerner, 1974) probably more significant than rolling friction in producing acoustic emissions.

5. Failure is always accompanied by a large acoustic emission output, which in some cases is audible. At such a stage all emission mechanisms are probably functioning, thereby producing a large emission response.

Table IV-1 summarizes test data presented previously. Along with stress and strain data, the table gives the accumulated acoustic emission counts at arbitrarily selected percentages of maximum axial stress (Lord and Koerner, 1974).

Acoustic Emission Response of Granular Soils

To investigate the response of acoustic emission of granular soils, Koerner, Lord, McCabe and Curran (1977) tested four granular soils having different physical characteristics under drained conditions in both isostatic and triaxial creep modes. The samples were tested in the creep (sustained stress) mode so that machine noise would be eleiminated as compared to conventional strength testing. Table IV-2 lists the physical characteristics of the soils tested.

All tests were performed on 2.8 inches (70 mm) diameter by 6.0 inch (150 mm) high samples in a consolidated drained condition. The pickup accelerometer was ½ inch (12.7 mm) in diameter by 3/4 inch (19 mm) in length and was embedded in the center of the sample as it was prepared. The connecting coaxial cable was taken out through a port in the test cell to an amplifier and counter.

Table IV-1

Summary of Test Data for Dry Soils. (Lord and Koerner, 1974)

No.	Soil, % Sand/ % Clayey Silt	Max Stress psi	Strain at Failure, %	Total Acoustic Emission Count at 100% of Max Load Stress	90%	75%	50%	25%
1	100/0	296	4.4	860	780	720	580	350
2	67/33	344	3.6	600	475	440	350	180
3	50/50	327	4.2	280	200	160	130	30
4	33/67	231	3.3	210	135	100	70	20
5	0/100	185	4.0	105	80	40	20	5

Table IV-2

Effect of Particle Characteristics on Acoustic Emissions in Granular Soils. (Koerner and others, 1976)

Soil type (1)	Particle shape[a] (2)	Coefficient of uniformity[b] (3)	Effective size[c] (4)	Friction angle, in degrees (5)	Cell pressure, in pounds per square inch (6)	AE_{ISO}[d] (7)	E_{T}[e] (8)	AE_{TRIAX}[f] (9)
Sand Drain Soil No. 1	Subangular	8.4	0.45	35	5	1.7×10^4	111×10^2	2×10^5
					10	7.0×10^4	45×10^2	3×10^5
					20	15.0×10^4	45×10^2	12×10^5
Ottawa Sand No. 2	Round	2.0	0.20	35	5	0.2×10^4	36×10^2	2×10^5
					10	0.5×10^4	36×10^2	3×10^5
					20	1.2×10^4	17×10^2	4×10^5
Concrete Sand No. 3	Angular	2.4	0.21	39	5	0.04×10^4	56×10^2	8×10^5
					10	0.2×10^4	38×10^2	9×10^5
					20	1.8×10^4	33×10^2	14×10^5
Beach Sand No. 4	Subround	1.5	0.24	42	5	0.01×10^4	7×10^2	1×10^5
					10	0.10×10^4	7×10^2	2×10^5
					20	0.38×10^4	6×10^2	4×10^5

[a]Based on a relative scale of angular, subangular, subround, round, or very round.
[b]Defined as $CU = d_{60}/d_{10}$.
[c]d_{10}, the particle size at which 10% of the entire sample is finer, given in millimeters.
[d]Cumulative acoustic emission counts under isostatic conditions at cell pressure equilibrium.
[e]Coefficients of emittivity, i.e., slope of initial portion of AE versus deviator stress curve in units of counts per pounds per square inch for triaxial tests.
[f]Cumulative acoustic emission counts under triaxial creep conditions at failure.

In the first series of tests, hydrostatic pressure was applied to the specimen producing isostatic conditions. Cumulative acoustic emission counts were recorded with time after the pressure increment was applied. Figuer IV-2 shows the response curves for these tests. Other than the final level of acoustic emission counts, the time for the acoustic emission to cease, i.e., equilibrium of particles reorientation, varied primarily with shape. Samples containing the rounder particles (soils labeled No. 2 and No. 4) ceased emitting much before those samples with angular particles.

Using the same soils and experimental test setup, a series of triaxial shear creep tests were performed. The deviator stress (or principal stress difference) versus strain behavior is given in figure IV-3, and the deviator stress versus acoustic emission behavior is given in figure IV-4 for the four soils under consideration. Note the almost identical behavior patterns of stress/strain and stress/acoustic emission curves at all levels of confining pressure. This behavior indicates a basic relationship between strain and acoustic emission. In addition to listing the limiting acoustic emission counts at failure (Col. 9 of table IV-2), a modulus of emittivity was also calculated. The value in table IV-2 is expressed in units of counts per pound per square inch since the value is intuitively more helpful on a unit stress basis.

Summary

Particle Shape. The more angular the soil particles contained within the total sample, the more emittive is the sample under stress. Sample No. 3 and No. 1 (angular and subangular, respectively) are significantly more emittive in both the initial and final stages of triaxial

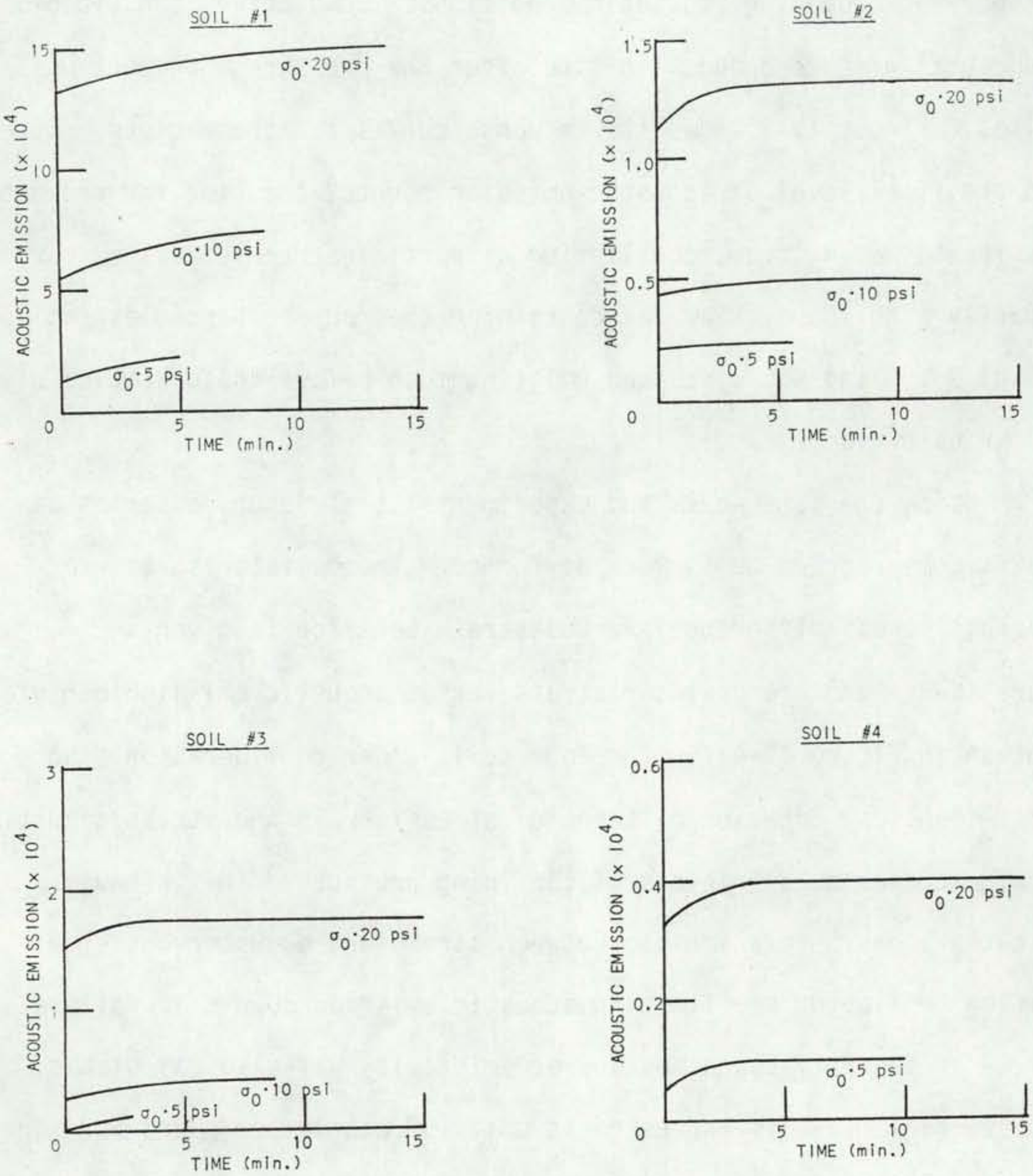


Figure IV-2. Isostatic Test Results (Time Versus Acoustic Emission in Units of 10,000 Counts) for Four Granular Soils Listed in Table IV-2. (Koerner, Lord, McCabe, and Curran, 1976)

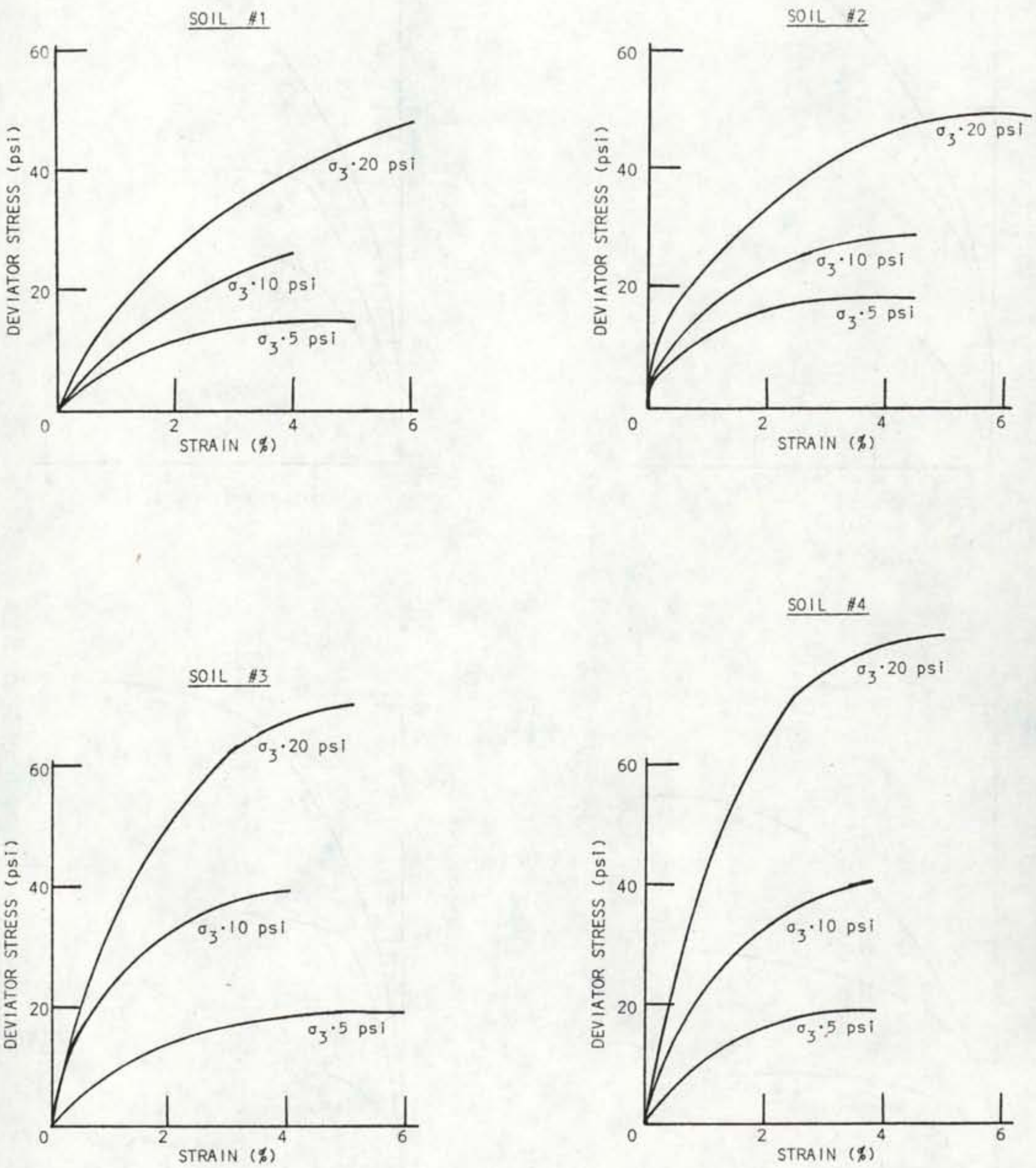


Figure IV-3. Triaxial Shear Test Results (Deviator Stress Versus Strain) for Four Granular Soils Listed in Table IV-2. (Koerner, Lord, McCabe, and Curran, 1976)

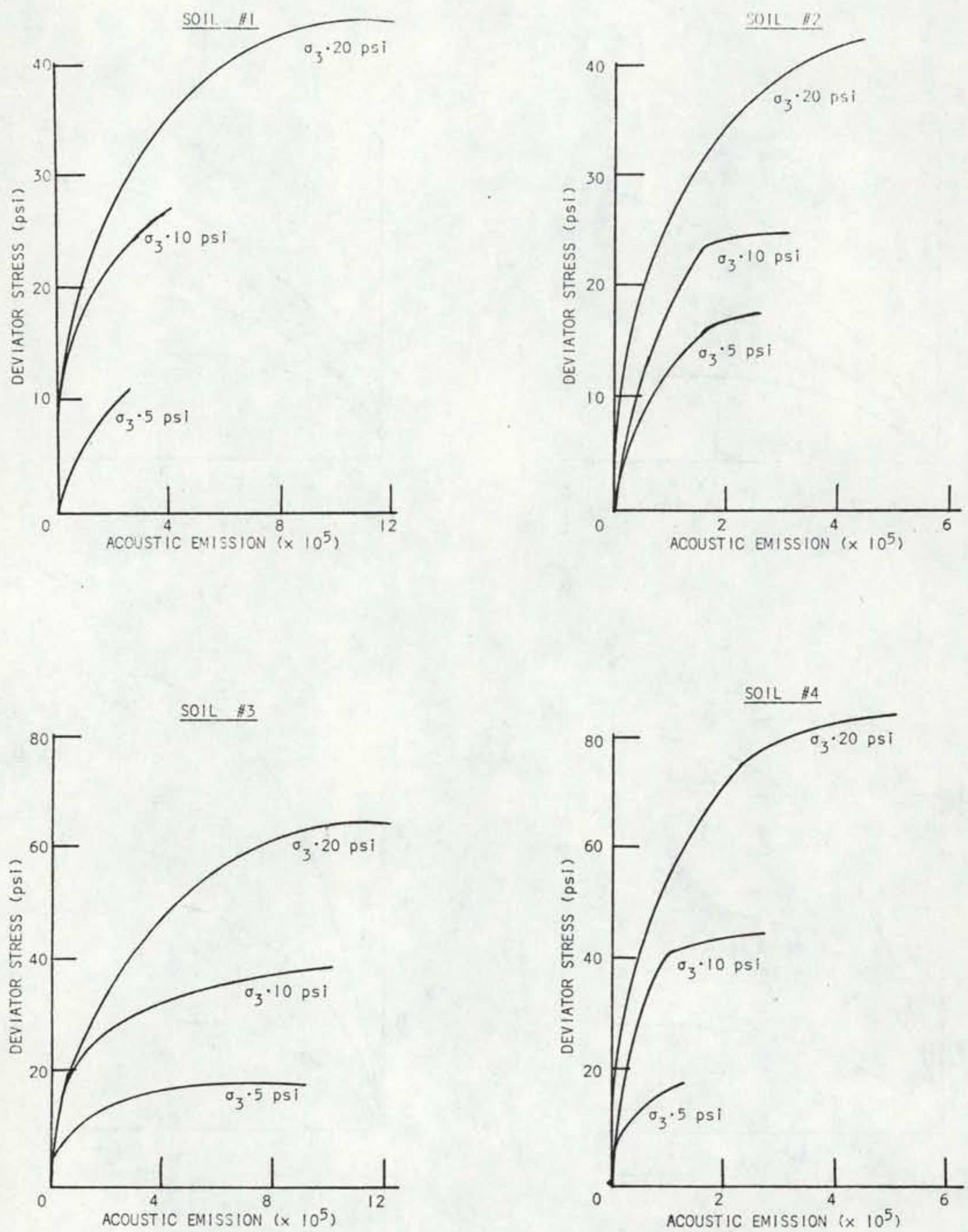


Figure IV-4. Triaxial Shear Test Results (Deviator Stress Versus Acoustic Emission in Units of 100,000 Counts) for Four Granular Soils Listed in Table IV-2. (Koerner, Lord, McCabe and Curran, 1976) testing than the other two samples. In general, the isostatic behavior of these angular soils, and particularly of soil No. 1, is also more emittive.

Coefficient of Uniformity. As coefficient of uniformity increases, so does the level of cumulative acoustic emissions. This is a strong conclusion for the triaxial test behavior, and is in almost perfect agreement with the isostatic test results. However, the more angular soils also happen to have the highest coefficient of uniformity. The actual cause of greater emissions may therefore be a combined effect.

Conclusions

From this study of acoustic emission response of granular soils, some of the fundamental properties of acoustic emissions have been evaluated. It was found that:

1. Velocity of emissions is from 400 fps to 800 fps (120 m/s-240 m/s) which is in the low range of wave velocities in porous media.
2. The predominate frequency of the emissions is from 500 Hz to 8 KHz, the actual value depending on the confining pressure.
3. Attenuation of acoustic emissions in granular soils is dependent strongly upon frequency in the region below 1 KHz - values range from 0.2 dB/ft to 40 dB/ft (0.007 dB/cm-1.3 dB/cm), while above 1 KHz the attenuation varies from 100 dB/ft to 300 dB/ft (2 dB/cm to 10 dB/cm).
4. Angular nonspherical particles produce greater acoustic emissions than soils consisting of rounded, spherical ones,

and furthermore, the time for emissions to cease (equilibrium reached) is greater for angular soils than for rounded soils.

5. Well-graded soils, with high coefficient of uniformity, produce large levels of acoustic emission counts.
6. Variation of particle size did not have a significant effect on the generated acoustic emission counts in the size range studied (Koerner, Lord, McCabe and Curran, 1977).

Acoustic Emission Response of Cohesive Soils

To investigate the response of cohesive soils, Koerner and others (1977) used four soil types in a triaxial test series. The types of soils and their properties are shown in table IV-3.

The triaxial test samples were 2.8 inches (70 mm) in diameter and 5.8 inches (150 mm) high. Except for the test sequence evaluating water content effects, all remolded samples were compacted at optimum water content in six layers to achieve a predetermined target density and void ratio. The accelerometer, including an attached 3 cm long, 5mm diameter waveguide, was suspended in the lower central portion of the mold while soil was tamped in place around it. The coaxial connecting cable was brought through the bottom platen and out of the triaxial chamber through one of the drainage ports. All triaxial tests were consolidated, drained and were conducted in creep mode in order to eliminate drive motor noise.

Each load increment was held until an equilibrium rate of deformation and acoustic emission was attained. The actual load increment time varied from 30 to 300 minutes depending on the magnitude of the load applied and soil type.

Table IV-3

Properties of Cohesive Soils Used. (Koerner, Lord, and McCabe, 1977)

Soil number[b] (1)	Soil description (2)	Type[c] (3)	W_l (4)[d]	W_p (5)[e]	Pl (6)[f]	G_s (7)[g]	W_{opt} (8)[h]	γ_{test} (9)[i]	C (10)[j]	ϕ (11)[k]
5	Clayey Silt	ML	47	37	10	2.62	23	-	41	29
6	Kaolinite Clay	MH	52	33	19	2.60	29	-	28	29
7	Silty Clay	CL	43	24	19	2.64	34	-	48	10
8	Bentonite Clay	CH	570	58	512	2.20	43	-	62	5

[a] All soils passed No. 200 sieve.

[b] Soils numbered 1 through 4 are found in Ref. 9.

[c] Unified soil classification system.

[d] Liquid limit, as a percentage.

[e] Plastic limit, as a percentage.

[f] Plasticity index (W_l - W_p), as a percentage.

[g] Specific gravity of solids, in grams per cubic centimeter.

[h] Optimum water content, as a percentage.

[i] Unit weight - varied slightly according to test series, see text.

[j] Cohesion, in kilonewtons per square meter.

[k] Friction angle, in degrees.

The effect of confining pressure on the acoustic emission behavior of cohesive soils was evaluated for two of the four soils in table IV-3. The clayey silt (No. 5) with a total unit weight of 1.69 g/cm^3 and void ratio of 0.95 and Kaolinitic clay (No. 6) with a total unit weight of 1.81 g/cm^3 and void ratio of 0.84 were each tested at confining pressures of 5 psi, 10 psi, and 30 psi (34 KN/m^2, 69 KN/m^2, 138 KN/m^2). The response curves are given in figures IV-5 and IV-6. The parallel behavior of stress/strain and stress/acoustic emission curves are noted easily.

The effect of water content on acoustic emission was conducted on the clayey silt (No. 5). The samples were compacted at different water contents and tested in unconfined compression. Figure IV-7 shows the results, which indicate a decrease in strength and acoustic emissions with increasing water content. The extremely low number of emissions recorded at higher water content approaches the liquid limit of the soil being monitored.

To investigate the relationship between plasticity index and acoustic emission, the four soils (PI of 11%, 19%, and 512%) were compacted to achieve a void ratio of 0.89 and tested in consolidated-drained triaxial creep at 5 psi (34 KN/m^2) confining pressure. The results are presented in figure IV-8. Cumulative acoustic emission counts are plotted versus percentage failure stress so that soils of different strength can be compared directly. The most emissive soil is the clayey silt (No. 5), which has the lowest PI, and correspondingly the greatest amount of larger silt-sized particles. The least emittive soil is Bentonite clay (No. 8), with extremely high PI and no silt-sized material. As shown in figure IV-8, the Kaolinite clay (No. 6) and silty clay (No. 7) have

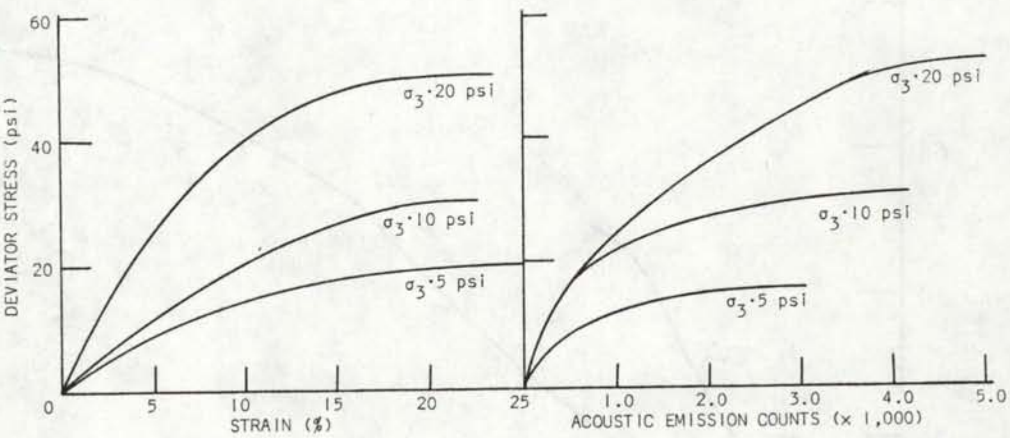


Figure IV-5. Triaxial Creep Response of Clayey Silt (Soil No. 5) at Varying Confinning Pressures. (Koerner, Lord, and McCabe, 1977)

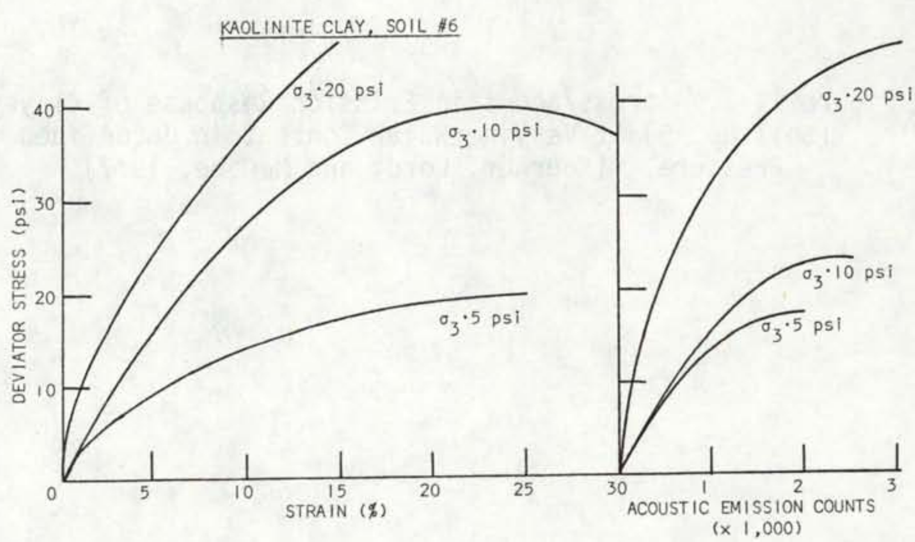


Figure IV-6. Triaxial Creep Response of Kaolinite Clay (Soil No. 6) at Varying Confining Pressures. (Koerner, Lord, and McCabe, 1977)

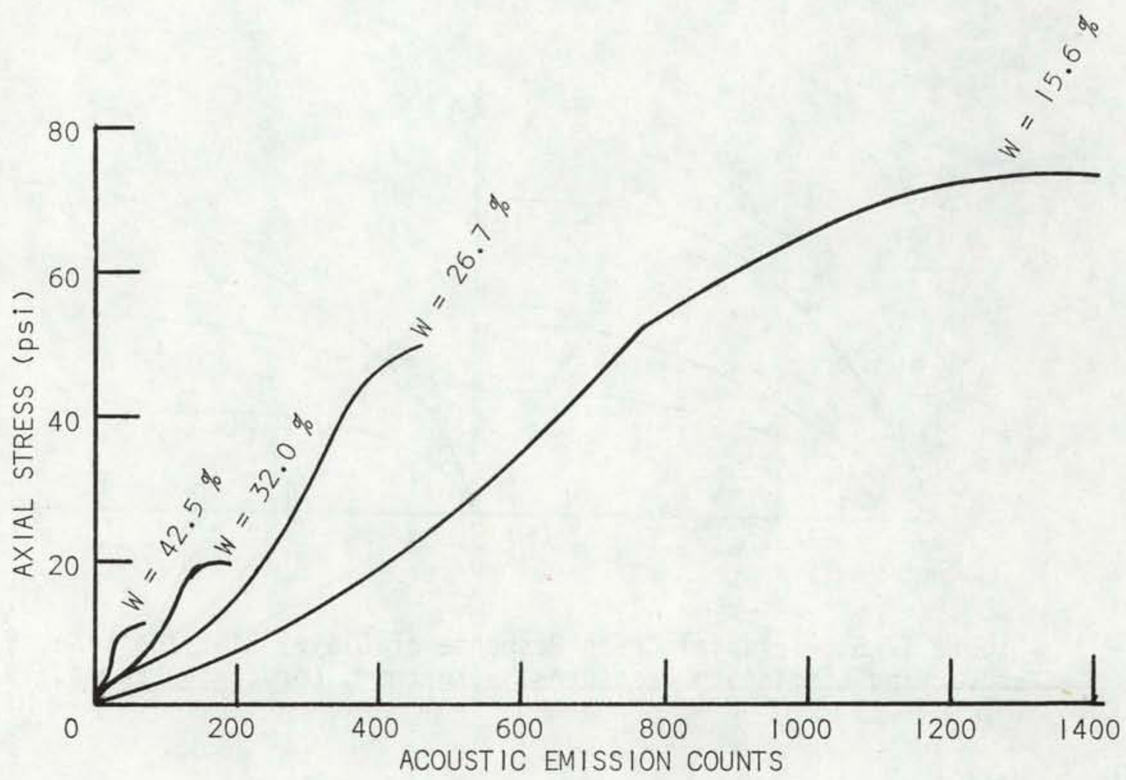


Figure IV-7. Stress/acoustic Emission Response of Clayey Silt (Soil No. 5) at Varying Water Content in Unconfined Pressure. (Koerner, Lord, and McCabe, 1977)

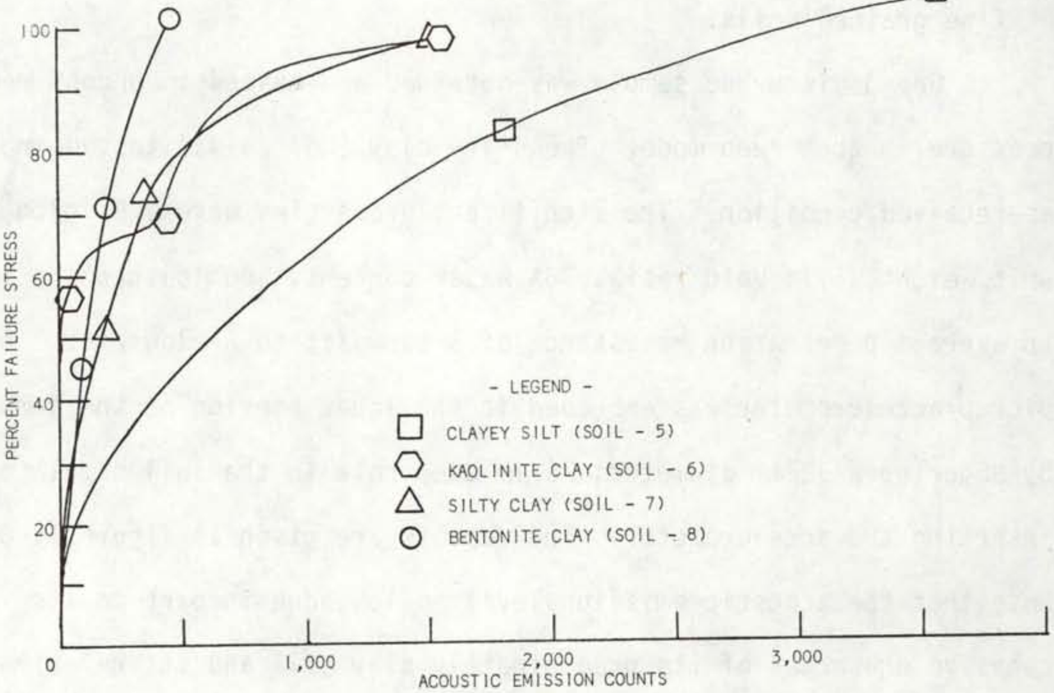


Figure IV-8. Stress/acoustic Emission Response of Four Cohesive Soils in Triaxial Creep Tests Showing Significance of Plasticity Index. (Koerner, Lord, and McCabe, 1977) approximately the same emission response and plasticity index. A strong correspondence exists between acoustic emission response and plasticity of fine grained soils.

One undisturbed sample was obtained and tested in unconfined pressure in the creep mode. The silty clay (No. 7) was tested in the as-received condition. The significant properties were 1.97 g/cm^3 total unit weight, 1.14 void ratio, 56% water content, 100% saturation, and an average penetration resistance of 3 blows/ft to 6 blows/ft. The pickup accelerometer was embedded in the lower portion of the sample by augering a 12 mm diameter, 25 mm deep hole in the soil sample and inserting the accelerometer. The results are given in figure IV-9. Note that the acoustic emission level is low, due in part to its cohesive character of its predominately clay soil and its relatively high water content. However, the acoustic emission response resembles closely the stress/strain behavior shown in the left-hand part of the figure.

Conclusions

From this study of the acoustic emission response of cohesive soils it was found that:

1. Increasing the confining pressure in consolidated-drained triaxial creep tests affects the shape of the stress versus acoustic emission counts curve in the same manner that it affects the stress/strain curve for cohesive soils. This parallel behavior indicates that acoustic emissions in soil is a deformation-related phenomenon.

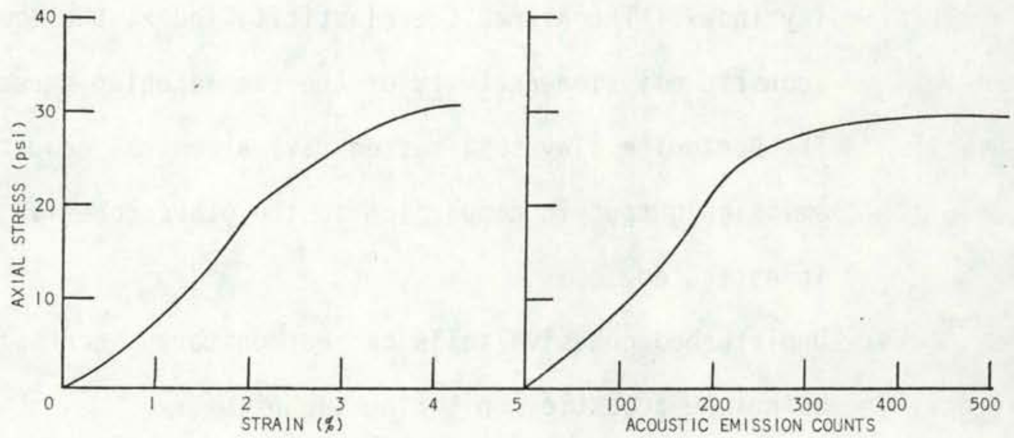


Figure IV-9. Unconfined Compression Test Results for Undisturbed Sample of Silty Clay (Soil No. 7) at 56% Water Content. (Koerner, Lord, and McCabe, 1977)

2. Acoustic emission activity in cohesive soils under stress decreases as water content increases. Cohesive soils at, or near, their liquid limits are not very emittive, within the limits of the amplifiers and accelerometers used in this study.
3. Acoustic emission activity is affected strongly by plasticity index. The higher the plasticity index, the lower the acoustic emission activity of the sample being stressed. The Bentonite clay soil tested gave a nominal acoustic emission output in comparison to the other cohesive soils investigated.
4. Undisturbed cohesive soils can be monitored successfully using the acoustic monitoring technique.
5. Regarding stress history of undistrubed cohesive soils, acoustic emission rates are relatively low at stresses below the preconsolidation pressure, and increase when the stress level exceeds the preconsolidation pressure. Unfortunately, the pressure at which the acoustic emission rate was observed to increase is, for the soil investigated here (a sandy silty clay known as a preconsolidated marl of low plasticity), considerably higher than the preconsolidation pressure as conveniently determined (Koerner, Lord, and McCabe, 1977).

Acoustic Emission and Slope Stability

The use of acoustic emission as a method of detecting slope stability is not new. Goodman and Blake (1966) studied rock noise in landslides and slope failures and Mearno and Hoover (1973) studied subaudible rock noise to determine slope stability. Koerner and Lord (1974) monitored three earth dams and these basic conclusions were drawn:

1. Earth dams that do not generate acoustic emissions are not deforming under their imposed loading system and are safe. Such dams are in a state of equilibrium and need not be inspected for considerable time or until a new loading condition is encountered.
2. Earth dams that generate acoustic emissions to a moderate degree are deforming slightly under their imposed loading condition and are to be considered marginal. Continued monitoring of such dams is required until such time that the emissions cease or increase to the following conditions.
3. Earth dams that generate large amounts of acoustic emission are deforming to high degree and are considered unstable. Immediate remedial measures are required.

Lord and Koerner (1975) continued monitoring the three earth dams above and began monitoring an iron ore stockpile in Maryland. The results of their research are reported in papers published in 1975 and 1976. The conclusions are essentially the same as reported in their 1974 paper.

Koerner, McCabe and Lord (1978) had monitored, or were in the process of monitoring eighteen field projects, most of which were small earth dams containing lagoons of hazardous materials. They concluded that deforming soils generate emissions and stable ones do not. Later in 1978, Koerner, Lord, and McCabe made the conclusions that:

1. Soil masses that do not generate acoustic emissions are probably not deforming and are therefore stable.

2. Soil masses that generate moderate levels of acoustic emissions, from 10 to 100 counts/minute for the soil types, equipment and sensitivities used in these tests, are deforming slightly and are to be considered marginally stable. Continued monitoring is required until such time that the emissions cease or increase to the following condition.
3. Soil masses that have high levels of acoustic emissions, from 100 to 500 counts/minute for the soil types, equipment and sensitivities used in this study, are deforming substantially and are to be considered unstable. Immediate remedial measures are required.
4. Soil masses that generate very high levels of acoustic emissions, greater than 500 counts/minute for the soil types, equipment and sensitivities used in this study, are undergoing large deformations and can be considered to be in a failure state. Emergency precautions to assure safety of nearby residents and their personal property should be immediately initiated.

Acoustic Emission and Seepage

Acoustic emission for detection of seepage is not widely used and is still in the research stage.

The two cases of acoustic emission for seepage detection discussed in Chapter III will not be discussed here except for figure IV-10 which shows the general agreement between seepage monitoring and acoustic emission activity.

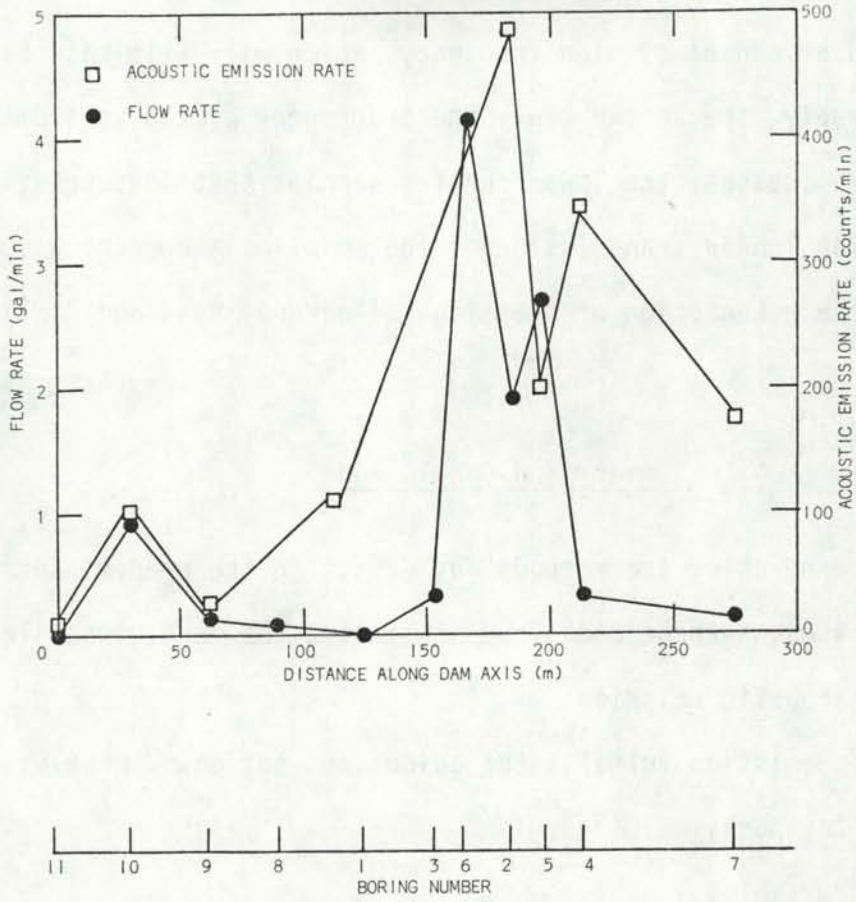


Figure IV-10. Results of Seepage Study Beneath Earth Dam Using Acoustic Emission and Standard Flow Measurement Techniques. (Koerner, Lord, and McCabe, 1977)

One parameter that must be considered when monitoring seepage with acoustic emission is the frequency range. If the pickup is made directly at the source of the emission, i.e., downhole or in water monitoring, the pickup transducer can be of high frequency, which will eliminate background noise. Conversely, the farther away the transducer pickup is from the source of the emissions, the lower the frequencies that must be utilized. This permits for longer transmission of the emission along the waveguide without complete attenuation of the signal (Koerner, Reif and Burlingame, 1979).

Summary and Conclusions

After researching the methods for detecting the predominant failure mechanisms, seepage and slope stability, the most versatile tool appears to be acoustic emission.

Acoustic emission fulfills the guidelines set down, namely:

1. Inexpensive,
2. Portable,
3. Ease of operation, and
4. Ease of interpretation.

Acoustic emission has proven itself as a tool for monitoring slope stability and appears very promising as a tool in seepage detection.

CHAPTER V

FIELD WORK

In order to become familiar with the acoustic emission monitoring system, field work was initiated.

The purpose of the field work was to:

1. Become familiar with setting up the equipment,
2. Become familiar with the different equipment settings to insure proper technique, and
3. Become familair with data interpolation.

Before beginning field work, a demonstration of the acoustic emission monitoring system was attended. The test was conducted at J. R. Simplot Company's Conda, Idaho, phosphate mining facility. The purpose of the field test was to show the capabilities of acoustic emission to monitor the stability of open pit mining slopes and waste pile or ore pile slopes. The test was conducted by Acoustic Emission Technology Co. Inc., Sacramento, California.

The instrumentation included:

1. <u>Piezoelectric transducer</u>. An AET Model AC30L piezoelectric crystal type cut to be resonant at 30 KHz.
2. <u>Preamp and filter</u>. The signal generated from excitation of the sensor was amplified to 40 dB by the AET Model 140A preamp. The preamp also acted as a filter with a bandpass of 4.5 KHz to 55 KHz.

3. Counter. The system counter was an AET Model 204 G (presently not commercially available) with a bandwidth of 100 HZ to 1 MHz.

4. Strip chart recorder. Data were recorded on the AET Model VP67235 strip chart recorder.

Waveguides consisted of ½-inch steel rebar driven into the stockpile and welded end to end to penetrate through to the foundation of the stockpile. Inverted cone shaped platforms were welded onto the top end of these waveguides.

To initiate failure successive cuts were made in a phosphate ore stockpile with a rubber-tired frontend loader.

The sequence of events were:

1. Take initial readings to determine background "noise" levels,
2. Make a cut, observe the rising acoustic emission count rate, let the slope come back to equilization, and
3. Make a second cut, observe the rising acoustic emission count until failure occurred.

As it turned out three cust were made before failure occurred.

Monitoring prior to any cuts indicated that settlement and other "noise" created 11 counts per minute. The first cut resulted in high counts. From a count rate of 231 events per minute immediately after cut cessation, the count rate decayed to 48 events per minute four minutes after the cut. They remained at or below 53 events per minute for the next 25 minutes. It is estimated that the AE level would have returned to the 11 events per minute rate an hour after the cut.

Cut 2 recorded an order of magnitude increase over cut 1. The initial recording after cut 1 is 231 events per minute whereas the initial recording after cut 2 was 4160 events per minute. Just after the 4160 events per minute were reached the same exponential decay was noted (as in cut 1) up to 17 minutes after the cut when AE increased dramatically. It is supposed that this is due to slope adjustments and that these irregularities would continue until the slope had restabilized itself.

After cut 3, the order of magnitude increase expected was delayed 5 minutes, but did occur. The irregularities in the decaying curve, cut 2 noted above, are more pronounced in cut 3. The cut 3 data show a sudden and continuous fluctuation indicating the sharply increased instability of the slope. A one-inch tension crack appeared at the head of the slope at this time. Adjustments along this crack produced the highest data rate of 14,000 events per minute (Leaird, 1979, unpublished).

Dams selected for field study are known to have seepage problems. The dams were referred by the Idaho Department of Water Resources, Dam Safety Division.

Beal Dam

Beal Dam is a small homogeneous earth dam, approximately 30 feet (9 m) in height, and constructed of weathered granite. Beal Dam is located in High Valley, Idaho, approximately 80 miles north and west of Boise.

During an inspection of Beal Dam by the Department of Water Resources, seepage was noted on the downstream toe, right side of dam just below mid height.

Testing of Beal Dam using accoustic emission was done on August 1, 1979. A visit was made on July 31, 1979, but testing was postponed due to a battery failure of the system counter.

The instruments used were:

1. Waveguides. Piezometer tubes installed in the seepage area were used as waveguides. The piezometer tubes were 3/4-inch steel pipe with screw caps. One of two steel inverted cones (used to seat the piezoelectric transducer-accelerometer) had a screw cap (to match those found at the site) welded on to it. This made for ease of operation because all that had to be done to monitor at each piezometer tube location was to screw on the inverted cone, make sure the accelerometer was attached correctly and monitor.
2. Sensor-accelerometer. The sensor was an AET Model AC20L piezoelectric crystal type cut to be resonant at 30 KHz.
3. Preamplifier. The preamplifier used was a 204A preamplifier with no additional filtering capability.
4. System counter. The system counter was an AET Model 204A modified to a 204B.

Equipment Setting

Gain: 76 dB (total system gain)

Threshold: Fixed .30 volts

Rate: Total count mode

Scale: 1

Wind Conditions

The wind blew throughout the monitoring period at an estimated speed of 5-7 knots.

Wind noise will affect AE monitoring making it necessary to protect the accelerometer, (i.e., cover with a box, wrap accelerometer with foam rubber, etc.).

Acoustic Emission Counts

Piezometer tube No.	Time, monitored (minutes)	Counts (total)
1	20	23
2	20	535
3	20	45
4	20	458
5	30	231

Results

The results from this monitoring period indicate a relatively stable condition. There is reason to believe that the wind had some part of the higher count number for piezometer tubes number 2 and 4 because these tubes extended 1 to 2 feet above the ground surface. The most ideal situation would have been for the accelerometer to be right at or just above the surface and covered.

AE counts of piezometer tubes 2 and 4 seem high but compared to a scale devised by Koerner and others (1978) the counts are actually minor. Koerner's scale states that soil masses which generate no acoustic emissions are probably not deforming, and are in a state of equilibrium. Soil masses generating moderate levels of acoustic emissions (10 counts/ minute to 100 counts/minute) are marginally stable and require continued monitoring. Soil masses generating high levels of acoustic emissions

(100 counts/minute to 500 counts/minute) are deforming actively and unstable, requiring immediate remedial action. Soil masses generating very high levels of acoustic emissions (greater than 500 counts/minute) are in or very near the failure state. Using Koerner's scale as a guideline, 535 counts in 29 minutes (approximately 26 counts/minute) would put the piezometer tube locations 2 and 4 in the marginally stable area.

Woodall Dam

The Woodall Dam is similar to the Beal Dam, made of the same type earth fill and approximately the same height. It is located about one mile east of Beal Dam.

The Woodall Dam has what appears to be seepage through the length of the toe. According to Mr. Beal, there is no reservoir seepage but that during construction there were springs at each end of the dam that were covered during construction.

The equipment used was the same as at Beal Dam except for the waveguides. There were no convenient piezometer tubes in the Woodall Dam as in the Beal Dam. Five-foot lengths of 3/4-inch rebar were used as waveguides, spaced five feet apart. At one end of the rebar were treads so that the same inverted cone used at Beal Dam could also be used on Woodall Dam. Nine waveguides were installed at the toe of the dam in the "seepage". The toe was saturated, some sloughing had occurred, and a standing clear water pool was present at a point just below the toe.

Equipment Setting

All settings were the same except the system gain was changed to 86 dB. The higher the dB the more sensitive the instrument, which also results in higher stray "noise", (i.e., wind, traffic, etc.).

Wind Conditions

The wind conditions were about the same as the previous day at Beal Dam. The accelerometer was covered with a box to prevent interference but it is believed that the wind was still a factor.

Acoustic Emission Counts

Station	Time (minutes)	Counts (total)
1	20	903
2	20	503
3	20	1112
4	20	190
5	20	1496
6	20	1351
7	20	492
8	20	298
9	20	358

Results

As with Beal Dam the results appear to be high but if the highest total count, 1496, is converted into an average counts/minute, that is, 75 counts/minute, the Woodall Dam also falls into the marginally stable area according to Koerner's scale.

Winchester Dam

Winchester Dam is located just outside the City of Winchester, Idaho, and is owned by the Idaho Department of Fish and Game, Boise, Idaho.

Winchester is an earthfill dam with concrete core. It is composed mostly of sandy clay and gravelly silt.

The Winchester Dam has leaked since the first filling. The dam has changed in width over the years more than likely to accommodate U. S. 95, business route, which runs over part of the dam (see figure V-1).

Equipment Setting

Gain: 76 dB

Threshold: Fixed .40

Rate: 60 seconds

Wind Condition

Light wind.

Strip Chart Recorder

The AET Model VP6723S strip chart recorder was used to record the data.

Traffic Conditions

Traffic along U. S. 95 business route was heavy at this time and certain vehicles were noted to have greater effect than others.

Results

Four stations were monitored at the Winchester Dam site.

Station one was located at drill hole 3 on the south side of U. S. 95 business route.

Station two was located at drill hole 2 on the north side of U. S. 95 business route a few feet west of the spillway.

Station three was located at drill hole 7 just east of drill hole 2. Station three was abandoned due to the inability of the instrument to monitor at this site. It is believed that the drill stem was oscillating

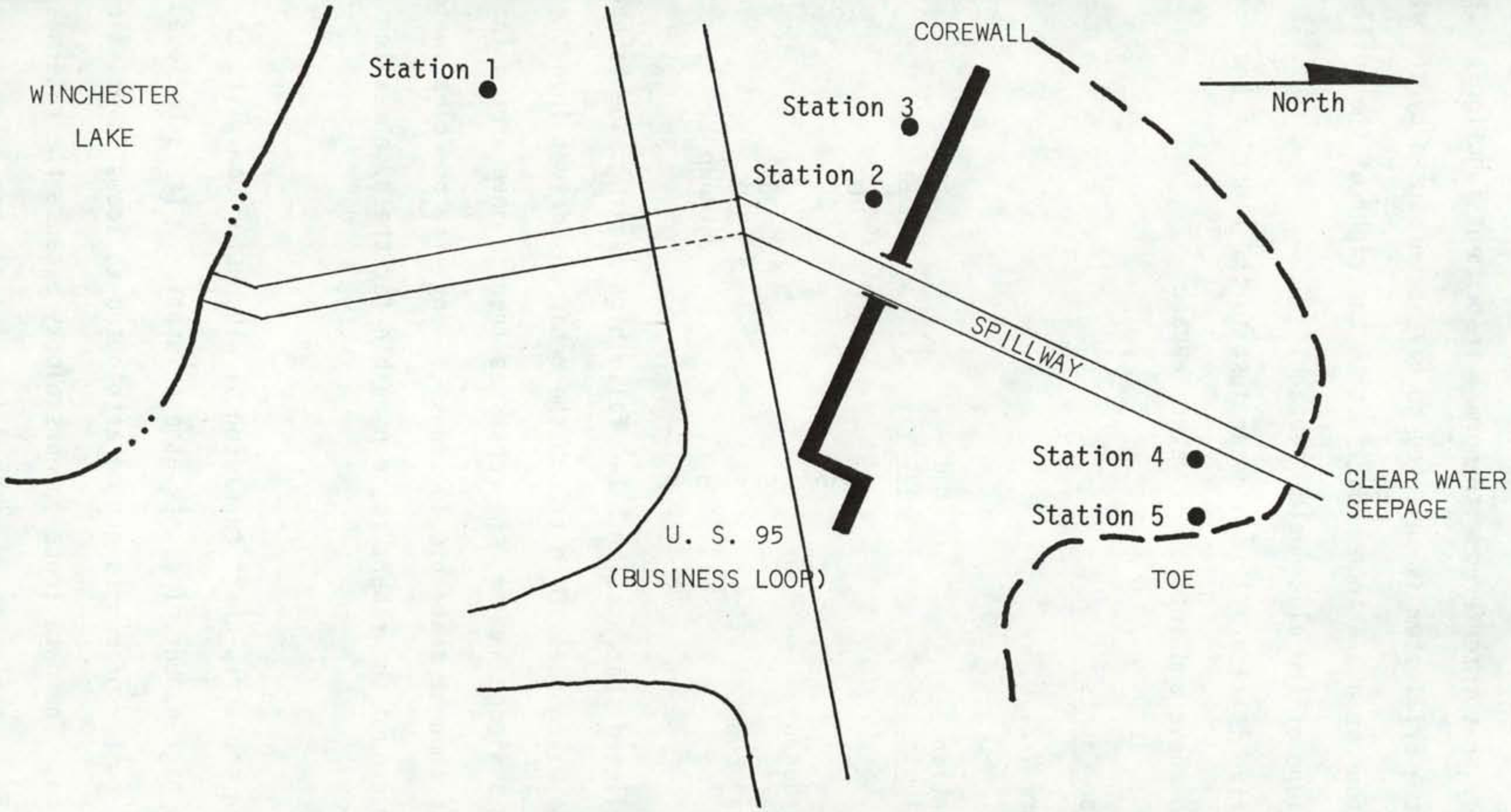


Figure V-1. Winchester Dam Showing Stations Monitored at some frequency that interfered with normal instrument functions. This oscillation of the drill stem is believed to have been caused by the wind.

Station four is a 3/4 inch rebar just to the right of the spillway, a few feet up slope of the clear water seepage.

Station five is also 3/4 inch rebar just to the right of the syphon pipe also above and into clear water seepage.

Acoustic Emission Counts

See figure V-2.

Station rate 1 minute	Time minutes	Counts
1	5	0
2	5	60
3 abandoned		
4	5	30
4 total counts	2½	300
5	5	unknown

Note: It is believed that station 5 had false high readings due to the looseness of the waveguide. Even though the rebar was driven into the embankment with a sledge hammer, the rebar was free to move around in its hole. It is thought that this looseness, along with the clear water seepage flowing around the rebar, is responsible for the high readings.

Results

Observing the strip-chart recording of Winchester Dam gives an impression of a high number of counts at each station. It is believed that these high number of counts can be attributed to acoustic emission, traffic, and wind. The data trace appears blocky because the rate was one minute and each station was monitored for five minutes. The blockiness

Figure V-2. Strip Chart Recording Showing Acoustic Emission Data.

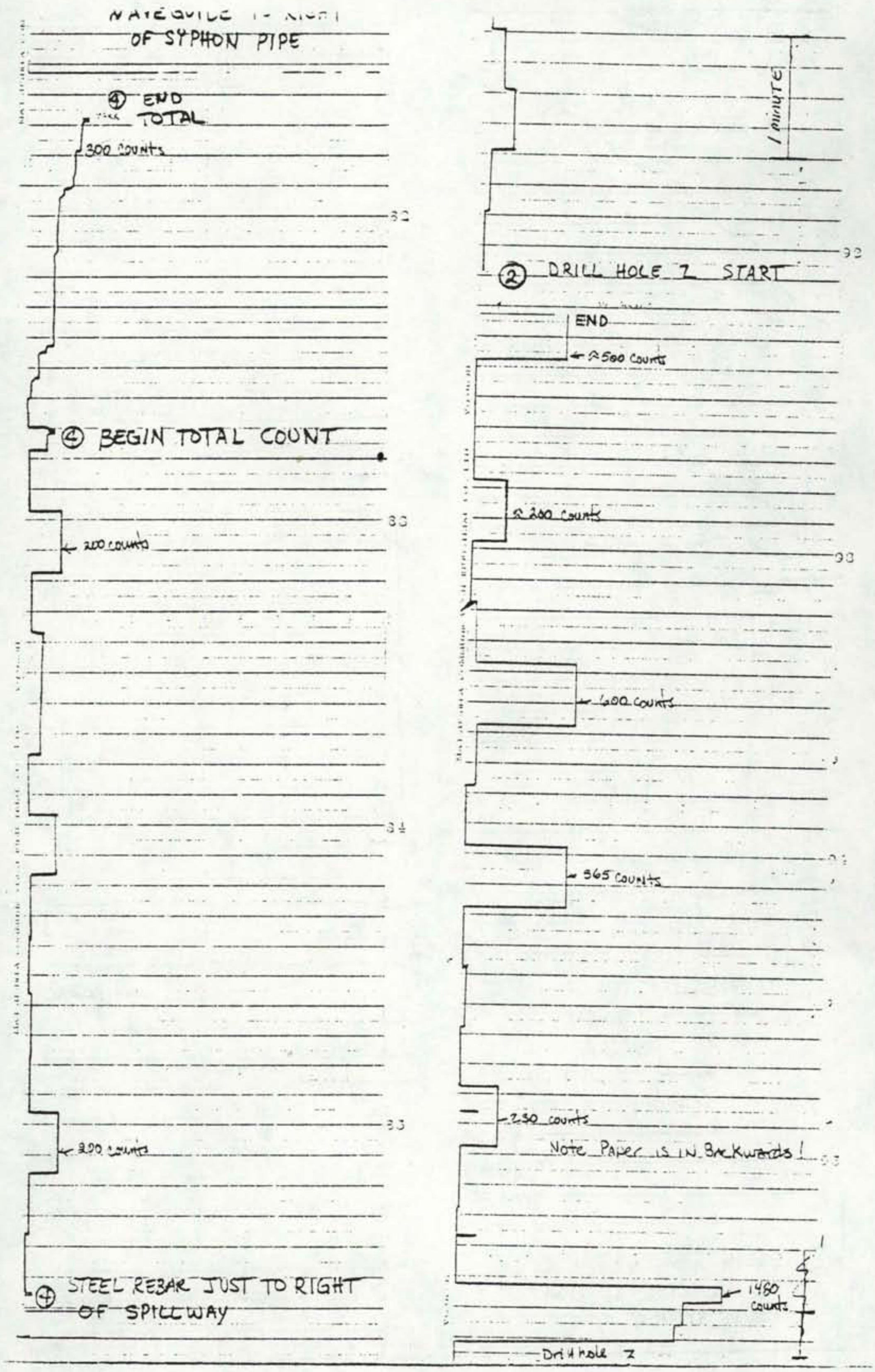
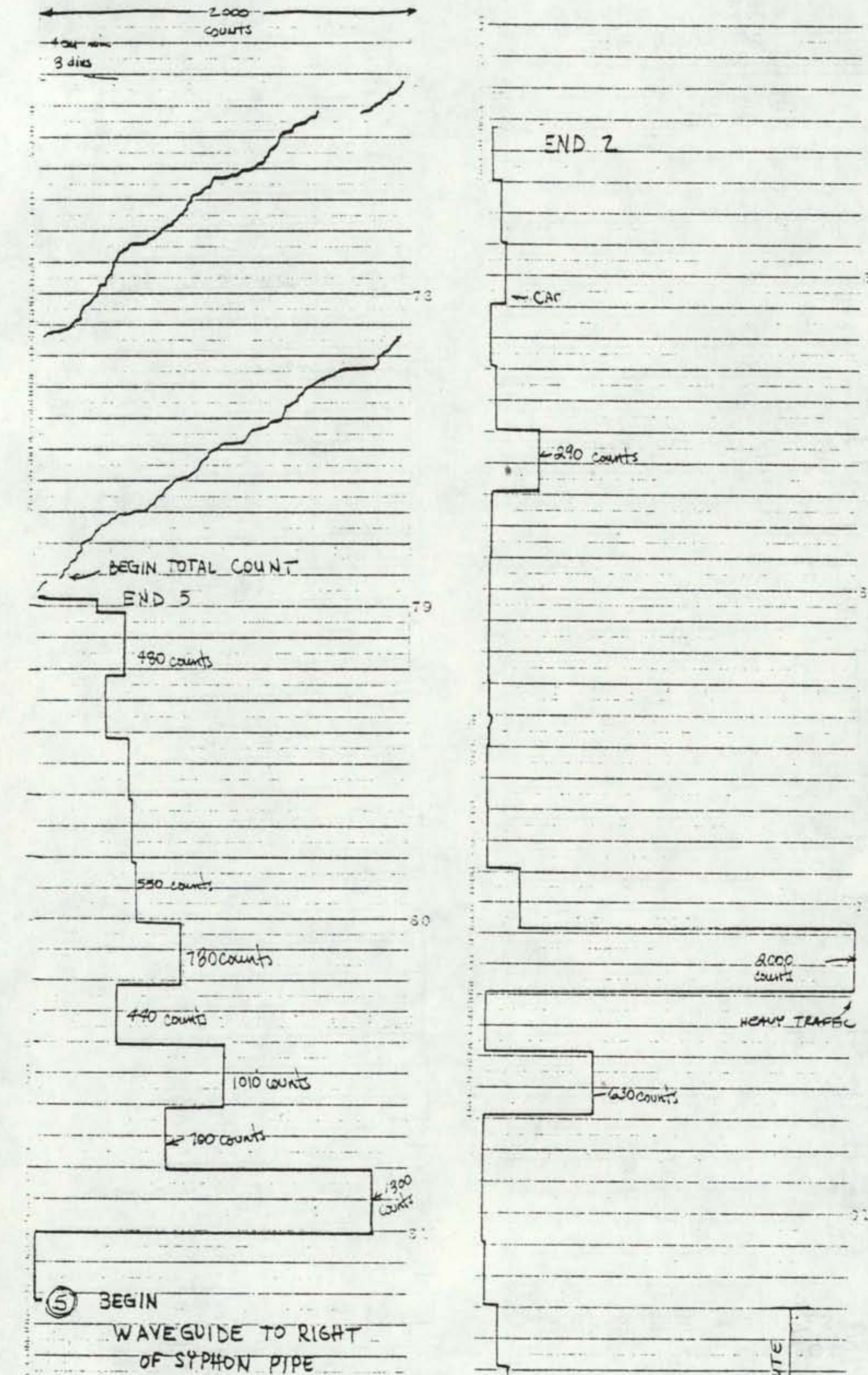


Figure V-2. Continued.



is due to the fact that at the end of each one minute period the machine automatically clears itself and begins counting. As the counts go up the pen moves up and as the peak number of counts is reached the data curve levels off. Although the peak level is reached the paper drive is still turning the paper giving it the flat-topped peak, then as the one minute cycle is over the instrument again clears itself and the pen again attempts to return to the zero line. Sometimes, counts are coming into the system before the pen can reach the zero line and the pen will begin another blocky trace. The blocky appearances of the trace may at first glance appear to be high peaks but in actuality the block traces show the structure to be in a somewhat stable condition. On Koerner's scale the dam is marginally stable, but would require more monitoring for safety.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The Acoustic Emission Monitoring System appears to meet the equipment guidelines set down for this project, which are:

1. Inexpensive,
2. Portable,
3. Ease of operation, and
4. Easily interpreted and accurate data.

As stated previously, the Acoustic Emission Monitoring System is an excellent tool for monitoring slope stability and appears to be promising for seepage detection. However, more work needs to be done in this area.

Acoustic Emission should be considered a part of a total dam inspection program. Acoustic Emission data, although interpreted easily, should not be considered the last word on dam stability but used as a guideline. The conclusions drawn by Koerner, Lord and McCabe (1978) summarized here:

> Soil masses that generate moderate levels of acoustic emissions from 10 to 100 counts/minute are deforming slightly and are to be considered marginally stable. Soil masses that have high levels of acoustic emissions, from 100 to 500 counts/minute are deforming substantially and are to be considered unstable. Soil masses that generate very high levels of acoustic emissions greater than 500 counts/minute are undergoing large deformations and can be considered in a failure state.

These conclusions should be considered guidelines used in conjuction with the experience of the operator/inspector, age of the dam, wind conditions, outside noise and past history of the dam.

Recommendations

Monitoring Program

There are two ways to appraoch a monitoring program using the Acoustic Emission System.

1. Monitor known trouble areas in older dams; and
2. Monitor by use of a grid if no known trouble areas are known or for a new embankment dam.

Length of Waveguides

Waveguides should be long enough to efficiently monitor an area. If deemed necessary by the operator, waveguides should extend through the embankment and into the foundation or abutment.

Waveguide Placement

Waveguides should be placed in suspect areas, foundations, toes, crests and abutments if deemed necessary by the operator/inspector. Any metal aperture that is partially or totally buried in the embankment can be monitored acoustically (e.g. piezometer tubes).

Length of Monitoring Time

Each station, or waveguide, should be monitored for enough time for background emissions to be gathered about that station (e.g. 20 minutes). The operator/inspector may then, upon future monitoring, have enough information that some conclusion as to the stability of the structure may be drawn under the presently imposed loading conditions. As loading conditions change, counts may change and so may stability.

Interpretation of Data

The data from each acoustic emission monitoring period should be plotted in some convenient manner and the trend of the plot over time serve as a guide to the stability of the structure. Increases in counts/minute would indicate deformations are taking place and perhaps warrant more frequent monitoring.

General Information

Thorough record keeping is a must. At each station (waveguide) the following information should be noted:

Location. Mapped or verbal description of location of waveguide.

Weather Conditions. Strong winds may affect counts.

Equipment Settings. Notes should be made on:

1. Count/event mode used,
2. Rate,
3. Scale,
4. Gain, and
5. Threshold (fixed or floating; threshold setting if fixed).

Comments. Any comments by the operator/inspector that he or she feels may be relevant to the monitoring of that station.

REFERENCES

Ambler, J. N., 1912, "The Failure and Repair of the Winston (N. C.) Water Works Dam", in Engineering News, v. 67, no. 15, April 11, 1912, p. 667.

American Society of Civil Engineers, Transactions, 1891, "Report of the Committee on the Cause of the Failure of the South Fork Dam", v. 24, no. 477, June, 1891, p. 431.

American Society of Civil Engineers, 1972, "Performance of Earth and Earth-Supported Structures", v. 1, pt. 1, Victor Braunig Dam, San Antonio, Texas.

Anderson, G. G., 1909, "The Failure of the Bluewater Dam in New Mexico", Engineering News, v. 62, no. 14, Sept. 30, 1909, p. 353.

Baker, C. W., 1918, "Otay Rock-fill Dam Failure", in Engineering News, v. 75, no. 5, p. 236.

Bassel, B. M., 1904, "Earth Dams - A Study", Engineering News Publishing Co., New York.

Bennett, R., 1916, "More Proof of Need for Supervision of Dams", Engineering Record, v. 74, no. 12, Sept. 16, 1916, p. 357.

Block, E. R., 1925, "Partial Failure of Earth Dam at Horton Kansas", in Engineering News-Record, v. 95, no. 2, July 9, 1925, p. 58.

Brownell, E. H., 1901, "The Failure of Two Earth Dams at Providence, R. I.", in Engineering News, v. 45, no. 12, March 21, 1901, p. 212.

Cadman, J. D. and Goodman, R. E., 1967, "Landslide Noise", Science, v. 15, December 1, p. 1182-1184.

Case, C. B., 1914, "Hebron Earth Dam Washed Out", Engineering Record, v. 69, no. 22, May 30, 1914, p. 629.

Clevenger, W. A.*, <u>When is Foundation Seepage Unsafe?</u>, Source and year unknown. *Chairman of the Board, Woodward-Clyde Consultants, San Francisco, California.

Colorado State Engineer, 1910, Annual Report, Denver, Colorado.

Comtroller General, 1977, Report to the Congress: Slow Progress in Developing and Implementing a National Dam Safety Program, Department of the Army, Corps of Engineers, (Civil Functions, CED-77-94, June 29.

Cook, J. C. 1974, "Status of Ground Probing Radar and Some Recent Experience", Subsurface Exploration for Heavy Construction, Engineering Foundation Conference, ASCE, 1974, pp. 172-194.

Corps of Engineers, 1949, "Slope Protection for Earth Dams", Dept. of the Army, Waterways Experiment Station, March.

Coxon, R., and Cook, D., 1976, "Some Simple Approaches to Leakage Detection in Dams", The Transactions of the 12th International Congress on Large Dams, v. 2, International Commission on Large Dams, Paris, France, pp. 527-540.

Critchlow, H. T., 1928, "Failure of Non-State-Inspected Dam Under Construction", in Engineering News-Record, v. 100, no. 3, Jan. 19, 1928, p. 116.

Dobrin, M. B., 1976, Introduction to Geophysical Prospecting, 3rd Edition, McGraw-Hill Book Co., Inc., New York, N. Y.

Dolphin, L. T., Bollen, R. L., and Oetzel, G. N., 1974, "An Underground Electromagnetic Sounder Experiment", Geophysics, v. 39, no. 1, February, pp. 49-55.

Ellerbruch, D. A., and Adams, J. W., 1974, "Microwave Measurement of Coal Layer Thickness", Final Report NBSIR 74-387, National Bureau of Standards, Boulder, Colorado, September.

Ellis, T. G., Greene, D. M., and Wilson, W. W., 1876, "On the Failure of Worcester Dam", in American Society of Civil Engineers, Transactions, v. 5, no. CXXV, June 15, 1876, p. 244.

El-Said, M. A. H., 1956, "Geophysical Prospecting of Underground Water in the Desert by Means of Electromagnetic Interference Fringes", Proceedings of the IRE, January, pp. 24-50.

Emerson, F. B., 1925, "180-ft. Dam Formed by Landslide in Gros Ventre Canyon", Engineering News-Record, v. 95, no. 12, Sept. 17, 1925, p. 467.

Engineering News, 1887, "The Bursting of a Dam at Ingersoll, Canada", in v. 17-18, April 9, 1887, p. 233.

Engineering News, 1892, Article in v. 27, June 16, 1892, p. 608.

Engineering News, 1894, "A Narrow Escape for an Earth Dam, Altoona, Pa., Water Works", in v. 31, June 7, 1894, p. 473.

Engineering News, 1894, Article in v. 32, no. 17, Oct. 25, 1894, p. 333.

Engineering News, 1901, Article in v. 45, no. 17, May 2, 1901, p. 313.

Engineering News, 1901, Article in v. 45, no. 21, May 30, 1901, p. 385.

Engineering News, 1902, Article in "Engineering News of the Week", v. 47, no. 23, June 5, 1902, p. 448.

Engineering News, 1092, "The Failure of the Waterworks Dam at Utica, N. Y.", v. 48, no. 15, Oct. 9, 1902, p. 290.

Engineering News, 1903, "Two Dams Fail at Shelton, Conn.", v. 49, no. 9, Feb. 26, 1903, p. 185.

Engineering News, 1905, Article in v. 53, no. 26, June 29, 1905, p. 693.

Engineering News, 1912, "A Newspaper Story and an Engineer's Account of a Dam Failure", v. 67, no. 19, May 9, 1912, p. 861.

Engineering News, 1912, Article in v. 67, no. 23, June 6, 1912, p. 1102.

Engineering News, 1914, "Reservoir Embankment Failure; Turlock Irrigation District, California", v. 72, no. 2, July 9, 1914, p. 106.

Engineering News, 1914, "Partial Failure of Standley Lake Dam, Near Denver, Colorado", v. 72, no. 6, Aug. 6, 1914, p. 326.

Engineering News, 1915, "Failure of Lyman Dam, Arizona", v. 73, no. 16, April 22, 1915, p. 794.

Engineering News, 1916, "Overtopped Earth Dam Fails", v. 76, no. 5, Aug. 3, 1916, p. 232.

Engineering News, 1916, "Lake Toxaway Earth Dam Fails", v. 76, no. 7, Aug. 17, 1916, p. 331.

Engineering News, 1916, "Dam Fails for Third Time at Schenectady", v. 76, no. 17, Oct. 26, 1916, p. 817.

Engineering News, 1917, "Earth-Dam Failure in Bohemia Due to Bad Design".

Engineering News-Record, 1917, "Ouachita River Dam Fails", v. 79, no. 11, Sept. 13, 1917, p. 524.

Engineering News-Record, 1918, "Two Large Earth Dams Have Remarkable Similarity", v. 80, no. 15, April 11, 1918, p. 704.

Engineering News-Record, 1923, "Failure of Aphishapa Earth Dam in Southern Colorado", v. 91, no. 9, p. 357.

Engineering News-Record, 1925, "Condemned Dam in Oregon Fails When Over-topped", v. 94, no. 8, Feb. 19, 1925, p. 332.

Engineering News-Record, 1925, "Undermining Causes Failure of French Landing Dam", v. 94, no. 18, April 30, 1925, p. 735.

Engineering News-Record, 1926, "Flood Damage to Puddingstone Earth-Fill Dam", v. 96, no. 22, June 3, 1926, p. 913.

Engineering News-Record, 1926, "Earth Dam Fails by Shrinkage Under Frost Crust", v. 96, no. 23, June 19, 1926, p. 942.

Engineering News-Record, 1927, "Earth Dam Near Hemet California, Destroyed by Flood", v. 98, no. 10, March 10, 1927, p. 423.

Engineering News-Record, 1927, "Earth Dam on Maquoketa River in Iowa Washed Out", v. 98, no. 24, June 16, 1927, p. 1000.

Engineering News-Record, 1927, "Brief Notes", v. 99, no. 5, Aug. 4, 1927, p. 196.

Engineering News-Record, 1928, "Notable Dam Failures of the Past", v. 100, no. 12, March 22, 1928, p. 472.

Engineering News-Record, 1928, "High Earth Dam Threatened by Washout", v. 100, no. 19, May 10, 1928, p. 750.

Engineering News-Record, 1928, "Leak Endangers Earth Dam in Utah", v. 100, no. 21, May 24, 1928, p. 826.

Engineering News-Record, 1928, "Pleasant Valley Dam Leak is Believed Now to be Under Control", v. 100, no. 22, May 31, 1928, p. 872.

Engineering News-Record, 1929, "Reconstruction of Lafayette Dam Advised", v. 102, no. 2, Jan. 31, 1929, p. 190.

Engineering News-Record, 1929, "Balsam Dam Failure Caused by Erosion of Rip-rap", v. 102, p. 885.

Engineering News-Record, 1929, "Rockfill Dam on Virgin River Nev., Washed Out During Construction", v. 103, no. 14, Oct. 23, 1929, p. 526.

Engineering News-Record, 1930, "Hydraulic-Fill Dam of Fine Volcanic Ash Fails Disasterously", v. 104, no. 21, May 22, 1930, p. 869.

Engineering News-Record, 1930, "Two Michigan Earth Dams Fail", v. 105, no. 2, July 10, 1930, p. 71.

Engineering News-Record, 1930, "Corpus Christi Dam Failure", v. 105, no. 25, Dec. 18, 1930, p. 974.

Engineering News-Record, 1936, "Elk City Earth Dam Fails During Deluge", v. 116, May 7, 1936, p. 678.

Engineering News-Record, 1936, "Overtopping Causes Failure of Earth Dam", v. 116, June 11, 1936, p. 850.

Engineering News-Record, 1937, "Rain Wrecks Spillway of Louisiana Dam", v. 119, no. 19, Nov. 4, 1937, p. 765.

Engineering News-Record, 1938, "Foundation Caused Dam Failure", v. 120, no. 8, Feb. 24, 1938, p. 281.

Engineering News-Record, 1938, "Irrigation Dam Fails in Washington", v. 120, April 28, p. 602.

Engineering News-Record, 1938, "Irrigation Dam Fails in Washington", v. 120, no. 17, April, p. 603.

Engineering News-Record, 1938, "Small Reservoir Dam Fails in Montana", v. 121, no. 5, Aug. 4, 1938, p. 126.

Engineering News-Record, 1938, "Small Ohio Dam Carried Out", v. 121, no. 12, Sept. 22, 1938, p. 341.

Engineering News-Record, 1939, "Earth Dam Troubles", v. 122, no. 4, January 26, vol. p. 144.

Engineering News-Record, 1939, "Earth Dam Fails in Montana" and "Montana Dam Break Being Investigated" in v. 122, no. 12 and no. 13, March 23, and March 30, 1939, vol. p. 407 and 423.

Engineering News-Record, 1939, "Arkansas Earth Dam Fails During Construction", v. 122, no. 17, April 27, 1939, vol. p. 535.

Engineering News-Record, 1939, "Large Slide in Fort Peck Dam Caused by Foundation Failure", v. 122, no. 19, May 11, 1939, vol. p. 647.

Engineering News-Record, 1940, "Heavy Damage in Texas Follows Four Days of Rains", v. 125, Nov. 28, 1940, vol. p. 715.

Engineering News-Record, 1948, "To Rebuild Wrecked Dam in Fraser Valley, B. C.", v. 140, no. 7, Feb. 12, 1948, p. 229.

Engineering News-Record, 1952, "Flood Tears Gap in Dam", v. 148, no. 17, April 24, p. 24.

Engineering News-Record, 1964, "A Dam to Watch", v. 173, no. 24, Dec. 10, 1964, p. 33.

Engineering News-Record, 1975, "Failure Threat at Leaky Dam Causes Drawdown of Lake", v. 195, no. 12, Sept. 18, p. 51.

Engineering News-Record, 1976, "Teton Panel Suspects Grout Curtin", v. 197, no. 3, July 15, p. 11.

Engineering News-Record, 1976, "New German Dike Breaks, Flooding Wide Area", v. 197, no. 5, July 29, p. 13.

Engineering News-Record, 1976, "Floods Wreck 442-ft-high Pakastani Earthfill", v. 197, no. 12, Sept. 16, p. 13.

Engineering News-Record, 1976, "Mexican Levee Collapse Kills at Least 40", v. 197, no. 16, October 14, p. 13.

Engineering News-Record, 1977, "Johnstown is Inundated Again by a Record, 500-year Flood", v. 199, no. 4, July 28, p. 9.

Engineering News-Record, 1977, "German Earthfill Fails; No Casualties", v. 199, no. 9, Sept. 1, p. 13.

Engineering News-Record, 1977, "Dam Collapses in Southeast Spur Federal Inspection Plan", v. 199, no. 19, Nov. 10, 1977, p. 13.

Engineering Record, 1905, "The Failure of an Earth Dam", v. 52, no. 7, Aug. 12, 1905, p. 189.

Engineering Record, 1911, "Reparing the Earth Dam of the Julesburg Reservoir", v. 63, no. 17, April 29, 1911, p. 467.

Engineering Record, 1912, "Seepage Through and Earth Dam", v. 66, no. 8, Aug. 24, 1912, p. 223.

Fedler, L. B., 1909, "What happened to the Trout Lake Dam", in a letter to the editor in v. 60, no. 17, Oct. 23, 1909, p. 476.

Fellows, A. L., 1917, "One Engineer's Opinion as to Why the Standley Lake Dam Sloughed", in Engineering News-Record, v. 78, no. 9, May 31, 1917, p. 445.

Field, J. E., 1923, "Failure in Aphishapa Earth Dam in Colorado - II", Engineering News-Record, v. 91, no. 9, p. 418.

Gelder, R. W., 1917, "The Case of the Standly Lake Dam - A Reply", in Engineering News-Record, v. 78, no. 12, June 21, 1917, p. 612.

Golze, A. R., 1977, "Handbook of Dam Engineering", Editor, Van Nostrand-Reinhold Co.

Goodman, R. E., and Blake, W., 1966, "Rock Noise in Landslides and Slope Failure", Highway Research Record, No. 119, Highway Research Board, Washington, D. C., p. 50-60.

Grant, and Winefordner, 1966, "Grouting a Dam Cutoff in Cavernous Limestone", ASCE Proceedings, Journal of Construction Division, September.

Green, A. T., 1969, "Detection of Incipient Failures in Pressure Vessels by Stress-wave Emissions", Nuclear Safety 10, 4-18.

Green, P. E., 1936, "Solving a Dam Break by Deduction", in Engineering News-Record, v. 116, April 16, 1936, p. 556.

Hanna, F. W., and Kennedy, R. C., 1938, The Design of Dams, McGraw-Hill Book Co., Inc., New York (Tables XXXIX and IL, p. 396 and 398).

Hayes, J. E., 1917, "The Case of the Standley Lake Dam", in Engineering News-Record, v. 78, no. 9, Mar 31, 1917, p. 440.

Hagen, A. and Metcalf, L., 1918, "Middle Section of Upstream Side of Calveras Dam Slips into Reservoir", in Engineering News-Record, v. 80, no. 14, April 4, 1918, no. 679.

Hinderlider, M. C., 1914, "Failure of Horse Creek Earth Dam", in Engineering News, v. 71, no. 16, April 16, 1914, p. 828.

Henry, S. T., 1929, "Reconstruction of Table Rock Core Earth Dam After Dangerous Washout and Slump", in Engineering News-Record, v. 103, no. 24, Dec. 12, 1929, p. 934.

Hill, W. R., 1920, "A Classified Review of Dam and Reservoir Failures in the United States", in Engineering News, v. 47, no. 25, June 19, 1902, p. 506.

Hinderlider, M. C., 1915, "Rock-fill Dam with some Extraordinary Foundation Problems", in Engineering News, v. 73, no. 14, April 8, 1915, p. 660.

Holmes, W. H., 1927, "Earth-Filled Dam Leakage Stopped by Grouting with Cement", in Engineering News-Record, v. 98, no. 22, June 2, 1927, p. 900.

Horn, H. M., and Deere, D. U., 1962, "Frictional Characteristics of Minerals", Geotechnique, London, England, v. 12, 1962, p. 319-335.

Jenson, J., 1914, Break in Hatchtown Dam", Engineering Record, v. 69, no. 26, June 27, 1914, p. 727.

Jorgensen, L., 1920, "The Record of 100 Dam Failures", in Journal of Electricity, v. 44, no. 6, March 15, 1920, p. 274.

Justin, J. D., 1932, Earth Dam Projects, John Wiley and Sons, Inc., London.

Justin, J. D., 1939, "More Light on Ft. Peck", in Engineering News-Record, v. 122, Annual Water Works Number, June 8, vol. p. 753.

Kaiser, J., 1953, "Erkenntnisse und Folgerungen aus der Messung von Gerauschen beizugbeanspruchung von Metallischen Werkstoffen", Arkiv fuer das Eisenhiittenwesen, v. 25, 43.

Keys, W. S. and MacCary, 1971, Application of Borehole Geophysics to Water-Resources Investigations; Techniques of Water Resources Investigations of the U. S. Geological Survey, Book 2, Collection of Environmental Data, U. S. G. P. O., Washington, D. C.

Kleinschmidt, H. S., 1917, "Defiance of Statute and Engineering Law Wrecked Mamooth Dam", Engineering News-Record, v. 79, no. 2, p. 52.

Koerner, R. M., and Lord, A. E., 1974, "Earth Dam Warning System to Prevent Hazardous Material Spills", AIChE/EPA Conference on Control of Hazardous Materials Spills, San Francisco, California, August.

Koerner, R. M., and Lord, A. E., 1976, "Acoustic Emission Monitoring of Earth Dam Stability", Water Power and Dam Construction, London, England, p. 45-49.

Koerner, R. M., Lord, A. E., and McCabe, W. M., 1977, "Acoustic Emission Behavior of Cohesive Soils", Journal of the Geotechnical Engineering Division, ASCE, v. 103, no. GT8, August.

Koerner, R. M., Lord, A. E., and Mcabe, W. M., 1978, "Advances in Acoustic Emission Monitoring", Water Power and Dam Construction, v. 30, no. 10, October, p. 38.

Koerner, R. M., Lord, A. E., and McCabe, W. M., 1978, "Acoustic Emission Monitoring of Soil Stability", Journal of the Geotechnical Engineering Division, ASCE, v. 104, no. GT5, Proc. Paper 13753, May, p. 571-582.

Koerner, R. M., Lord, A. E., McCabe, W. M., and Curran, J. W., 1977, "Acoustic Emission Behavior of Granular Soils", Journal of the Geotechnical Engineering Division, ASCE, v. 103, no. GT8, Proc. Paper 13129, August, p. 837-850.

Koerner, R. M., Lord, A. E., Jr., Okrasinski, T. A., and Reif, J. S., 1978, "Detection of Seepage and Subsurface Flow of Liquids of Microwave Interference Methods", presented at the April 10-14, National Conference on Control of Hazardous Material Spills, Miama Beach, Florida.

Koerner, R. M., Reif, J. S., and Burlingame, 1979, "Detection Methods for Location of Subsurface Water and Seepage", in Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, v. 105, no. GT11, November, 1979.

Latimer, C. A., 1922, "Small Dam Failure Wrecks Large Residence Section", in Engineering News-Record, v. 89, no. 3, July 20, 1922, p. 121.

Leaird, J. D., 1979, Slope Stability Monitoring with Acoustic Emission", Acoustic Emission Technology Corp. Inc., Sacremento, July.

Leaird, J. D., 1980, Theory and Current Practice in Acoustic Emission/Microseismic Activity as Applied to Geotechnical Problems, Acoustic Emission Technology Corporation, Sacramento, California, January.

Lessons From Dam Incidents, 1973, International Commission on Large Dams, 2nd Edition, September.

Lessons From Dam Incidents, 1975, Published jointly by the ASCE and the Committee on Failures and Accidents to Large Dams of the United States Committee on Large Dams.

Lippencott, J. B., 1898, "The Failure of the Sane River Dam, Turlock Irrigation District, California", in Engineering News, v. 40, no. 16, p. 242.

Longley, F. F., 1910, "Failure of an Earth Dam with Concrete Core Wall at Erindale, Ontario", Engineering News, v. 63, no. 15, Aug. 14, 1910, p. 439.

Lord, A. E., and Koerner, R. M., 1974, "Acoustic Emission Response of Dry Soils", Journal of Testing and Evaluation, JTEVA, v. 2, no. 3, May, p. 159-162.

Lord, A. E., and Koerner, R. M., 1975, "Acoustic Emissions in Soils and Their Use in Assessing Earth Dam Stability", Journal of Acoustical Society of America, v. 57, no. 2, February.

Lundien, J. R., 1971, "Terrain Analysis by Electromagnetic Means - Laboratory Measurement of Electromagnetic Propagation Constants in the 1.0 to 1.5 GHz Microwave Spectral Region", Technical Report 3-693, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mass.

Mearns, R., and Hoover, T., 1973, "Subaudible Rock Noise (SARN) as a Measure of Slope Stability", Final Report, CA-DOD-TL-2537-1-73-24, Department of Transportation, Division of Highways, Sacremento, California, August.

Merriman, Mansfield, 1930, American Civil Engineering Handbook, Chapman and Hall, Ltd., London.

Middlebrooks, T. A., 1953, "Earth Dam Practice in the United States", Transactions, Am. Soc. Civil Engrs, Centennial Volume, p. 697. (Table 2.1:1, p. 116, Earth and Earth Rock Dams).

Mitchell, A. K., 1912, "Failure of a Log and Earth-fill Dam at Union Bay, B. C.", in Engineering News, v. 67, n9. 9, Feb. 29, p. 414.

Murphy, E. C., 1905, "Failure of Lake Avalon Dam Near Carlsbad, N. M.", in Engineering News, v. 54, no. 1, July 6, p. 9.

Murphy, E. C., 1907, "Destruction of Debris Barrier No. 1, Yuba River California", Engineering News, v. 58, no. 6, Aug. 8, p. 133.

Musson, E. F., 1905, "The Failure of Two Reservoir Dams by Flood at Sherburne, N. Y.", Engineering News, v. 54, no. 11, Sept. 14, p. 274.

Nagle, J. C., and Shuler, T. C., 1928, "Slumps Complicate Garza Dam Construction", in Engineering News-Record, v. 100, no. 20, May 17, p. 772.

Obert, L., 1941, "Use of Subaudible Noises for Prediction of Rockbursts", U. S. Bureau of Mines RI 3555, U. S. Department of the Interior, Superintendent of Documents, U. S. Government Printing Office, Washington, D. C.

Pence, W. D., 1911, "Failure of the Dells and Hatfield Dams and the Devistation of Black River Falls, Wisc., Oct. 6, 1911", Engineering News, v. 66, no. 16, Oct. 19, 1911, p. 482.

Piatt, C. E., Jr., Editor, 1978, "Underground Radar", in News in Engineering, Engineering Experiment Station, College of Engineering, Ohio State University, Columbus, Ohio, v. 50, no. 3, May 1978.

Public Law 92-367, 1972, National Dam Inspection Program, 92nd Congress, H. R. 15951, August 8.

Redpath, B. B., 1973, "Demolition of Fort Meade Dam, Sturgis, South Dakota, June 1972", U. S. Army Engineers Waterways Experiment Station, Misc. Paper E-73-1, January.

Rubin, L. A., Griffin, J. N., and Still, W. L., 1976, "Subsurface Site Investigation by Electromagnetic Radar - Phase I Report - Feasibility", ENSCO, Inc., Springfield, Virginia, March 31.

Ruhling, G. H., 1915, "Failure of a Small Earth Dam", in Engineering News, v. 73, no. 23, June 10, 1915, p. 1121.

Safety of Small Dams, 1974, Engineering Foundation Conference, American Society of Civil Engineers, 345 E. 47th Street, New York, p. 67.

Saville, C. M., 1907, "The Earth Slip in the Face of the Embankment of the North Dyke of the Wachusett Reservoir", Engineering News, v. 57, no. 17, April 25, 1907, p. 464.

Saville, C. M., 1916, "Investigations for Dam and Reservoir Foundations" in Engineering News, v. 75, no. 26, p. 1229.

Schuyler, J. D., 1908, Reservoirs for Irrigation, Water-power, and Domestic Water Supply, John Wiley and Sons, Inc., New York, 2nd Ed.

Schuyler, J. D., 1909, "Nodes on the Failure of Necaxa Dam", Engineering News, v. 62, no. 3, July 15, 1909, p. 72.

Shaughnessy, M. M., 1925, "Sheffield and Gem Lake Dam Failures", in Engineering News-Record, v. 95, no. 5, July 30, 1925, p. 194.

Sherard, J. L., 1953, Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams., Tech. memorandum 645, U. S. Department of Interior, Bureau of Reclamation.

Sherard, J. L., Woodward, R. J., Gizienski, S. F., and Clevenger, W. A., 1963, Earth and Earth Rock Dams, John Wiley and Sons, New York.

Sherard, J. L., Ryker, N. C., and Decker, R. S., 1972, "Piping in Earth Dams of Dispersive Clay", Proceedings, Specialty Conference on the Performance of Earth and Earth Supported Structures, ASCE, 1972, v. 1, pp. 589-626.

Silent, R. A., 1916, Failure of the Lower Otay Dam", in Engineering News, v. 75, no. 5, p. 334.

Sowers, G. F., 1961, "The Use and Misuse of Earth Dams", Consulting Eng., July.

Sterling, Guy, 1916, "Analysis of the Failure of an Earth-fill Dam", Engineering News, v. 75, no. 2, p. 56.

Tatro, C. A. and Liptai, R. G., 1962, "Acoustic Emission from Crystalline Substances", Proceedings of the Symposium of Nondestructive Testing, Southwest Research Institute, San Antonio, Tex., p. 145-174.

Tibbetts, F. H., 1929, "Repair of Breaks in Outlet Tunnel of Rock-Fill Dam", Engineering News-Record, v. 102, no. 23, June 6, p. 904.

Tschantz, B. A., 1979, Carter Administration Review and Resolution of Dam Safety Problems, MIT-Stanford Seminar and Workshop, New Perspectives on the Safety of Dams, Cambridge, Mass., June 25-29, 1979.

Unterberger, R. R., 1979, "Locating Discontinuities in Geologic Environments by Radar and Sonar Probing Techniques", presented at the June 18-21, Rapid Excavating and Tunneling Conference, Atlanta, Georgia.

Walters, R. C. S., 1962, Dam Geology, Butterworth and Co. Ltd., London.

Wegemann, Edward, 1927, Design and Construction of Dams, Chapman and Hall, Ltd, London, 8th Ed.

Whited, W., 1904, "The Recent Failure of a Waterwords Dam at Scottdale, Pa.", in Engineering News, v. 52, no. 5, Aug. 4, 1904, p. 107.

Willis, W. N., 1916, "Undermined By Seepage Flow, Lake Toxaway Dam Fails", Engineering Record, v. 74, no. 9, Aug. 26, 1916, p. 273.

APPENDIX I. SUMMARY OF EQUIPMENT SURVEY

METHOD	PRIMARY USE	ADVANTAGES	DISADVANTAGES	STAGE OF DEVELOPMENT	APPROXIMATE COST
Acoustic Emission	Slope stability and seepage	Gives advanced warning of slope instability. Traces seepage flow and relative magnitude. Inexpensive.	Background noise troublesome. Not presently widely used. Still in research.	Available Research in progress	\$5500.00 Total for portable field unit.
Borings See Table	Seepage and slope stability	Gives exact subsurface conditions.	Very expensive. Difficulty in taking borings in some terrain. Interpolation of data complicated. Many corrections to data must be determined.	Available.	\$600-\$700 per 500 ft. hole direct cost when run with another log. (This does not include the cost to drill).
Crest Settlement monuments	Crest stability	Gives exact amount of crest settlement.	Limited to crest area.	Available.	Very cheap.
Geophysical (seismic)	Seepage and slope stability	Refraction method can identify type of liquid. Relatively common use. Technique well established.	Expensive Refraction needs dense lower layer.	Available.	Seismic Refraction: \$20-\$60 per depth determination with portable equipment. \$120-\$160 per depth determination with vibroseis.
Geophysical (electric)	Seepage	Cost less than seismic Relatively common use. Identify type of liquid. SP gives direction and relative magnitude of flow.	Salts and metal troublesome. Depth limited. SP method not widely used.	Available	\$4000 - \$6000
Gravimeter	Determine gravity anomelies		Could be used in foundation problems dealing with rock.	Available.	\$9600 basic
Infrared	Seepage	Uses natural phenomenon. Covers large or small area.	Limited detail in complicated topography. Expensive Not widely used. Still in research.	Available. Research in Progress	\$5000.00

METHOD	PRIMARY USE	ADVANTAGES	DISADVANTAGES	STAGE OF DEVELOPMENT	APPROXIMATE COST
Magnitometer	Detect changes in earth's magnetic field.	Equipment is becoming lighter, more portable and less expensive.	Not widely used in Geotechnical work except in rock/metallic related projects. Could be adopted for foundation (rock) work.	Available	$3000.00
Microwave Pulsed	Seepage and limited slope stability.	Traces surface of water. Good penetration depth. Continuous data for contouring. Locates voids, fractures and faults.	Expensive. Needs sharp interface. Upstream detection not possible.	Limited availability. See Table	
Dienometer tube	Monitor water level	Inexpensive. Gives exact water level in dam. Widely used.	More expensive to install after structure completed.	Available.	Very expensive.
Radar. See microwave pulsed pg. and Table	Seepage and limited slope stability			Limited availability.	Downhole $15,000.00 purchase.
Slope indicators (tilt meters) (1) downhole (2) on surface	slope stability	Gives changes of slope due to settlement, upheaval, etc.	(1) The need for drill holes (2) To adequately monitor a dam many indicators would be needed to cover the dam.	Available.	(1) Probe: $3000.00
Slope Stability Analysis Bishop's Method Swedish Slices	Slope stability	Gives good indication of factor of safety of slopes.	Need soil samples to define parameters. Need computer time.	Available.	Consultant's Fee.
Sonar	Seepage and limited slope stability	See microwave; pulsed and Table			
Strain Gague (soil)	slope stability	Measures soil strain over given distance. Measures settlement of embankments for long of short periods.		Available.	

METHOD	PRIMARY USE	ADVANTAGES	DISADVANTAGES	STAGE OF DEVELOPMENT	APPROXIMATE COST
Temperature	Seepage	Uses natural phenomenon.	Lengthy readout time. Requires 0.01°C sensitivity. Not widely used.	Available. Still in re-search.	
Tracers non-radioactive	Seepage	Inexpensive. Defines flow path. Flourescent dyes are detectable at low concentrations. Most widely used.	Source of seepage is required. Often difficult to place. Absorption is a problem. Dilution is common.	Available.	Very Inexpensive
Radioactive	Seepage	Easily detected. Widely used.	Difficult to place Expensive. Health and environmental hazard.	Available.	Inexpensive
Water balance	Seepage	Inexpensive Estimates magnitude of problem. Common use.	Some variables almost im-possible to measure. Does not give leak location.	Available.	Inexpensive.
Water velocity	Seepage	Simplicity of device. Good for wide range of leaks. Gives relative size of leaks. Gives location of leak.	Needs sensitive velocity meters. Needs quiet surrounding water. Needs stationary support or large boat for deep water. Problem with small leaks.	Available	Flow meters $1500.00
Benthonic	Seepage	Less sensitive equipment than velocity meters. Good for wide range of leaks. Gives relative size of leak. Gives location of leak.	Needs sediment layer. Needs stationary support or large boat for deep water. Problem with small leaks.	Available.	

APPENDIX II. EXPLANATION OF SUBCATEGORIES USED IN TABLE II-I

A PIPING

- A1 Embankment piping
- A2 Foundation piping
- A3 Outlet
- A4 Animal burrows
- A5 Abutment
- A6 Thru cracks

B OVERTOPPING

- B1 Inadequate spillway
- B2 Completed structure
- B3 Uncompleted structure
- B4 Partial

C SEEPAGE

- C1 During filling
- C2 Along outlets
- C3 Embankment
- C4 Abutment
- C5 Foundation
- C6 Upstream
- C7 Downstream

D SLIDING

- D1 Upstream
- D2 Downstream
- D3 Due to seepage
- D4 During filling
- D5 Drawdown
- D6 Abutment
- D7 During construction
- D8 Foundation
- D9 Core pressure
- D10 Reservoir slopes

E APERTURE WORKS

- E1 Gate
- E2 Spillway
- E3 Outlet pipe
- E4 Blockage

F SETTLEMENT

- F1 Crest
- F2 Upstream
- F3 Downstream
- F4 Core
- F5 Foundation

G POOR CONSTRUCTION PRACTICES

- G1 Poor compaction

H BLOWOUT

- H1 Overtop

I Breach

J SLOPE PROTECTION

- J1 Internal
- J2 Upstream
- J3 Downstream

K CRACKING

- K1 Upstream
- K2 Downstream
- K3 Drawdown
- K4 Crest

L EARTHQUAKES

- L1 Cracking
- L2 Settlement
- L3 Slide

APPENDIX III. LIST OF DAM FAILURES

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Abberton, G. Britain	1940	1937	Lessons, Table VI (1975)	17 5	E	A-3	Dam slide. (Lessons, 1975)
Ahraura, India	1954		Lessons, Table VI (1975)	26 8	E	F-2	Main dam, internal erosion. (Lessons, 1975)
Alamo Arroyo Site	1960	1960	Lessons, Table IV (1975)	68 21	E	A-1	Leakage, foundation piping. (Lessons, 1975)
Alcora, Ky.	1936	1966	Lessons, Table IV and p. 103 (1975)	265 81	E	MR	Deterioration of spillway slab due to sulfate and alkali attack combined with freeze-thaw cycle. (Lessons, 1975)
Alexander, Hawaii	1932	1930	Lessons, Table IV and (1975) Middlebrooks (1953) Justin (1932) ENR, v. 104, (5-22-1930) p. 869	95 29 140 43	E-H	DDC	Sliding embankment downstream slope. (Lessons, 1975). Core pressure slide before completion. (Middlebrooks, 1953), core pressure slide due to internal liquid pressure. (Justin, 1932). Dam under construction so no water behind it. Failure similar to Calveras and Necaxa dams in that the core had built up considerable pressure. Began at toe as a bulge then gushed out as a liquid. Liquid was not core itself but believed to have originated in transition zone. Ten days before failure it was noted that the area of the break had ceased to drain. (ENR, v.104). Failure due to defective drainage and partial liquefying of the walls of clayish material deposited near the faces of the dam. The relatively wide core of excessive fine material and the presence of semi-liquid material, caused the downstream face just above the small rock toe to bulge out expelling a gush of liquid mud. (Justin, 1932).
Alpine, Ca.	1906	1969	Lessons, Table IV and p. 105 (1975)	45 14	R-E	MR	Stability of the structure, especially for earthquake loadings, had marginal safety and the spillway capacity was inadequate. (Lessons, 1975)
Alum Fork	1936	1930	Lessons, Table IV (1975)	115 35	E	A-2	Leakage, foundation. (Lessons, 1975)
American Falls, Id.	1926	1929	Lessons, Table IV and p. 106	94 29	GE	MR	A gravity dam with earthfill embankment on each end. Incident refers to concrete deterioration on main dam. (Lessons, 1975)
Amistad, Tx.	1969	1965 1969	Lessons, Table IV and p. 108 (1975)	287 87	G-E	A-1	1965 cofferdam failed due to mismatched piling interlocks during construction. 1969 concrete adjacent to guides and seals of upstream gate due to rapid drop of gate. (Lessons, 1975)
Anaconda, Mont.	1898	1936	ENR v. 121 Middlebrooks (1953) Lessons, Table IV (1975)	72 22	E concrete core	F-1	No known reason. (Lessons, 1975). Seepage slide. (Middlebrooks, 1953). Owned by Anaconda Copper Mining Co. Gave no reason for its collapse. (ENR, v. 121)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Angels,	1895	1895	Lessons, Table IV (1975)	52 16	Gravity Earth	F-1	Leakage foundation piping. (Lessons, 1975)
Ansonia, Conn.	1875	1894 1912	ER, v. 30, 11-10-1894 EN, v. 47 Middlebrooks, (1953)	72 22	E concrete core		Seepage slide. (Middlebrooks, 1953). Blowout under dam. (Saville, 1916) Two of the reservoir dams were carried away, miscellaneous or unknown cause. (Hill, 1902). A concrete retaining wall had been put in to replace part of the earth embankment. The water worked its way under the foundation about the middle length of the wall. (EN, v. 47, No. 23). Water forced its way beneath retaining wall. (Joregenson, 1920). Failure due to water leaking along pipe laid through embankment, causing a gap 200 ft (61 m) and 35 ft (11m) deep. (Hill, 1902) (Jorgensen, 1920).
Apa, Turkey	1962	1963	Lessons, Table VI (1975)	31 9	E	A-2	Cracking. (Lessons, 1975)
Apishapa, Colo.	1920	1923	ENR, v. 91 p. 357, 418 Lessons, Table IV (1975) Middlebrooks, (1953)	112 34	E rolled concrete core	F-1	Leakage embankment piping (Lessons, 1975). Piping through settlement cracks (Middlebrooks, 1953). Settlement cracks had been causing leakage at each end and through the foundation. A year had elapsed since the leaks began and repaired soon after the repairs had been made and leaks had all but stopped a large amount of water entered the reservoir at the east end from a cloudburst in the headwater region (without excessive rains near basin). A few minutes after this water entered the basin a stream burst from downstream slope near west end and 35 ft ( ) below top. The water entering the east part of dam and water streaming from west end was determined to be the same. Soon the top caved in from the east to west. Total failure was then at this point. (ENR, v. 91, No. 9, p. 35) Cause of failure due to 1) unsuitable material; 2) insufficient water in puddling and rolling and; 3) thickness of the layers of the fill was too great. These three conditions led to severe settlement which formed caverns conditions 30 ft below the crest. These caverns were unknown to the workers and led to failure when the reservoir was filled. (Due to the rain in the headwaters) (Field, 1923). Failure began with the formation of a nearly horizontal crack which extended clear across the upper portion of the embankment. When first observed, this crack intersected the water surface at a point 100 ft (30 m) from the east end and dipped slightly down toward the west end, intersecting the downstream slope 35 ft (11 m) below the crest, 150 ft (46 m) from the west end. There can be little doubt that the crack had considerable initial width before the water broke through the upstream slope. The crack pattern and greater settlement of the upstream slope than the crest supply clear evidence that the lower portion of the embankment settled more than the upper. The very dry and rigid upper portion undoubtedly tended to arch the 300 ft (91m) across the valley and could not settle with the lower portion resulting in the crack and subsequent failure.

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Apishapa, Colo. (continued)							The leak occurred when water level was raised above previous high water level. Final failure was due to gradual piping process. (Sherard, 1953) On the day of failure, a settlement on the upstream face at the water's edge was noted about 100 ft (30 m) west of the east end. Water was passing into the depressed area into the body of the dam, a few minutes later water emerged on the lower slope about 500 ft (152 m) west of the point of entrance and 30 ft (9 m) below the crest. The water quickly ate back to a point immediately downstream from the point of entrance. Cause of failure is attributed to the solubility and lack of compactness of the soil. (Justin, 1932)
Arm Brook Site	1963	1964	Lessons, Table IV (1975)	60 18	E	A-1	Leakage, foundation (Lessons, 1975).
Ashizawa, Japan	1912	1956	Lessons, Table VI (1975)	15 5	E	F-2	Main dam, overtopped (Lessons, 1975)
Ashti, India		1883	ASCE, v. 49 Proc. Wegmann (1927) p. 234 Middlebrooks, (1953)	58 18	E rolled puddled core		Seepage through foundation. (Middlebrooks, 1953) A serious slip occurred after prolonged rains. The slip was attributed to the fact that the dam is founded for considerable portion of its length on clay soil containing nodules of impure lime and alkali, which make it semi-fluid when saturated. (Wegmann, 1927) The slip occurred after very heavy rains; the crest subsided 16 ft (5 m) causing the toe to bulge. The downstream portion of the foundation is believed to have become partially saturated and plastic, causing the slip. (Justin, 1932)
Avalon, New Mexico	1893	1893 1904	EN, v. 54 Middlebrooks, (1953) Lessons, Table IV (1975)	58 18	E-R	F-1	1893 overtopped (Lessons, 1975) 1904 leakage, embankment piping (Lessons) piping into rock (Middlebrooks, 1953)
Avoca, Pa		1892	EN, v. 47 Middlebrooks (1953)		E		Overtopped (Middlebrooks, 1953) Insufficient spillway (Hill, 1902) (Jorgensen, 1920)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Bad Axe Watershed No. 33	1965	1965 1967	Lessons, Table IV and p. 114	52 158	E	A-2	Seepage problems in both abutments developed during first filling of reservoir. Seepage in left abutment developed to a point of incipient failure in 1967. A sinkhold developed upstream. (Lessons, 1975)
Badua, India	1963 1966	1963 1964	Lessons, Table VI (1975)	42 13	E	A-2	1963 and 1964; Spillway cracking. (Lessons, 1975)
Baker City, Ore.		1890	EN, v. 47 Middlebrooks, (1953)		E		Dam burst, miscellaneous or unknown cause. (Hill, 1902)
Baldhill, N.D.	1951	1951-70	Lessons, Table IV and p. 115 (1975)	61 19	E	MR	From 1951-1970, wave action moved the upstream riprap and eroded the gravel bedding. (Lessons, 1975)
Baldwin Hills Res., Ca.	1951	1963	Lessons, Table IV (1975) Golze 1977, p. 188	262 80	E	F-1	Leakage, foundation piping (Lessons, 1975); failure due to man induced seismic activity. Movement along a fault produced slight offset, separation and leakage through foundation. (Golze, 1977) A few hours before dam failed a caretaker heard an unusual sound of running water in the spillway discharge pipe while making his routine inspection. He then inspected the manholes on the downstream berms of the main embankment and noted flows of muddy waters several times greater than normal. In the drainage inspection chamber, he found the reservoir underdrain pipes "blowing like fire hoses," discharging muddy water. Shortly thereafter, a small amount of seepage emergence was discovered on the downstream slope of the main dam believed to be coming from the east abutment. The seepage flow steadily increased until the east abutment was breached. About four hours passed between the first discovery of unusual flow and breaching. "The physical cause of failure was earth movement. The earth movement, due to subsidence, manifested itself by opening and by offsetting at a fault, a plane of weakness. Erodible material in and adjacent to the fault provided conditions that permitted rapid and complete failure." (Lessons, 1973)
Balsam, N.H.	1927	1929	ENR, v. 54 ENR, v. 102 p. 885 Lessons, Table IV (1975)	60 18	E concrete core	F-1	Flow discharge, destroyed spillway, eroded toe causing sluffing. (Lessons, 1975) Due to heavy rains, high discharge, washing away of riprap, causing sluffing of dam. (ENR, v. 102)
Bartley, G. Britain	1931	1927 1929	Lessons, Table VI (1975)	?	E	A-3	Dam, slide, 1927 and 1929. (Lessons, 1975)
Barton, Id.	1910	1922	Sherard, (1953) Middlebrooks, (1953)	40 12	E rolled		Seepage slide. (Middlebrooks, 1953); seepage began directly after construction and increased slightly every year. Due to seepage a slide occurred on the downstream slope. (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Beargamil, Australia	1912	1945 1945	Lessons, Table VI (1975)	17 5	E	A-1	Dam, slide: 1945 (Lessons, 1975) Dam, slide: 1945 (Lessons, 1975)
Bear Gulch, Ca.	1890	1914	Lessons, Table IV (1975) Sherard, 1953	61 19	E rolled	A-1	Sliding embankment upstream slope. (Lessons, 1975) 1914 drew down slide; 1930 downstream toe became moist and began to slough, 1936 cracks downstream slope indicating incipient slide, repaired: 1944-45 drawdown cracks. (Sherard, 1953)
Beaver Park, Colo.	1914	1914	Lessons, Table IV (1975) ASCE Trans., v. 65	98 30	R	A-1	Leakage, foundation. (Lessons, 1975) Dam is rock with concrete face. Problem with leakage through foundation of porous material. The dam was placed between a constriction in the canyon formed by the projection of a nose or promontory of rock extending out from the mountain on the left hand side of the creek, forming a narrow ridge or saddle. This projection appeared to be solid but caused much trouble. The basic formation at and near the dam is trachite, a soft porous material resembling sandstone. This formation is badly broken up and in seams, and does not necessarily present a threat to the stability of a large dam if properly designed and constructed. Leaking could not have been prevented. (Hinderlider, AIS)
Belden, Ca.	1958	1966-67	Lessons, Table IV and p. 123 (1975)	164 50	R	A-1	1966; accident because of gate control interchange during construction. 1967, gate failure due to improper use poor configuration of interconnection between studs and conduit. (Lessons, 1975)
Belle Fourche, S.D.	1911	1912	Lessons, Table IV (1975)	115 35	E	A-1	Slope protection concrete slabs moved. (Lessons, 1975)
Belle Fourche, S.D.	1911	1932	Lessons, Table IV (1975)	115 35	E	A-2	Sliding embankment upstream slope. (Lessons, 1975)
Belle Fourche, S.D.	1911	1965	Lessons, Table IV and p. 125 (1975)	120 37	E	A-1	1950 spillway was reported to have inadequate capacity along with deterioration of concrete; 1965 terminal drop structure failed completely and stream bed eroded 10 feet (3 m). (Lessons, 1975) In 1928 there was a 35 ft (11 m) drawdown in 125 days. As a result of this, five major cracks occurred on the downstream edge of the crest. They varied from 1/2 to 2 inches in width on the surface of the crest, 25-150 ft (8-46 m) long and 3-12 ft (1-4 m) in depth. The cracks were filled by washing them full of sand. In 1931 the reservoir was again drawn down rapidly to a point below the 1928 drawdown level. Due to this, a surface slide occurred on the upstream slope. The maximum depth of the surface normal to the slope was about 10 ft (3 m). (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Belmont, G. Britain	-	1923	Lessons, Table VI (1975)	25 8	E	A-1	Dam, slide. (Lessons, 1975)
Bila Desna, Czech.	1915	1916	Lessons, Table VI (1975)	17 5	E	F-1	Blowout, overtop. (Lessons, 1975)
Bilbery (Holmfirth) England		1852	Walters, 1962, p. 50		E		Failed through water forcing its way through fissures in the rock and through upward pressure rapidly washing away the earthern embankment. (Walters, 1962)
Bishop Creek, Ca.	1908	1909	EN, v. 60, No. 6		E	DDC	A freshnet in the creek washed out part of a small earth intake dam under construction and was due to the early arrival of high water and before spillway was in proper shape. (EN, v. 60)
Black Beauty	1951	1951	Lessons, Table IV (1975)	50 15	E	A-2	Deformation, differential transverse embankment cracks. (Lessons, 1975)
Blackfoot	1911	1913	Lessons, Table IV (1975)	49 15	E-R	A-1	Leakage, foundation piping. (Lessons, 1975)
Black Rock	1907	1909 1936	Lessons, Table IV (1975)	70 21	E	F-2	Leakage foundation piping both incidents. (Lessons, 1975)
Blaen-Y-CwM, G.Brit.	1937	1936	Lessons, Table VI (1975)	18 5	E	A-2	Dam, leakage. (Lessons, 1975)
Blairtown, Wyo.		1888	EN, v. 47 Middlebrooks, (1953)		E rolled		Piping along an outlet. (Middlebrooks, 1953) Piping along outlet laid through embankments. (Hill, 1902)
Blanchard Hydroelec. Station	1925	1939	Lessons, Table IV (1975)	62 19	G-E	MR	Poor construction practice. (Lessons, 1975)
Blithfield, G. Britain	1953	1962	Lessons, Table VI (1975)	16 5	E	A-1	Main dam; overtopping. (Lessons, 1975)
Blue Water. N. Mex	1908	1909	EN, v. 62 (9-30-1909) Middlebrooks, (1953) Justin, (1932)	35 11	R		Overtopped (Middlebrooks, 1953); due to insufficient spillway. (Justin, 1932) Due to excessive rains; water entered the spillway and outlet pipes were opened; water level was still rising. About 2:00 AM the watchman observed a leakage on the

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Blue Water, N. Mex. (continued)							north side of the dam and also a slight percolation on the south side near the junction of the dam with the spillway wall. The latter subsequently ceased by the chocking of a "slough" of the bank, and remained sealed. Final failure came 2:00 PM the next day when dam was overtopped. (Anderson, 1909) Failed by overtopping. (Jorgensen, 1920)
Bolan, Pakistan	1965	1976	ENR, v. 197, No. 12	442	E		Torrential rains and floodwaters brought the water level to 44 ft behind the dam. More rain and wave action sent water over the top and through the dam near its center.
Bolton, Conn.	1940	1938	Middlebrooks, (1953)	10 3	E rolled	DDC	Overtopped. (Middlebrooks, 1953)
Bon accord, South Africa	1925	1937	Lessons, Table VI (1975)	18 5	E	F-2	Dam, slide. (Lessons, 1975)
Bonney Reservoir, Co.	1901	1903	EN, v. 47 ENR (4-25-1903) Middlebrooks (1953)	34 10	E rolled		Break in dam. (Middlebrooks, 1953) Break in dam over 100 ft (30 m). (Jorgensen, 1920)
Bowman	1872	1928	Lessons, Table IV (1975)	168 51	R	A-1	Leakage, Tunnel. (Lessons, 1975)
Bradford, England		1896	ASCE Proc., v. 49 Moddlebrooks, (1953)	90 24	E rolled puddled core		Piping along outlet. (Middlebrooks, 1953) Piping along an outlet, failure due to the imperfect packing and bonding of the puddle core along the outlet pipes. (Justin, 1932)
Braunig	1962	1963	Lessons, Table IV (1975)	90 27	E	A-2	Slope protection; riprap too small. (Lessons, 1975)
Breakneck, Pa		1902	EN, v. 47 Middlebrooks, (1973)		E rolled		Overtopped. (Middlebrooks, 1953) Cloud burst in the valley above the reservoir. The downpour was so great that the large flood channel and spillway, which had been ample to take care of all floods for 16 years, were overtaxed and the water flowed over the top of the dam its entire length to considerable depth. The water flowing over the dam washed out the lower side of the dam badly, and finally, after support was gone, broke the core wall in several places. (EN, v. 47, No. 23)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURES
Bridgeport, Conn.	1855	1905	ER, v. 52 (7-22-1925) Justin, (1932)		E rolled		Overtopped (Middlebrooks, 1953) due to insufficient spillway (Justin, 1932) Inadequate spillway; dam overtopped. (Jorgensen, 1920) An inadequate spillway caused the washout of an earth dam. (ER, v. 52, No. 7)
Briseis, Australia	1934	1929	Lessons, Table VI (1975)	27 8	E	F-1	Blowout, overtop. (Lessons, 1975)
Brooklyn, NY	1893	1893	EN, v. 47 Middlebrooks, (1953)		E		Foundation seepage. (Middlebrooks, 1953) Puddle bottom of reservoir leaked, (Jorgensen, 1920). During first filling, all water that was pumped in leaked out. Failure due to improper construction of bottom. (Hill, 1920)
Brookstail No. 3 North Calif.	1970	1971	Lessons, Table IV and p. 127 (1975)	49 15	E	A-2	Due to reservoir filling and unusually heavy rains, the ground was saturated. The material on the hillside upstream and above the spillway slid. Unstable conditions of the hillside upstream and above the spillway noted prior to construction. (Lessons, 1975)
Broomhead, G. Britain	1934	1929 1930	Lessons, Table VI (1975)	31 9	E	A-2	Dam leakage (1929). (Lessons, 1975) Cracking (1930). (Lessons, 1975)
Brush Hollow, Colo.	1925	1923 1928	Sherard 1953 Lessons Table IV (1975)	100 30	E rolled	A-1	Conduit break due to settlement 1923. (Sherard, 1953) Sliding embankment upstream slope. (Lessons ,1953) Slide occurred due to drawdown 1928. (Sherard, 1953)
Buffalo Creek, NC	1973-74	1970	Lessons, Table IV and p. 129	105 32	E	DDC	Due to heavy precipitation, uncompleted structure overtopped. (Lessons, 1975)
Bully Creek, OR	1913	1913	ENR, v. 94	125 38	R concrete core	F-1	Condemned dam; abandoned 1913; overtopped, due to flooding, and subsequent breakage or core wall. (ENR, v. 94, 1925)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Calaveras, Calif.	1914	1918	Lessons, Table IV (1975) Middlebrooks, (1953) Justin (1932) EN, v. 72 p. 692 ENR, v. 80	220 67	E-H clay core	A-3	Sliding embankment; upstream slope. (Lessons, 1975) Excessive core pressure, (Middlebrooks, 1953)
Calaveras, Calif.	1914	1938	Lessons, Table IV (1975)	228 69	E-H	A-1	Flow discharge, damaged spillway. (Lessons, 1975) During construction, a serious slip occurred. It appears that the clay layers underlying the upstream toe lubricated the plane of slip. General cause of failure is with the increase in height of the semi-liquid clay core the hydrostatic pressure due to it (the core) kept increasing until it exceeded the stability of the firmer portion forming the toe, and the slip occurred. (Justin, 1932) Middle section of upstream side pushed into reservoir during construction. Clay core exerting outward pressure towards bottom due to its great height and softness. (Jorgensen, 1920) Central section of upper part of the upstream side of dam and a large part of day core back of it slid into the reservoir. Slide appears to be due to failure of hydraulic material to dewater and solidify. Excessive core pressure, lack of water behind dam prevented equalization of pressure. (Hayden and Metcalf, 1918)
Castelwood, Colo.	1890	1933	ASCE, Trans., v. 65 Middlebrooks, (1953)	70 21	R		Spillway over dam failed. (Middlebrooks, 1953)
Cave Creek Reservoir J-62, Nev.	1963	1969	Lessons, Table IV and p. 131 (1975)	78 24	E	A-1	Leakage along boundary of old dam and new raised section. Downstream slope sloughed. (Lessons, 1975)
Cedar Creek, Id.	1920	1971	Lessons, Table IV and p. 134 (1975)	78 24	E concrete core	A-1	History of problems associated with seepage and settlement at right abutment. 1971 water was seeping around abutment end of core wall and flowing vertically into boulders and rubble in fill material. (Lessons, 1975)
Cedar River, Wash.		1915			E		Gravel stratum beneath dam. (Saville, 1916)
Cercey Dam, France	1834-36	1842 1866	Sherard, (1953)	38 12			The dam has a history of slides on both upstream and downstream slopes. In 1842, just after the reservoir had been drawn down for the first time, a slide occurred in the upstream slope. In 1866, following rainstorms, fresh slides occurred at the same point on the upstream slope as in 1842, and also to either side of the original slide. (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Chambers	1885	1881	Lessons, Table IV (1974)	51 17	E	F-2	Flow discharge, spillway destroyed. (Lessons, 1975)
Charles Lee Tilden Park	1938	1964	Lessons, Table IV (1975)	88 27	E	A-1	Leakage, conduit. (Lessons, 1975)
Charmes, France	1902-06	1909	Sherard, (1953)	55 17	E		After reservoir had been drawn down 30 ft (9 m) from high water level, a slide started on the upstream slope. The slide continued to move slowly for about 6 weeks. (Sherard, 1953).
Chandler Lake, Ga.	1955	1955	Safety of small Dams 1974	25 8	E	F-1	No engineering design employed. The reservoir had been full only a short time when seeps were noted in the valley below the toe. No action was taken to control seepage. Some months after the reservoir had filled there was a hole in the dam 20 ft (6 m) in diameter upstream and downstream and the reservoir was emptied. (Safety, 1974)
Cheaha Creek Watershed 6, Ala	1970	1970	Lessons, Table IV and p. 137	93 28	E	DDC	Due to heavy precipitation, uncompleted structure overtopped. (Lessons, 1975)
Cheney, Kan.	1965	1971	Lessons, Table IV and p. 141 (1975)	126 38	E soil cement face	MR	Soil cement face partially removed by excessive wave height caused by extremely high winds. (Lessons, 1975)
Clear creek, Colo.	1928	1970	Lessons, Table IV and p. 146 (1975)	100 30	E	MR	Concrete in outlet conduit had eroded due to cavitation. (Lessons, 1975)
Clendening	1937	1937	Lessons, Table IV (1975)	64 20	E	DDC	Sliding embankment, upstream slope. (Lessons, 1975)
Coal Refuse. W.V. Dam No. 3	1973	1973	Wahler et al, (1973)		E		Four dams built by dumping coal waste and "back blading"; no real compaction. One day before failure cracking appeared (10:00 to 11:00 PM); 7:15 AM. next day, debris circling as a whirlpool on right side of dam. No change in water level, 7:50 AM large cracks and slumps on downstream face; 8:10 AM 75-100 ft (23-30 m) of right side failed with the remainder of the dam sliding into the breach. (Wahler, 1973)
Cobb Creek No. 1	1959	1959 1962	Lessons, Table IV (1975	75 23	E	A-2 A-1	Deformation differential, spillway structure. (Lessons, 1975 Leakage, foundation piping. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD	CAUSE OF FAILURE
Cobden, Ont. Canada		1894	EN, V. 48 Middlebrooks, (1953)	35 11	E rolled		Article in Engineering News v. 32, No. 17, no details given.
Cogswell	1934	1934	Lessons, Table IV (1975)	280 85	R	A-2	Slope protection concrete slab badly cracked. (Lessons, 1975)
Cold Springs, OR	1908	1931	Corps of Engrs.	98 30	E rolled		Riprap displaced by waves. (Middlebrooks, 1953)
Colley Lake		1960	Lessons, Table IV (1975)	60 18	E	A-2	Overtopping, completed structure. (Lessons, 1975)
Colorado Spr. 4 Colorado Spr. Co. Cold Springs, Co. (Middlebrooks)		1912 1912 1912	Lessons, Table IV (1975) Hannah and Kennedy 1938 ER, v. 66, p. 223 Middlebrooks, 1953	50 15	E rolled E	A-1	Sliding embankment downstream slope due to leakage embankment (Lessons, 1975) Partial failure due to piping. (Middlebrooks, 1953) Seepage through porous embankment and slide (Hanna and Kennedy, 1938) A seepage was noticed at several points along the downstream face at a depth of 26 ft (8 m) below the crest. This was followed by caving and slipping of the outer slope. A number of large holes were washed in the outer slope, one or two of which extended to within a foot or two of the crest. The line of seepage extended along and parallel to 26 ft (8 m) below the crest. It was thought that there was a layer of more permeable material just below the 26 ft (8 m) level which the water moved along. (En, v. 66, No. 8) Strong seepage through the dam was followed by caving and slipping of the outer slope. (Jorgensen, 1920)
Conconully, Wa.	1910	1967	Lessons, Table IV and p. 147 (1975)	70 21	E-H	MR	Concrete spillway deterioration due to inferior cement. (Lessons, 1975)
Conroe, Tex.	1973	1972	Lessons, Table IV and p. 150	64 20	E	DDC	Cracking of footings of upstream retaining walls of spillways caused by the consolidation of clay in cutoff trench below the retaining wall. (Lessons, 1975)
Conshohocken Hill, Pa.		1873 1876	ASCE Proc., v. 49 ENR, v. 18; Justin (1932) Middlebrooks, (1953)		E rolled no core		Piping (Middlebrooks, 1953); clay lining gave way (Justin, 1932) Bottom lined with 18-in clay upon which was laid a brick paving in cement. Bottom lining broke away in 1873. It was repaired and failed again in 1876-1879-1886. (Hill, 1902) (Jorgensen, 1920)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Cooney, Mont.	1936	1962	Lessons, Table IV and p. 152 (1975)	97 30	E rolled	A-1	1962 Flood discharge destroyed spillway. By June 1972, seepage developed through the right abutment and excessive seepage developed around the outlet. Sloughing occurred downstream slope in vicinity of outlet. (Lessons, 1975)
Corpus Christi, Tx.	1930	1930	Lessons, Table IV (1975)	61 19	E	F-2	Leakage foundation piping. (Lessons, 1975) Failure not certain at this writing; one possibility was initial penetration occurred from weir side of the abutment. A second possible conclusion is that initial penetration was through the embankment directly back of the abutment, by one of these three ways: 1) loose compaction of the earthfill close to the counterforked concrete abutment, leaving a zone of weakness for seepage; 2) unequal settlement of the fill on compressable soil and of the adjacent abutment on more rigid pile foundation, which might produce a line of weakness; 3) penetration under the cutoff or over its top or through the sheeting, along some line of weakness due to unknown cause. (Possibly earthquake.) (ENR, v. 105, No. 25, 1930, p. 974).
Costilia, N. Mex.	1920	1924 1941	ER, v. 75 Middlebrooks, (1953)	125 38			Embankment seepage (1924) Middlebrooks, (1953) Slough's (1941) Middlebrooks (1953) The dam developed some leakage after the first filling at level 9497. Leakage occurs at this elevation whenever the reservoir level reaches much above elevation 9500. This is thought to be due to greater compaction of material below 9497 or more pervious material used above 9497 elevation. In 1942, a shallow surface slide occurred on the downstream slope but was thought to be the result of saturation due to shallow percolation of excessive precipitation. (Sherard, 1953).
Courtright, Ca.	1958	1966	Lessons, Table IV and p. 153 (1975)	295 90	R concrete face	MR	Slope protection concrete slab badly cracked due to settling and movement downstream of the face. (Lessons, 1975)
Cowan Ford	1963	1965	Lessons, Table IV (1975	130 40	G-E	MR	Deterioration of concrete. (Lessons, 1975)
Crane Creek, Id.	1910	1928	Sherard (1953) Lessons, Table IV (1975)	63 19	E puddle core	A-1	Piping into tunnel outlet. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.:	CAUSE OF FAILURE
Crane Creek, Id.	1920	1969-70	Lessons, Table IV and p. 159 (1975)	62 19	E concrete core	MR	Inadequate spillway and slides in downstream slope. (Lessons, 1975) Cause of failure is a collapse of a section of the roof or walls of an outlet conduit. An engineer measuring streams in the vicinity noticed a stream of clear water emerging from the left downstream abutment contact about mid-height. The next day a gate tender discovered a slough on the downstream slope at the area where the leak was discovered. The next morning, bubbles and foam were noticed coming up from the upstream slope just opposite the slough area. Sacks of dirt were placed into the hole. About 12 hours later a larger hold appeared with some noise and two whirls or eddies. More sacks of soil were dumped in and the leak was stopped. The water level was drawn down and the hole repaired by refilling cavity with clay and compacting. There has been no trouble since. (Sherard, 1953)
Crane Valley, Ca.	1910	1949	Corps of Engrs.	130 40	H		Riprap displaced by wave. (Corps of Engrs, 1949)
Cranks Creek, Ken.	1963	1973	Lessons, Table IV and p. 161 (1975)	195 59	E	A-1	Due to heavy rains discharge damaged spillway; discharge eroded the area of the control weir. (Lessons, 1975)
Credit River, Ontario, Canada	1910	1910	EN, v. 63, p. 439	50 15	E concrete core		Overtopping (Middlebrooks, 1953) Due to flooding from rain and snow melt. Floods were not considered excessive. Water level did not reach the spillway level. During construction water had been diverted through sluiceways. The freshnet that caused the damage, came along, exceeding the capacity of the sluiceways and backing up water against the new work. The earth embankment had been partly completed on the upstream side of the core wall, but on the downstream side it had no support at all. Some of the concrete had been in place only 3 days. The pressure was too great for the structure to resist and failure occurred. The first break was quickly followed by more failure due to the scour of the flood and ice until 100 ft (30 m) of the dam was down. (Longley, 1910) Dam under construction; flood waters could not be carried away fast enough (Jorgensen, 1920).

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Dale Dyke, Eng.		1864	Wegmann, (1927) Middlebrooks (1953) Walters, 1962	95 29	E puddle core		Probable piping along outlet, (Middlebrooks, 1953); failure due to narrow crack on outer slope. (Wegmann, 1929) Poor design (Jorgensen, 1920) Built on alternating grits and shales coupled with somewhat poor construction of the embankment. Although true cause of failure is unknown it was probably due to undermining of the foundation caused by water percolating the gritstone under tne embankment owing to the cutoff trench not being deep enough and grouting procedures unknown at that time. (Walters, 1962)
Dallas, Tx.		1891	EN, v. 47 Hill, 1902	29 9	E rolled brick face core		Settlement (Middlebrooks, 1953) A break in reservoir embankment occurred June 1891. A large part of the bank sank vertically, due as was thought to quicksand beneath the foundation. The settlement was over 300 ft (91 m) in length and extended from the toe of the outer slope to within 5 ft (1.5 m) of the top of the inner slope. The brick and cement lining was cracked in an almost straight line at 11 ft (3 m) from the top and from this line to the top of the embankment there was a slight settlement. (Hill, 1902)
Dalton, NY		1912	EN, v. 67 p. 900 Middlebrooks (1953) Justin (1932)	29 9	E concrete core		Foundation piping (Middlebrooks, 1953) Piping, porous foundation. (Justin, 1932) Water worked its way through a sand bank against which one end of the structure rested and undermined to a depth of 10 ft (3 m) the glacial drift beneath the dam, a considerable section of which was destroyed. (EN, v. 67, p. 900)
Davis Reservoir, Ca.		1914	EN, v. 72, p. 106 Justin (1932) Middlebrooks. (1953)	39 12	E rolled		Piping around outlet, (Middlebrooks, 1953) No cutoffs on gate structure, (Juston, 1932) Failure due to lack of suitable cutoffs along the rear face of the gate structure which was in contact with the earth and the consequent passage of water along tnis smooth concrete surface. (Justin, 1932) At 2:00 AM night watchman discovered water pouring through a hole on right siue of gate structure, which rapidly tore away the fill on that side and was cutting into original hard pan sides. At 9:00 AM the gap had widened to 60-70 ft (18-21 m). Cause of failure may be attributed to any one of several conditions. 1) there was no provision for any cutoff wall on the side of the gate structure except the thin concrete facing. 2) the partially washed fill on the left side of the structure showed that it was composed of boulders, lumps of hardpan and fine sandy soil. It was not compacted into the hardpan on the bottom. When water was placed in the reservoir and before the water had gained any considerable head it is said to have leaked from under the fill. It is the opinion of someone who watched the construction of the fill that this leakage was the cause of the failure. (EN, v. 72, No. 2)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Debris Barrier No. 1, Calif.		1904	EN, v. 53 EN, v. 58 Middlebrooks, (1953)		H		Overtopped (Middlebrooks, 1953)
Dells and Hatfield	1905	1920	Jorgensen, (1920) Pence, (1911)				The two earth dams failed due to water flowing over top of dams (Jorgensen, 1920) Due to heavy rains the Dells dam was overtopped and breached. The water from the Dells dam and overtopped the Hatfield dam on its west wing wall of the spillway quickly stripping away the earth over the concrete corewall and washing out the dike around the end. (Justin, 1932) Failure due to overtopping. There had been excessive rains; for two days and just before dam was overtopped, workman had been raising crest with sandbags but had given up because water level was rising faster than the workers could raise crest level. (Pence, 1911)
Deruyler	1863	1940	Lessons, Table IV (1975)	72 22	E	A-1	Leakage, embankment. (Lessons, 1975)
Desabla Forebay, Ca.	1903	1932	Sherard (1953) Lessons, Table IV (1975)	53 16	E	A-1	Leakage, embankment. (Lessons, 1975) A leak developed through the fill that caused considerable cavitation in the upstream slope. (Sherard, 1953)
Detention W-1, Co.	1963	1966	Lessons, Table IV and p. 163 (1975)	57 17	E	A-1	June 1966 embankment overtopped by 2.5 feet and as a result an average of 1 foot depth of embankment was eroded from downstream slope. (Lessons, 1975)
Dickenson	1950	1954	Lessons, Tabke IV (1975)	62 19	E	A-1	Flow discharge, damaged spillway. (Lessons, 1975)
Drum Forebay	1913	1951	Sherard, (1953)	53 16	E		There had always been minor seepage through the dam. In 1951 there was some minor sloughing on the face of the dam due to a small stream at the maximum section. (Sherard, 1953)
Dry Creek, Mon.	1938	1939	ENR, v. 122 Middlebrooks, (1953)	46 14	E rolled		Piping (Middlebrooks, 1953) Failure associated with high spring runoff and porous foundation. Failure due to piping through foundation. (ENR, v. 122, No. 12 & 13)
Dudnava, India	1962	1962	Lessons, Table VI (1975)	32 10	E	A-2	Foundation, boiling. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Eagle Valley J-78, Nevada	1965	1965	Lessons, Table IV and p. 168 (1975)	74 22	E	A-2	During initial filling, heavy seepage emerged from the vertical bank of the stream immediately downstream of the embankment, due to inadequate construction of relief wells and toe drains. (Lessons, 1975)
East Branch	1957	1957	Lessons, Table IV (1975)	195 59	E	A-1	Deformation, differential transverse embankment cracks. (Lessons, 1975)
East Liverpool, Ohio	1901	1901	ER, v. 44 Middlebrooks, (1953)		E rolled		Piping along an outlet. (Middlebrooks, 1953) During first filling; break occurred over a pipe laid through the embankment. (Hill, 1902) (Jorgensen, 1920)
Edith C. Justus, Pa.	1971	1971	Lessons, Table IV and p. 169 (1975)	92 28	E grout curtain	A-2	During initial filling, excessive seeping noted from right abutment, just downstream of the embankment. Probable cause, inadequate grouting. (Lessons, 1975)
Eildon, Australia	1927	1929	Lessons, Table VI (1975)	40 12	E	A-1	Dam, slide. (Lessons, 1975)
El Estribon, Mex.	1940	1963	Lessons, Table VI (1975)	21 6	E	A-1	Other dam, sliding. (Lessons, 1975)
Elk City, Ok.	1925	1937	ENR, v. 116 Middlebrooks, (1953)	30 9	E rolled		Overtopped. (Middlebrooks, 1953) After heavy rains, dam overtopped by .15 ft (.45 m) for full length of dam during which a 150 ft (46 m) section from middle gave way. (ENR, v. 116, May 7, 1936, p. 678) The spillway had been raised to increase storing capacity of reservoir but the spillway capacity had not been increased. This was done without consultation with designer. (ENR, June 11, 1936, p. 850)
Ellington, Conn.		1890	EN, v. 47 Middlebrooks, (1953)		E rolled		Dam gave way due to miscellaneous or unknown cause. (Hill, 1902)
Emery, Ca.	1850	1966	Lessons, Table IV p. 170 (1975)	51 16	E	F-1	Piping of embankment material into conduit due to failure of conduit. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Empire, Colo.	1906	1909	ASCE Proc., v. 49 Middlebrooks, (1953) Justin (1932)	30	E rolled		Piping along an outlet. (Middlebrooks, 1953) due to settling. (Justin, 1932) Partial failure. Settling of embankment, caused a breaking of outlet conduit. Water entered through the breaks, washing away embankment around the conduit pipes causing further settlement, then destruction of the conduit and gate well, with washing away of the downstream and central portions of the embankment. (Justin, 1932)
Englewood	1921	1921 1970-72	Lessons, Table IV and p. 171 (1975)	111 34	E-H	DDC MR	Sliding embankment, upstream slope. (Lessons, 1975) Installation of pressure relief wells. Deterioration of concrete along edges and joints. (Lessons, 1975)
English, Ca.		1883	Schuyler (1908) Middlebrooks, (1953)				Overtopped. (Middlebrooks, 1953) Early in the morning and just prior to failure, a watchman heard two violent explosions, when he arrived at the dam he observed water pouring through a wide breach in the upper cribwork. Cause of failure is attributed to dynamite. (Shuyler, 1908) Rock filled, crib. Crib timber boards gave way, rock filling crumbled. (ENR, v. 100, No. 12, p. 472)
English water supply	1965	1965	Lessons, Table IV (1975)	52 16	E	A-2	Leakage, foundation. (Lessons, 1975)
Escanaba, MI, No. 1	1907	1930	ENR, v. 105, No. 2	36 11			Overflow of embankments at the end of spillway. (ENR, v. 105, No. 2, p. 71)
Escanaba, MI, No. 2	1910	1930	ENR, v. 105, No. 2	18 50			Overflow of embankment at the end of spillway. (ENR, v. 105, No. 2)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Fairview, Mass.		1922	ENR, v. 89 Middlebrooks, (1953)	30 9	E rolled concrete core		Piping, (Middlebrooks, 1953)
Fergus Falls, Min.		1909					Percolation under dam. (Saville, 1916)
Fellows, Mo.	1955	1965	Lessons, Table IV and p. 172 (1975)	104 31	E-R	A-1	Leakage due to poor construction; leakage was occurring at poor joints and spilled areas. (Lessons, 1975)
Flagstaff G., Australia	1963	1963	Lessons, Table VI (1975)	16 5	E	A-2	Foundation leakage. (Lessons, 1975)
Fontenelle	1964	1965	Lessons, Table IV (1975	139 42	E	A-2	Leakage, foundation. (Lessons, 1975)
Forsythe, Utah	1920	1921	Middlebrooks, (1953) Lessons, Table IV (1975) Sherard, (1953)	65 20	E rolled	A-1	Leakage, foundation piping. (Lessons, 1975) Piping under spillway; drawdown slide. (Middlebrooks, 1953) Piping under spillway; erosion of soil under spillway and spillway itself. (Sherard, 1953)
Fort Collins, Co.		1902	EN, v. 57 Middlebrooks, (1953)		E		
Fort Meade, SD	1924	1972	Lessons, Table IV and p. 175 (1975) Redpath, (1973)	55 16	R	A-1	Overtopping due to heavy rains. (Lessons, 1975) Rock with concrete upstream face and grouted rock downstream face. (Redpath, 1973)
Ft. Peck, Mont.	1940	1938	Lessons, Table IV (1975) Middlebrooks, (1953) ENR, v. 21	250 76	E-H H	DDC	Sliding embankment, upstream slope. (Lessons, 1975) Foundation slide. (Middlebrooks, 1953) Failure due to slump; began east end of dam, near gate shafts of gate tunnel, westward to mid section of upstream toe (an extra reinforced area), cause was not certain at writing. Slump similar to other slumps occurring in hydraulic earth-fill dams. (ENR, v. 122, no. 4, Jan. 1939) Failure due to shearing in foundation. (Justin, 1939)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Ft. Peck, Mont. (continued)							Slumping was preceded by settlement. A drain pipe was observed to be 1.5 ft (.45 m) lower than it should have been. Eyewitness report: Core pool began to settle, slowly then more rapidly, about same time, cracks were observed on upstream face 30 ft (m) below crest. Then upstream portions of upstream shell nearest pool began to slide in sinking core pool and some cracking and slumping took place on downstream beach. Simultaneously, the main mass of the upstream shell, almost intact was moving out into the reservoir. (ENR, v. 122, No. 19, p. 647)
Fourth Lake, Canada	1960	1961	Lessons, Table VI (1975)	22 7	E	A-2	Dam leakage. (Lessons, 1975)
Frankfurt, W. Germany	1976	1977	ENR, v. 199, No. 9	32			Break was preceded by a steep increase in ground water levels at the foot of the dam. Minutes before the break, water began to seep through the dam, eventually becoming a jet of water about 8 in. in diameter. Some small leaks were discovered two months prior to failure but investigation revealed no danger of failure at that time. (ENR, v. 199, No. 9)
Frazier, Ia.	1915	1935	Sherard, (1953) Middlebrooks, (1953)	28 8	E rolled		Seepage slide. (Middlebrooks, 1953) Failure due to progressive sloughing of the downstream slope due to a long period of a full reservoir. Sloughing cut back through dam until crest was breached, allowing full reservoir to pour through. (Sherard, 1953)
Frazier Valley, B.C. Canada		1948	ENR, v. 140 Middlebrooks, (1953)	12 4	E rolled		Overtopped. (Middlebrooks, 1953) Overtopping due to rapid snow melt and heavy rains. (ENR, v. 140, No. 7)
Fred Burr	1947	1948	Lessons, Table IV (1975)	53 16	E	F-2	Leakage, embankment piping. (Lessons, 1953)
French Landing, Mich.	1925	1928	ENR, v. 94, 1928	77 23	E	F-1	Undermining and washout of embankment during initial filling. began as leakage from toe. (ENR, v. 95, 1928)
Frenchmans Creek, Mont.	1951	1952	ENR, v. 118 Middlebrooks, (1953)		E rolled		Overtopped. (Middlebrooks, 1953) A flood stage exceeding all previous records resulting in a break in the natural ground around the spillway. Spillway capacity was suddenly exceeded and water cut into natural earth. (ENR, v. 140, No. 17)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD	CAUSE OF FAILURE
Fruit Growers Reservoir, Colo.	1898	1937	ENR, v. 118 Middlebrooks, (1953)	36 11	E rolled		Seepage slide. (Middlebrooks, 1953) Before the 1930's, the downstream slope frequently became moist and minor sloughs occurred. In 1937 water level reached highest level, to date, of 28 ft (9 m), a slide occurred on the downstream slope. The sliding material buried the downstream end of the outlet conduit, making it impossible to lower the reservoir. A stream of water poured from the slide area about 14 ft (4 m) below the crest. The earth in the upper portions of the slide was extremely wet and it was feared a second slide might occur. Part of the embankment was bulldozed out when the slide began to move again. The bulldozed cut was widened and deepened by flow and the reservoir was emptied. As the reservoir was drawn down, a drawdown slide occurred on the upstream slope directly opposite the slide on the downstream slope. This slide cut midway through the dam crest. (Sherard, 1953)
Furnace Brook Watershed 2, New Jersey	1971	1971	Lessons, Table IV and p. 179	54 16	E	A-3	Two joints between lengths of conduit opened, possibly due to yielding of foundation during construction. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Gatum, Panama		1912	ER, v. 66	115 35	H		Core pressure slide. (Middlebrooks, 1953)
Garza, Texas	1927	1926	Lessons, Table IV (1975) Middlebrooks, (1953) ENR v. 94 and 100	120 37 80 24	E-H H	DDC	Sliding embankment, upstream slope. (Lessons, 1975) Core pressure slide. (Middlebrooks, 1953) A problem occurred during construction with some of the hydraulic fill in building the dam. Material used was not suitable for 1:3 slopes. The clay used tended to ball up when pumped and after 10-15 days tended to squeeze together to form a solid mass. The hydraulic fill composed of clay balls was too soft to maintain ordinary slopes. The successive protective measures tended to flatten the slopes. (Nagle and Shuler, 1928)
Geary County St. Lake	1961	1971	Lessons, Table IV and p. 181 (1975)	56 17	E-R	MR	Riprap (limestone) destroyed by freeze-thaw cycles and wave action. (Lessons, 1975)
Gering Valley E. Neb.	1965	1965	Lessons, Table IV and p. 184	78 23	E	A-3	During filling transverse cracks were observed upstream slope; probably due to the differential settlement of the valley and abutment foundation material as they became fully saturated. (Lessons, 1975)
Gering Valley F, Nebraska	1965	1965	Lessons, Table IV and p. 185	80 24	E rolled	A-3	Transverse crack in upstream part of embankment near left abutment. Crack possible due to the differential consolidation of the foundation materials. (Lessons, 1975)
Germantown, Oh	1921	1971	Lessons, Table IV and p. 187 (1975)	100 30	E-H	MR	Concrete surfaces along edges and joints deteriorated. Upper portion of embankment placed with poor control. (Lessons, 1975)
Gilbert Run 2	1886	1913 1942 1963	Lessons, Table IV (1975)	50 15	E	A-2 A-1 A-2	Leakage, foundation. (Lessons, 1975) Flow discharge damage to spillway. (Lessons, 1975) Deformation total intake structure. (Lessons, 1975)
Golder	1964	1964	Lessons, Table IV (1975)	130 40	E	A-2	Leakage, foundation. (Lessons, 1975)
Goodrich	1860	1956	Lessons, Table IV (1975)	44 13		F-1	No known reason. (Lessons, 1975)
Goose Creek. S.Car.	1903	1916	Lessons, Table IV (1975)	210 64	R	F-1	Overtopping completed structure. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Goose Creek, S. Car	1903	1916	EN v. 76 p. 232 Middlebrooks (1953) Justin (1932)	22 7	E rolled		Overtopping. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) Washed out due to overtopping caused by heavy rains. (Justin, 1932) First overtopped 1912. Overtopped again in 1916 due to excessive rain. Early on Saturday the water began to overflow the spillways and run over the embankment. The supervisor tried to prevent overflow but by 7:30 PM when water was at its highest level and still rising and at a depth of 14 in. over the top of the dam, the downstream slope began to erode fast, and within one half hour, a breach was washed clear across the dam, which at that point was 50 ft (15 m) wide. The breach widened to 100 ft (30 m)wide and approximately 13.5 ft (4 m) deept. A cofferdam was laid on the upstream end of the breach and whole dam was covered with canvas, as incoming tide threatened temporary cofferdam. (EN, v. 76, No. 5)
Grand Rapids, Mich.	1874	1908	ER, v. 42 (7-14-1900) Middlebrooks, (1953) Justin, (1932) Hill (1902) Jorgensen, (1920)	25 8	E rolled clay core		Overtopping. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) Failure by overtopping. (Hill, 1902) (Jorgensen, 1920)
Grahm	1922	1923	Lessons, Table IV (1975)	112 34	E	F-1	Flow discharge, gate structure destroyed. (Lessons, 1975)
Grandview	1965	1960	Lessons, Table IV (1975)	80 24	E	DDC	Flow discharge damaged spillway. (Lessons, 1975)
Grassy Lake, Wy	1939	1940	Lessons, Table IV and p. 189	118 36	E	MR	The backfill behind the spillway walls became very saturated from seepage and run-off from abutment, causing excessive load on the walls combined with freezing during winter, pushed walls inward. (Lessons, 1975)
Great Western	1907	1958	Lessons, Table IV (1975)	61 19	E	A-1	Sliding embankment downstream slope. (Lessons, 1975)
Greenboth, G. Brit.	1962	1962	Lessons, Table VI (1975)	36 11	E	A-3	Dam, slide. (Lessons, 1975)
Greenlick, Pa.	1901	1904	EN, v. 52, p. 107 Middlebrooks. (1953)	60 18	E rolled	F-2	Seepage. (Middlebrooks, 1953)
			Lessons, Table IV (1975)		E rolled	S	Seepage embankment piping (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Greenville, S.C.		1927	ENR, v. 100, p. 750 ENR, v. 103, p. 934 Middlebrooks, (1953) Justin, (1932)	140 43			Castiron pipe failed. (Middlebrooks, 1953) Blow off pipe failed. (Justin, 1932)
Grosbois, France	1882 1900	1921	Sherard, (1953)	58 18	E		The dam suffered no trouble of any kind until 1921, when a "very important slide" occurred following a relatively rapid, complete reservoir drawdown. (Sherard, 1953)
Gros Vente	1925	1927	Lessons, Table IV (1975) Emerson, 1925	185 56	E	F-1	Overtopping completed structure. (Lessons, 1975) Formed as a result of a landslide, leakage occurred on filling. (Emerson, 1925) Natural dam created by a massive landslide. Due to heavy rains, dam was overtopped at a place where material was bad.
Guadalupe, Ca.	1935	1939 1944 1960	Lessons, Table IV and p. 191 (1975)	138 42	E Rolled concrete face	A-1	1939 cracks in concrete facing due to movement of slope; upstream 1944 additional movement cracking in June. 1944-1960 small periodic cracking and bulging. (Lessons, 1975)
Gunnison, Ca.		1890	EPG, Jour. v. 44 Middlebrooks, (1953) Hill, (1902) Jorgensen, (1920)	20 6	E rolled		Piping along an outlet. (Middlebrooks, 1953) Failure due to piping along outlet. (Hill, 1902) (Jorgensen, 1920)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Half Moon Bay, Ca.		1926	ENR, v. 96 Middlebrooks, (1953)	80 24	E rolled		Overtopping. (Middlebrooks, 1953)
Hamburg, Dike, W. Ger. 1976		1976	ENR, v. 197, No. 5				200 ft (61 m) breach in a new asphalt-lined canal dike south of Hamburg.
Harlan County	1952	1956	Lessons, Table IV (1975)	96 29	E	A-1	Slope protection, riprap too small. (Lessons, 1975)
Hatchtown, Utah	1908	1910 1914	Sherard (1953) Lessons, Table IV (1975) EN, v. 75 p. 60	65 20	E rolled puddle core	F-1	1910 seepage sloughs since first filling. (Lessons, 1975) Two days prior to failure, dynamite had been used to blast open a gate on a conduit. Too much dynamite was used causing cracking of the gate structure. (Sherard, 1953) Failure due to progressive piping along the only outlet conduit. (Justin, 1932) Destroyed 10-15 dams in its path before being stopped by another dam. (Jorgensen, 1920) Cause of break is not positively known, but seepage from the west hill might have saturated that part of the dam to the danger point. A new spring appeared in the west hill at an elevation of 40 ft (12 m) above the stream bed. It is possible this water caused a line of saturation downward and outward in the fill until it reached and overflowed the crest of the culvert. The culvert furnished a path to the river. (Jenson, 1914) Watchman noticed that slight seepage on bottom and south side on downstream end of outlet culvert increased and became cloudy. Causes of failure: 1) lack of cutoff collars on culvert; 2) defective foundations of culvert and core wall; 3) increase in tendency of water to creep along culvert due to manner and order of building and weather conditions during construction; 4) excavation, in the stratified sand and gravel underlying southerly part of site, of a borrow pit parallel and near upstream toe at a deep level, furnished an easy path from reservoir to body of the dam adding to already existing adverse conditions and; 5) the uncontrolled part of the water percolating so generally under north of base, woods tend to saturate embankment adjoining the culvert further reducing resistence to creep. (Engineering News, Jan. 1916). Undermining due to gravel layer under lava stratum. (Saville, 1916)
Hatfield, Wis.	1912	1920	Pence (1911)	92 28			Due to heavy rains and failure of the Dells dam this structure was also overtopped. (Pence, 1911)
Hebgen, Mont.	1915	1959	Lessons Table IV (1975) Sherard (1963) p. 164	123 37	E concrete core	A-1	Earthquake. (Lessons, 1975) Two types of damage associated with earthquake: 1) longitudinal cracks at top; 2) crest settlement. (Sherard, 1963)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Hebron, N. Mex.	1913	1914 1942	Lessons, Table IV (1975) ER, v. 69 p. 629 Middlebrooks (1953) Justin, (1932)	56 17	E rolled	F-1 F-2	1914 Piping through dam. (Middlebrooks, 1953) Piping due to gopher holes. (Justin, 1932) 1942 Overtopping completed structure (Middlebrooks, 1953) Leakage embankment piping (dispersive clay). (Lessons, 1975) 1913 After heavy rains, a concentrated leak developed near the outlet pipe, but did not increase with time. 1914 A heavy rain storm occurred over the whole of the large drainage basin. In the early morning hours, when the reservoir was still several feet below the crest, the water broke through the embankment. In a few hours the whole resrvoir capacity had spilled, and the break was cut 12 ft (4 m) below the original base of the embankment, and 200 ft (61 m) wide. A short distance from the breach a peculiar appearing hole opened through the dam at the same time as the failure. It was about mid-height of the dam, 10 ft (3 m) in diameter and thought to be an animal burrow. It was thought the piping that caused failure probably started in a similar hole. 1942 Dam was overtopped due to settlement and erosion of the crest. A 100 ft (30 m) section was completely washed out at this point. The dam had been repaired at writing. (Sherard, 1953) Water found its way through gopher holes and finally washed out a gap 200 ft (61 m) wide and 31 ft (9 m) high. (Jorgensen, 1920) Break was not witnessed until breach was quite large so the real cause is unknown. It is presumed that the water entered through a gopher, or other rodent hole, or a small crack, caused by settling of the embankment. Not being checked, the flow gradually enlarged the channel until a considerable volume found egress and rapidly cut and undermined the comparatively soft embankment, making a breach 200 ft (61 m) in width before the reservoir was emptied. There had been heavy rains prior to failure. (Case, 1914)
Heiwaike, Japan	1949	1951	Lessons, Table VI (1975)	20 6	E	F-1	Blowout, overtop. (Lessons, 1975)
Hemet, Ca.	1923	1927	ENR, v. 98	20 6	E puddled core	F-1	Overtopped due to heavy rains and flooding with subsequent breaching. (ENR, v. 98, 1925)
Hills Creek, Or	1962	1969-70	Lessons, Table IV and p. 195 (1975)	305 92	gravel with central core	A-1	Leakage, abutment; possible leakage through core or along the core foundation contact. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD	CAUSE OF FAILURE
Holmes Creek, Utah	1903	1924	Lessons Table IV (1975) Sherard, 1953	65 20	E rolled gravel core	A-1	Sloughing upstream slope; near full reservoir; thought to be caused by nearby explosion; sloughing downstream slope during construction. (Sherard, 1953). Sliding embankment, upstream slope. (Lessons, 1953)
Homme, N.D.	1951	1955-68	Lessons, Table IV and p. 208 (1975)	73 22	E	MR	Seeping water through cracks in concrete spillway or broken water stops eroded underlying filter, porous backfill and foundation material. (Lessons, 1975)
Hope Reservoir, RI	1882	1907	ER, v. 53 ER, v. 56 Middlebrooks, (1953)	23 7	E rolled		Seepage. (Middlebrooks, 1953)
Hornell, NY	1912	1912	EN, v. 58 Middlebrooks, (1953)		E concrete core		Seepage. (Middlebrooks, 1953) Partial failure in 1912, consisting of numerous leaks through the foundation. The leaks developed in spite of the fact that the core had been carried into the rock. (Jorgensen, 1920)
Horse Creek, Co	1912	1914	ER, v. 69 (2-14-1914) Hannah and Kennedy EN, v. 71, p. 828 Justin, (1932) Jorgensen, (1920) Saville, (1916) Hinderlider, (1914)	55 17	E rolled	F-2	Piping and sloughing; seepage under dam (Hanna and Kennedy, 1938) Failure due to the saturation of the rather loose fill in the downstream toe, causing a flow which undermined the outlet conduit. (Justin, 1932) Excessive seepage; cutoff not carried to rock, (Saville, 1916) Probably seepage under dam. 250 ft (76 m) went out. (Jorgensen, 1920) The dam had always leaked at the toe but never seriously. There was no warning of the break. Inspections a few days and even on the afternoon of failure gave no signs of weakness or failure. The theories given for cause of failure are: 1) water may have passed through one or more of the pervious strata or in the shale at or near the outlet conduit, causing a settlement and a failure of that structure; 2) water may have found its way through a pervious section of fill; 3) imperfect joint in the concrete gate tower may have permitted water to escape into the embankment around the tower, thereby causing the shower to settle and crack; 4) water may have found its way along the outside of the conduit, but this is hardly likely. (Hinderlider, 1914)
Horton, Kan.	1924	1925	ENR, v. 95 Middlebrooks, (1953) Black, (1925)	34 10	E rolled		Record breaking rainfall overtops structure and washes out 50 ft (15 m) partway down. (Black, 1925)
Hume, Australia	1936	1939	Lessons, Table VI (1975)	49 15	E	A-1	Dam, slide. (Lessons, 1975)
Hyland, Australia	1963	1963	Lessons, Table VI (1975)	24 7	E	A-1	Spillway breakage. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Kaddam, India	1957	1958	Lessons, Table VI (1975)	42 12	E	F-2	Blowout, overtop. (Lessons, 1975)
Kaila, India	1955	1965	Lessons, Table VI (1975)	26 8	E	F-2	Dam, slide. (Lessons, 1975)
Kanopolis	1948	1950	Lessons, Table IV (1975)	110 34	E	A-1	Leakage, foundation. (Lessons, 1975)
Karachunovsk, USSR	1950	1934	Lessons, Table VI (1975)	22 7	R	A-3	Surface, erosion. (Lessons, 1975)
Kedar Nala, India	1964	1964	Lessons, Table VI (1975)	20 6	E	F-2	Main dam, internal erosion. (Lessons, 1975)
Kelly Barnes Lake, Geo.	1940	1977	ENR, v. 199, No. 19	26 8	E		Exact cause of failure unknown. Sudden, almost total failure after heavy rains and severe flooding. (ENR, v. 144)
Kenray	1962	1962	Lessons, Table IV (1975)	55 16	E	A-1	Sliding embankment downstream slope. (Lessons, 1975)
Kern, Ore.	1948	1949	Sherard, (1953) Middlebrooks, (1953)	52 16	E rolled		Excessive settlement of fill. (Middlebrooks, 1953) caused by the rapid filling of reservoir and compaction of too dry embankment material during construction. (Sherard, 1953)
Kettering, England		1905	ER, v. 52 Middlebrooks, (1953) Jorgensen, (1920)	46 14	E rolled		Slide. (Middlebrooks, 1953) Settlement during construction; puddle core settled. (Jorgensen, 1920)
Kharagpur, India		1961	Lessons, Table VI (1975)	24 7	E	F-2	Blowout, overtop. (Lessons, 1975)
Killingsworth, Con.	1895	1938	ENR, v. 121 Middlebrooks, (1953)	18 5	E concrete core		Overtopping. (Middlebrooks, 1953)
Kingsley, Nebr.	1942 1941	1948 1942	Lessons, Table IV and p. 218 (1975 CE, v. 15	170 52	E-H	A-1	Slope protection; riprap too small. (Lessons, 1975) Loss of fill through concrete blocks, no filter. (CE v. 15)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Kingsley, Nebr.	1941		ENR, v. 120, p. 787 6-21-38 CE, v. 10, p. 623 10-1946 Engineer, v. 174, p. 46 7-17-42	158 42			This dam has a history of trouble with concrete face and riprap. It's located in an area known for its high winds and waves. (Lessons, 1975)
Kittaning Point	1877	1894	Lessons, Table IV (1975) EN, v. 31	50 15	E	A-1	Flow discharge, damage to spillway. (Lessons, 1975) Water flowed over the embankment for about 30 minutes. Within that time there was a sudden rise of water; supposed to have been caused by the wind, when for less than ten minutes the depth of overflow was about 1 foot (3 m) The first break began 26 ft (8 m) from the spillway and extended 90 ft (27 m), being about 8 ft (2 m) deep; the next break began 64 ft (20 m) further to the right, was 139 ft (42 m) long and about 6 ft (2 m) deep; then followed only 10 ft (3 m) of unbroken slope, succeeded by 77 ft (23 m) of break about 8 ft (2 m) deep; after this and another undamaged 10 ft (3 m) to where the final break occurred, about 64 ft (20 m) long and 10 ft (3 m) deep, terminating some 75 ft (23 m) from right end of main embankment. (EN, v. 31, June 7, 1894)
Kindbrook, Pa.		1894	EN, v. 32 Middlebrooks, (1953)		E rolled		
Knoxville Reservoir, Tn.		1883	EN, V. 47 Middlebrooks, (1953) Hill, 1902		E rolled		Foundation, seepage. (Middlebrooks, 1953) The puddle bottom of a double reservoir failed. (Hill, 1902)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Ingersol, Ontario Canada	1858	1837	EN, v. 17-18, p. 233		E		Dam had leaked every year during springtime. This time dam broke. Leaking began soon after construction. (EN, v. 17-18, p. 233)
Iron Bridge, Tex.	1960	1963	Lessons, Table IV and p. 209 (1975)	86 26	E	MR	Deterioration of limestone riprap of marginal quality. (Lessons, 1975)
Jackson Lake, Wyo.	1911	1973	Lessons, Table IV and p. 210 (1975)	68 21	E-G	MR	Severe freezing and thawing of concrete in sluiceway walls caused concrete to deteriorate. (Lessons, 1975)
Jeanette, Pa.		1903	ER, v. 48 (7-11-1903) Middlebrooks (1953) Justin (1932) Jorgensen, (1920)	20 6	E rolled		Overtopped. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) Spillway could not carry runoff; dam overtopped. (Jorgensen, 1920)
Jefferson County, Colo.		1897	EN, v. 47 Middlebrooks, (1953) Hill, (1902)				A small dam burst; miscellaneous or unknown cause. (Hill, 1902)
Jemey, N. Mex.	1953	1958	Lessons, Table IV (1975) Lessons, p. 99 (1973)	137 42	E	AR	Sliding abutment slope. (Lessons, 1975) A shallow slope failure occurred in the reservoir during drawdown. (Lessons, 1973)
Jennings Creek, Watershed No. 3	1962	1963	Lessons, Table IV (1975)	69 21	E	F-2	Leakage, foundation piping. (Lessons, 1975)
Jennings Creek Watershed No. 5	1962	1962	Lessons, Table IV (1975)	66 20	E	A-1	Leakage, foundation. (Lessons, 1975)
Jennings Creek Watershed No. 13	1962	1962	Lessons, Table IV (1975)	71 22	E	A-1	Leakage, foundation. (Lessons, 1975)
Jennings Creek Watershed No. 16	1960	1964	Lessons, Table IV (1975)	55 17	E	F-2	Leakage, foundation piping. (Lessons, 1975)
Jennings Creek Dam No. 17, Tenn.	1964	1965	Lessons, Table IV and p. 211 (1975)	77 23	E	A-1	Leakage, foundation, leakage through cavernous limestone foundation. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Jennings Creek Dam No. 18, Tenn.	1963	1965-69	Lessons, Table IV and p. 213 (1975)	97 29	E	A-1	Leakage, foundation; leakage through cavernous limestone foundation. (Lessons, 1975
Jewell Brook Watershed No. 2, Vermont	1969	1970	Lessons, Table IV and p. 214 (1975)	60 18	E	A-1	Upstream slope slid during drawdown. Slides occurred in areas that had been found below grade during construction were filled to grade and compacted. (Lessons, 1975)
Johnson, Neb.	1940	1942-45	Corps of Engineers Middlebrooks, (1953)	47 17	E rolled		Loss of filter through riprap. (Middlebrooks, 1953)
Johnstown, Pa.	1852	1889	ASCE Trans. v. 24 Middlebrooks, (1953) Justin, (1932) Lessons, (1973) ENR, v. 100 Jorgensen, (1920)	70 21	E rolled		Overtopping. (Middlebrooks, 1953); Due to insufficient spillway (Justin, 1932) Due to heavy rains the dam was overtopped. (ASCE Trans. v. 24, No. 477) A sag was left in middle of dam along with obstruction of spillway to prevent fish from entering spillway. Water level rose and went over crest at low place because spillway was inadequate. (Hill, 1902) (Jorgensen, 1920) The cause of failure was overtopping, contributed to by a lowering and disking of the crest, the closing of a pipe tunnel outlet and obstruction of the spillway. (ENR, v. 100, No. 12, p. 472) In 1862 a break in the culvert enclosing the sluice pipes caused great loss of water. Due to heavy rains, the run-off exceeded the capacity of the spillway; the dam was overtopped and breached. The breach was about 420 ft (128 m) wide at the top and 50 to 200 ft (15-61 m) wide at the bottom. The failure occurred rapidly. (Lessons, 1973)
John Zinc Ranch No. 2, Okla.	1972	1973	Lessons, Table IV and p. 216 (1975)	50 15	E	A-2	Sliding embankment, downstream slope; after filling of reservoir some seepage and sliding on downstream slope. (Lessons, 1975)
Julesburg, (Jumbo) Colo.	1905	1907 1910	ER, v. 63 Justin, (1932) Sherard, (1953) Middlebrooks, (1953)	70 21	E rolled	F-2	Serious leakage began 1907, dam failed with 24 foot head in 1910. (Middlebrooks, 1953) Failure is believed due to seepage of water along the underlying sandstone layers. No special precautions were taken to prevent seepage along the rock. There was neither cutoff trench or cutoff wall. These conditions combined with the fact that the line of saturation of this dam was very high, led to failure. (Justin, 1932) First stage of failure was lower toe of the embankment and 34 ft (11 m) below the surface. The underlying rock stratification was found to be of a very open and porous character, so much as to admit the passage of water from the reservoir under the base of the dam through the rock. This running water gradually washed out sand filling contained in the pockets and cavities and had reached such a depth that there resulted a hydrostatic pressure of sufficient amount to lift the overlying mass of rock and earth. The failure occurred at the point of least resistance to upheaval, or the toe of the lower slope. The dam had not been "keyed" into the foundation but was built from the surface. (ER, v. 63, No. 7)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Julesburg, (Jumbo) Colo.	(continued)						1906 Directly following the first filling there was considerable leakage appearing at the downstream toe. The leakage occurred in several streams along the dam. The largest occurred at a point where there had been outcrops of porous, friable limestone under the dam. 1910 A section of the west embankment about 400 ft (122 m), centered on the above leak, washed out completely. There had been no indication of unusual activity at this point on the previous day and events leading up to the washout were unobserved. The rock under the dam was seen in the walls of the break to be open and porous with solution cavities and channels up to 2 ft (.6 m) in diameter. Large blocks of the foundation rock, were lifted from the dam foundation and carried bodily a long distance downstream. (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURES
Lafayette, Ca.	1932	1938	Lessons, Table IV (1975)	100 30	E	DDC	Sliding embankment upstream and downstream slopes. (Lessons, 1975)
Lafayette, Ca.	1929	1928	Middlebrooks, (1953) ENR, v. 54	140 43	E rolled	DDC	Foundation Slide during construction. (Lessons, 1975) Failure due to subsidence of the foundation. The foundation acted as a plastic and flowed from region of greatest load to region of lowest load, causing subsidence in the region the soil flowed axay from. Foundation is thick alluvial deposit. (ENR, Jan. 31, 1929)
La Fruta, Tex.	1930	1930	ENR, v. 105, 106, 107 Middlebrooks, (1953)	61 19	E rolled		Foundation, piping. (Middlebrooks, 1953)
Lake, N. Mex.		1893	EN, v. 47 Middlebrooks, (1953)	52	E-R		Overtopped. (Middlebrooks, 1953)
Lake Avalon, N. Mex.		1894	EN, v. 35, 36, and 54 Jergensen, (1920) Middlebrooks, (1953) Murphy, (1905)	48 15	E rolled		Overtopped. (Middlebrooks, 1953) Failed by water forcing a passage through the dam, not by overflowing. (Jergensen, 1920) Failed by water forcing a passage through the dam. There are two opinions as to why the dam failed; 1) animals burrowed into the earth part of the downstream side and weakened the earth facing; 2) failure occurred near base. There had been leakage through base, prior to failure. Attempts to stop leak by sheet piling had only partial success. Cause was still unknown because failure occurred at night. (Murphy, 1905)
Lake Almanor	1927	1928	Lessons, Table IV (1975)	130 40	E	A-2	Leakage embankment piping. (Lessons, 1975)
Lake Bancroft, Va.	1913	1972	Lessons, Table IV and p. 224 (1975)	69 21	E-G	F-2	After excessive rainfall due to tropical storm. Rain fell on previously saturated soil, some of which had been rendered impervious due to development. Due to these conditions, reservoir level rose rapidly and structure was overtopped. (Lessons, 1975)
Lake Coedty, Wales		1925	ENR, v. 96 Middlebrooks, (1953)		E rolled		
Lake Dixie, Tex.		1940	ENR, v. 125 Middlebrooks, (1953)		E rolled		Overtopped (Middlebrooks, 1953) Due to heavy rains, 40 ft (12 m) gap was opened in the dam. (ENR, v. 125, Nov. 1940)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Lake Francis, Ca. (Old dam)	1899	1899	ASCE Trans., v. 59 v. 58, 1907, p. 140 Middlebrooks, (1953)	50 15	E rolled	F-2	Piping along an outlet due to settling and seepage. (Middlebrooks, 1953) A few days after completion heavy rains occurred. A great crack, caused by settling, appeared on the north end of the dam. This allowed water to escape through a constantly widening breach. The gap was 98 ft (30 m) wide on top and 30-40 ft (9-12 m) on the bottom. (ASCE Trans., v. 59)
Lake Francis, Ca. Old New	 1899	 1899 1903	ASCE Trans., v. 58, 1907 v. 59, p. 106 Middlebrooks, (1953) Justin, (1932) Lessons, Table IV (1975)	50 15 77 23	E rolled E	A-1	Piping along on outlet due to settling. (Middlebrooks, 1953) After heavy rainfall, a few days after completion, failure and breachment along settlement cracking. (Justin, 1932) Deformation, differential transverse embankment cracks. (Lessons, 1975)
Lake Francis, Ca.	1899	1935	Lessons, Table IV (1975)	77 23	E	F-2	Leakage, foundation piping. (Lessons, 1975) After heavy rains a large stream of water emerged from the toe of the embankment near the northern 36 inch cast iron outlet pipe. This was due to cracks in the pipe, evidently due to cracking. A few minutes after the leak at the outlet pipe was noticed, a stream of water broke through the crack near the right abutment. The water appeared on the downstream face of the dam about 20 ft (6 m) above the creek bed. This speedily grew in size, rapidly washing away the earth on the outer slope until the crest of the dam was observed to be sinking for a distance of 50 ft (15 m), and then the whole mass sloughed out with a rush. The failure was due to the embankment, which was constructed dry with very little compaction, settling considerably on initial reservoir filling. (Sherard, 1953)
Lake George, Colo.		1914	ASCE, Proc. v. 49 Middlebrooks, (1953) Jorgensen, (1920)		E puddled core		Piping. (Middlebrooks, 1953) Water found passage between the earthfill and original surface near one end (Jorgensen, 1920)
Lake Grahm, Tex.	1958	1958-64	Lessons, Table IV and p. 227 (1975)	83 25	E	MR	From 1958-1965 wave action caused sandstone riprap to crumble exposing embankment material. (Lessons, 1975)
Lake Malloya, N. Mex.	1914	1942 1955	Sherard (1953) Lessons, Table IV and p. 228, (1975)	50 15	E rolled	A-1	1942 Overtopped for 6 hours did not fail. (Lessons, 1975) 1955 Cast iron outlet pipe broke under downstream slope. (Lessons, 1975) 1952 Spillway capacity exceeded; dam overtopped to depth of 15 inches. Resulted in severe erosion of downstream slope, but dam was not breached. (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Lake Marie, Mo.	1880 1965	1972	Lessons, Table IV and p. 229 (1975)	50 15	Railroad fill and E	A-2	Old railroad fall converted into dam; in 1970 massive slide developed in downstream slope. Boils developed in the upper part of the slide mass. (Lessons, 1975)
Lake Orinda	1925	1962	Lessons, Table IV	48 15	E	MR	Erosion of outlet pipe due to chemical erosion. (Lessons, 1975)
Lake Palo Pinto, Tx.	1964	1966-68	Lessons, Table IV and p. 230 (1975)	63 19	E	A-1	Flow discharge damaged spillway. (Lessons, 1975)
Lake Patagonia, Az.	1968	1969	Lessons, Table IV and p. 231 (1975)	154 46	E	A-2	Shortly after first filling, large transverse cracks up to 2 in. (5 cm) in width developed through the dam above each abutment contact. A longitudinal crack in the crest of the dam is central part of the valley. The cracks are thought to have developed because of steep abutments, rapid reservoir filling and/or settlement of foundation and embankment materials. (Lessons, 1975)
Lake Toxaway, NC	1902	1916	ENR, v. 94 Lessons, Table IV (1975) ER, v. 74 EN, v. 76 p. 331	64 19	E rolled masonary core	F-1	Leakage, embankment piping. (Lessons, 1975) Early on the morning of failure, a small leak appeared at the base of the dam, which widened until complete failure. The exact cause is unknown but probable cause was that the bond of the core to the underlying bedrock was defective and that a pervious layer permitted piping to begin. (Justin, 1932) There had always been a small leak near the bottom just north of the stream bed. It was thought that the core wall, which was thick and very rigid, had been cracked by settling and movement. It is thought that due to heavy rains, the spillway had been overtaxed, exerting a higher pressure on the dam, which being weakened by the leakage, could not handle, resulting in failure. (EN, v. 76, No. 7) A small stream of spring at the foot of the dam, which had been running since the first filling, became larger, but remained constant. Seven days later the spring became muddy. That evening it began caving in and soon started giving away. About 270 ft (82 m) in the center went out. The break appears to have been caused by water flowing through seams or fissures in the rock 5 ft (1.5 m) above and 20 ft (6 m) to the left of the creek channel. The increased head in the lake, remaining so long, increased the flow of this stream, causing undermining at this point. (Willis, 1916)
Lake Waco	1930	1947	Lessons, Table IV (1975)	69 21	E	A-1	Flow discharge, damaged spillway. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Lake Yosemite, Ca.	1880	1943	Lessons, Table IV (1975) Sherard, (1953) Schuyler, (1908)	52 16	E rolled	A-1	Sliding embankment downstream slope. (Lessons, 1975) Sliding embankment due to saturation of downstream toe by seepage. (Shuyler, 1908) There had always been seepage under the dam. In 1943 a slide occurred on downstream slope. The slide was repaired. The slide was attributed to saturation of downstream toe by seepage and may have been due to local weak foundation area. (Sherard, 1953)
Lancaster, Pa.		1894	ER, v. 39 (2-8-1894) Middlebrooks, (1953) Hill, (1902) Jorgensen, (1920)	21 6	E rolled Puddled core		Piping along on outlet, (Middlebrooks, 1953) During first filling. Failure due to piping along an outlet, laid through the embankment. The water was seen to gush out from the outer slope of the embankment, and in an instant a break 30 ft (9 m) was made through the embankment. (Hill, 1902) (Jorgensen, 1920)
La Pay, Mexico	1974	1976	ENR, v. 197, No. 16	10 3	E		High winds and torrential rains from hurricane Liza destroyed the levee that was being used as a dam.
La Regader, Columbia	1938	1937	Lessons, Table VI (1975)	37 11	E	F-2	Main dam settlement. (Lessons, 1975)
Laural Run, Pa	1961	1977	ENR, v. 199, No. 4 (1977)	42 13			Overtopped and washed out. (ENR, v. 199)
Lebanon, Pa.	1884	1893 1919	ER, v. 27, p. 475 Middlebrooks, (1953) Justin, (1932) Jorgensen, (1920) EN, v. 67, No. 19	40 12	E rolled		Piping between fill and foundation. (Middlebrooks, 1952); foundation is porous sandstone, (Justin, 1932) A slip on the outer slope of some 140 cu. yards. This was due to softening by rain. (Jorgensen, 1920) 1919 A slip occurred, confined almost entirely on the outer slope of the brest. Before slip occurred there had been a heavy downpour of rain and frost had just disappeared. (EN, v. 67, No. 19)
Lebanon, Ohio		1882	EN, v. 9 Middlebrooks, (1953)	30 9			Failure by overtopping. (Hill, 1902) (Jorgensen, 1920)
Lee Lake	1923	1938	Lessons, Table IV (1975)	47 14	E	F-2	Flow discharge destroyed spillway. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Leroox Creek, Colo.		1905	EN, v. 54 (7-22-1905) Middlebrooks, (1953) Justin, (1932) Jorgensen, (1920)	25 8	E rolled		Overtopping, (Middlebrooks, 1953); due to insufficient spillway. (Justin, 1932) Inadequate spillway; dam overtopped. (Jorgensen, 1920)
Lidderdale, Colo.		1909	ASCE, Proc. v. 49 Middlebrooks, (1953) Justin, (1932)	79 6	E rolled		Overtopping. (Middlebrooks, 1953); due to insufficient spillway, (Justin, 1932) Due to flooding, dam was overtopped and a 100 ft (30 m) section was washed out. Failure result of insufficient spillway. (Justin, 1932)
Lima, Mont.		1894	EN, v. 31 EN, v. 47 Hill, (1902) Jorgensen, (1920)	40 12	E rolled		Erosion at spillway. (Middlebrooks, 1953) Spillway washed out; dam intact; insufficient spillway. (Hill, 1902)(Jorgensen, 1920)
Linville, NC	1919	1919	ASCE Trans., v. 84 Middlebrooks, (1953)	160 49	H		Core too flat. (Middlebrooks, 1953)
Little Deer Creek	1962	1963	Lessons, Table IV (1975)	85 26	E	F-1	Leakage, embankment piping. (Lessons, 1975)
Littlefield	1929	1929	Lessons, Table IV (1975)	125 38	R	F-1	Leakage, embankment piping. (Lessons, 1975)
Little Rocky Run, Illinois	1971	1971	Lessons, Table IV and p. 235 (1975)	59 18	E	A-2	Leakage foundation, winter of 1972 and 1973, seepage developed in the left abutment along the bedrock contact of through the overlying sandy and gravelly materials. (Lessons, 1975)
Little Wolfe, Il.		1935	Green, (1936)	40 12	E clay core		Due to heavy rains, inadequate spillway; settlement of core (and dam) due to non-uniform construction (core). Overtopping and breaching due to heavy rains and settlement. (Green, 1936)
Lock Alpine, Mich.		1926	ENR, v. 96 p. 242 Middlebrooks, (1953)	25 8	E rolled		Settlement on being saturated. (Middlebrooks, 1953) Failure due to shrinkage under a frozen crust from crest of dam. At the same time there was high water level in reservoir due to melting snow. There had been some shrinkage or settlement in dam where the rising water level found escape. Spillway and overflow shafts were at full capacity. (ENR, v. 96, 1926)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Logan Martin, Ala.	1964	1964	Lessons, Table IV and p. 237 (1975) Grant, (1966)	97 30	G-E	A-2	Leakage foundation. (Lessons, 1975) On filling a small amount of seepage was noted as reservoir level reached the top of the power pool. Seepage increased during the next three years, and then more or less stabilized. Failure possibly due to the erosion of natural fill in solution channels. (Grant, 1966)
Lone Pine Reservoir	1936	1936	Lessons, Table IV (1975) Lessons, p. 100 (1973)	101 31	E-R	AR	Leakage foundation. (Lessons, 1975) Bottom is composed of jointed basalts, permeable sandstones and limestones with sinkholes and interconnected openings plus possible salt beds at depth. Project has been abandoned. (Lessons, 1973)
Long Tom, Id.	1906	1916 1915	Lessons, Table IV (1975) Sherard, (1953)	60 18 50 15	E E puddle core	A-1	Deformation total tunnel. (Lessons, 1975) Collapse of a section of roof or walls of outlet tunnel caused by water flow through a rodent burrow or natural flow of water along a rock face. (Sherard, 1953)
Lanqwalds Pond, Mas.	1910	1922	Latimer, (1922) ENR, v. 89 (7-20-1922) p. 121 Middlebrooks, (1953)	30 9	E concrete core	F-1	Failure due to undermining of core wall by leaks or springs. (Latimer, 1922) Piping. (Middlebrooks, 1953)
Lon Hagler, Colo.	1967	1969-70	Lessons, Table IV and p. 241 (1975)	65 19	E	MR	From 1969-70 deterioration of schist riprap due to freeze-thaw cycle and wave action. (Lessons, 1975)
Lookout Shoals	1915	1916	Lessons, Table IV (1975)	83 29	E	F-2	Overtopping completed structure. (Lessons, 1975)
Lower Hell Hole	1966	1964	Lessons, Table IV (1975)	.410 125	R	DDC	Overtopping during construction. (Lessons, 1975)
Lower Otay, Ca.	1886	1916	Lessons, Table IV (1975) EN v. 15 p. 334 Middlebrooks, (1953) EN, 2-3-1916 2 17-1916 Justin, (1932) Silent, 1916	130 40	E-R	F-1	Overtopping; insufficient spillway; completed structure. (Lessons, 1975) R and concrete core. (Middlebrookes, 1953). Steel core. (Justin, 1932) Due to heaviest rainstorm ever recorded at that time, dam was overtopped. Failure started after overflowing crest, washed out backing of central core. Break started near center of dam. Structure was rock fill with steel diaphram. (Baker, 1916; unprecedented rainfall gave a runoff that overtopped this poorly designed and built rock-fill dam, dependent on a rivited steel plate diaphram for water tightness. In a few minutes the downstream portion of the rock-fill melted away, then the steel diaphram was torn from the top downward and the remainder of the dam opened like a pair of gates. (Silent, 1916)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Lower Otay, Cal. (continued)							Due to heavy flooding, the dam was overtopped, removing the stone fill; after a few minutes the core wall burst. The lack of sufficient spillway capacity was the cause of failure. (Justin, 1932)
Lower San Fernando, Cal.	1915-1920	1971	Lessons, Table IV and p. 367 (1975) Golze, (1977)	142 43	E-H	A-1	Stability earthquake. The February 9, 1971 earthquake caused liquefication of a portion of the old hydraulic fill, resulting in a massive upstream slide. (Lessons, 1975) Front of dam failed as a landslide. (Golze, 1977)
Lyman, Ariz.	1913	1915	EN, v. 73 p. 794 Justin, (1932) Lessons, Table IV (1975)	65 20	E rolled puddled core	F-2	Piping; embankment and sloughing. (Justin, 1932) Leakage embankment piping. (Lessons, 1975) Failure began as a rush of water coming out of the base in the center of the channel. The water on downstream side of the dam suddenly rose 30 ft (9 m) and a section of the dam was cut out having a span of about 75 ft (23 m). This quickly collapsed, and the gap widened to about 350 ft (107 m). (Justin, 1932) The dam had just been inspected prior to failure. No evidence of cracking, settling or seepage could be detected. The first evidence of break was a sudden rush of water coming from the base of the dike in the center of the channel. The accepted theory of failure is that the portion of the dike across channel below the outlet conduit did not have an opportunity to dry out and properly settle; consequently, this section was materially weak and gave out when the pressure became too great. (The dam was constructed in two sections, one across the channel while river was at low water, the other (above conduit level) after conduit level was reached). As the portion of the dike across the channel was built against an increasing pressure of water it was no doubt saturated to the core wall and above the outlet conduit by means of capillary attraction, which destroyed the bond. (EN, v. 73, no. 16)
Lynde Brooks, Mas.		1876	EPG, Jour. v. 44 Saville, (1916) Middlebrooks, (1953)	27 8	E rolled mason faced core		Piping along an outlet. (Middlebrooks, 1953) Stratum of porous material under gatehouse. (Saville, 1916)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Madison, Mont.	1908	1970	Lessons, Table IV and p. 244 (1975)	47 14	R	MR	Intake structure and right abutment concrete wall strengthened. (Lessons, 1975)
Magic, Idaho	1910	1911	Lessons, Table IV (1975) Middlebrooks, (1953) Sherard, (1953) ER, v. 60	130 40	E rolled and hydraulic	A-2	Leakage embankment, (Lessons, 1975); piping through dam, (Middlebrooks, 1953); a short time after reservoir filled for first time, muddy seepage appeared downstream slope. A hole appeared shortly thereafter in this spot (approximately 14 ft (4.2 m) diameter and 8 ft (2.4 m) to 10 ft (3 m) deep), with general movement of embankment material toward slump. Shortly after this slump developed two additional muddy leaks appeared on downstream slope at approximately the same elevation. These were nearly evenly spaced between original leak near center of dam and right abutment. Similar slumps appeared above these leaks after 24 hours. The middle slump had settled 57 ft (17 m) in two months, almost to the original foundation. The leaks which emerged from downstream slope at first filling indicate a probable flow of water between hydraulic fill and folled fill sections. (Sherard, 1953)
Mahonoy City, Pa.		1892	EN, v. 27 v. 47 ER, v. 26 Middlebrooks, (1953) Hill, (1902)		E rolled		Piping. (Middlebrooks, 1953) Cause of failure unknown. Dam had been under repair. (Hill, 1902)
Mammoth, Utah	1908	1917	ENR, v. 79 ER, v. 66 Lessons, Table IV (1975)	70 21	E and hydraulic	F-1	Flow discharge, destroyed spillway; overtopped during construction. (Lessons, 1975 Watchman returned from dinner to find dam breached; no previous warning; spillway and outlet tower incomplete; water found its way around or under upper end of log flume (temporary spillway) which had not been seated properly in the earth fill. Water filled the depression between dike and heart wall. This water saturated the earth-fill sufficiently to overturn or push out by sliding a portion of the heart wall. Reservoir level had not changed; dam failure due to lack of spillway and unsafe height that reservoir had attained combined with imcomplete and inadequate outlet tower. (Kleinschmidt, 1917)
Maquoketa, Io.	1924	1927	ENR, v. 98 Middlebrooks, (1953)	20 6	E rolled	F-1	Piping at junction with concrete spillway. (Middlebrooks, 1953) Cause unknown; there had been heavy rains and embankment had been saturated. (ENR, v. 98, 1925)
Marion County	1938	1938	Lessons, Table IV (1975)	55 17	E	A-2	Leakage foundation. (Lessons, 1975)
Marshall Creek, Kan.		1937	ENR, v. 119 Middlebrooks, (1953)	80 24	E rolled	DDC	Foundation failure during construction. (Middlebrooks, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Marshall Lake, Colo.	1909 1908	1908 1909	Lessons, Table IV (1975) Middlebrooks (1953) ER, v. 62	86 26 70 27	E rolled	DDC	Deformation. (Lessons, 1975) Seepage. (Middlebrooks, 1953)
Marston	1911	1925	Lessons, Table IV (1975)	60 18	E	A-1	Deformation, differential transverse embankment cracks. (Lessons, 1975)
Marie Gomey, Mexico	1946	1943	Lessons, Table VI (1975)	49 15	E	A-2	Cracking. (Lessons, 1975)
Martin Davey Dam, Tex.		1940	ENR, v. 125 Middlebrooks, (1953)		E rolled		Overtopped. (Middlebrooks, 1953) Due to heavy rains, a gap 250 ft (76 m) was opened in the dam. (ENR, v. 125, Nov. 1940)
Masterson, Oregon	1950	1951	Lessons, Table IV (1975) Sherard (1953)	60 18	E-R rolled earth	A-2	Deformation, differential transverse embankment cracks. (Lessons, 1975) During heavy rainstorm with rapid water rise, crest of embankment settled four feet (1.2 m) near the center. At the same time, an oval shaped tunnel opened through the embankment at the high water elevation. After openeing of the tunnel, water poured through the embankment. Slumps occurred on downstream slope below tunnel opening. Cracks and tunnel healed themselves by sloughing. (Sherard, 1953) The dam was just completed and was unused for two months. During this time no cracking or settlement was noticed. During a rainstorm, the water rose rapidly to within 17 ft (5 m) of the dam crest. As the reservoir rose the crest of the dam settled a total of 4 ft (1 m) near the center. This severe settlement caused two cracks to open across the crest and run diagonally down the upstream slope approximately parallel to the abutments. The cracks were open a maximum of 5 inches across the crest, and 3 to 4 inches down the upstream slope. At the same time, an oval-shaped tunnel opened through the embankment at the high water elevation. After the tunnel opened, water poured through the embankment and the reservoir ceased to rise. The water pouring through the tunnel entered the downstream pervious blanket and cascaded down the lower zone composed of heavy-dumped quarried rock. Slumps occurred in the downstream slope above the tunnel. The material from these slumps undoubtedly droped vertically into the tunnel and was carried away. After water level was lowered, the cracks on the upstream slope healed themselves off below the water level. The tunnel gradually sloughed off and closed up. Failure is attributed to the fact that the dam was built of alternating layers of compacted and loose material. When water came in contact with dry material, settlement occurred, while the compacted material remained in place. This differential settling forming passage ways for water to penetrate the dam. (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
May Dam, Turkey	1960		Lessons, p. 100 (1973)	92 28	E	AR	The bottom of the reservoir consists of alluvium, karstic limestone, conglomerate and Marl. Water seeps through the alluvium and forms sinkholes. (Lessons, 1973)
McCloud, Ca.	1965	1964	Lessons, Table IV and p. 242	230 76	E-R	DDC	Overtopping cofferdams during construction due to heavy rains. (Lessons, 1975)
McCloud, Ca.	1965	1972	Lessons, Table IV and p. 242	230 70	E-R	MR	Flow discharge, slide gate failed. (Lessons, 1975)
McMahon Gulch	1924	1926	Lessons, Table IV (1975)	55 17	E	F-2	Overtopping completed structure. (Lessons, 1975)
McMillian, N. Mex.	1893	1915 1937	Sherard, (1953) Lessons, (1975) Schuyler (1908)	57 17	E&H	A-1	Leakage, embankment. (Lessons, 1975) Upstream piped into rick downstream. (Shuyler, 1908) In 1915 water broke through the thin earth upstream section about 11 ft (3 m) below the crest. The leakage emerged from a width of about 150 ft (46 m) at the downstream toe. The water quickly eroded a large hole at the high water level which was quickly filled in with sandbags. In 1937 a similar break occurred, and was even mo:e extensive. Two days were spent sandbagging the whole length of the dam and failure was averted. (Sherard, 1953)
Meeks Cabin, Wy.	1971	1969	Lessons, Table IV and p. 246	174 53	E-R	DDC	Settlement and spreading of conduit foundation occurred, opening joints up to 9 in. (22.86 m). (Lessons, 1975)
Melzingah, NY		1897	ER, v. 36 (7-17-.897) Middlebrooks, (1953) Justin, (1932) Hill, (1902) Jorgensen, (1920)	24 7	E rolled mason core		Overtopping. (Middlebrooks, 1953), due to insufficient spillway. (Justin, 1932) Failure by overtopping; a freshet flowing over crest of both dams. (Hill, 1902) (Jorgensen, 1920)
Mica dam and Res.	1973	1973	Lessons, p. 99 (1973)	800 244	E	AR	The dam was under construction at writing (1973) and it was thought at that time that following saturation of the rock mass some movement may develop but it was not expected to be of large magnitude. (Movement confined to reservoir bank). (Lessons, 1973)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Middlefield, Mass.		1901	ER, v. 43 (5-4-1901) Middlebrooks, (1953) Justin, (1932)	20 6	E rolled		Overtopping. (Middlebrooks, 1953); due to insufficient spillway. (Justin, 1932) Failure by overtopping. (Hill, 1902)(Jorgensen, 1920) At one end of the dam was a wasteway, controlled by "two series of gates, one above the other" over which there was a bridge. The gates were partly open, when it was discovered that the water was overflowing the adjoining roadway. Access to the gates was impossible. The dam gave way in the center, but the break was somehow slow. (EN, v. 45, No. 17)
Mill Creek	1899	1957	Lessons, Table IV (1975)	67 20	E	F-2	Leakage, foundation piping. (Lessons, 1975)
Mill Creek	1941	1941 1945	Lessons, Table IV (1975)	145 44	E	A-2	Leakage, foundation. (Lessons, 1975) Chemical damage, corrosion outlet pipe-leakage, piping embankment. (Lessons, 1975)
Mill River, Mass.	1865	1874	ASCE Trans, v. 3 and 4 Justin, (1932) Hill, (1902) Jorgensen, (1920) Saville, (1916)	43 13	E concrete rubble core		Seepage. (Middlebrooks, 1953) Water found its way under the core wall and destroyed the embankment. (Hill, 1902) (Jorgensen, 1920) 1874 Layer of course gravel; core wall not carried to rock; no inspection during construction and improper construction. (Saville, 1916) No compaction or engineering supervision was used on the dam. As a result when water level was raised, embankment became saturated and failed. (Justin, 1932)
Millville, Utah	1907	1909	ER, v. 60 p. 324 Middlebrooks, (1953)	36 11	E rolled puddle core		Piping through foundation. (Middlebrooks, 1953) Failure due to foundation failure. Foundation underlying part of the dam's described as quicksand and "slush material." There were supposed to be 16 ft (5 m) of sheet piling driven into the foundation, but actually only received 9 ft (3 m). Before failure there was some seepage through the downstream toe in an old river channel. It appears that the failure was a blowout, and that the soft, saturated foundation soil was forced to flow downstream, starting the disaster. (Justin, 1932)
Mineral Wells, Tex.	1919 1943	1970	Lessons, Table IV and p. 249 (1975)	71 22	E	A-1	Repeated use of spillway caused erosion and flood of 1970 demolished the lower portion of spillway chute. (Lessons, 1975)
Mission Lake, Kan.	1924	1925	ENR, v. 95		E rolled		Settlement with overtopping. (Middlebrooks, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Modesto Irrigation No. 1, Ca.	1909	1927	Holmes, 1927	32 10	E puddled core		Leakage through dam due to improper construction of core with puddling. (Holmes, 1927)
Mohawk, Ohio		1913 1915	EN, v. 73 Middlebrooks, (1953) Jorgensen, (1920) Ruhling, (1915)	18 5	E rolled		Seepage. (Middlebrooks, 1953) Wooden spillway too small, and flood water washed out large section about midway between banks. When dam was rebuilt reinforced concrete was given faces and top. In 1915 earthfill settled and concrete lining gave way. Water again rushed out. (Jorgensen, 1920) Damaged by floods, 1913, due to inadequate spillway; repaired. Whole dam was concrete covered to make spillway of complete structure. During repair not much compaction had been done. After concrete facing was added, settlement of earth caused cracking of face. Face was built up with concrete. More settling occurred and by winter horizontal cracks appeared and dam began to leak. This eroded the new fill (repaired section) which in turn caused the cracks to widen. After several days of freezing followed by warm rains, facing completely ruptured and water broke through in large mass. (Ruhling, 1915)
Montpelier Creek, ID	1969	1971	Lessons, Table IV and p. 251 (1975)	82 25	E-R	A-2	Leakage foundation; during first filling, seepage developed in the left abutment. Lesser quantity of seepage emerged from right abutment. Wet areas and small boils developed in the valley at the downstream toe. (Lessons, 1975)
Montreal, Quebec, Canada		1896	EN, v. 47 Middlebrooks, (1953)	18 5	E-R		Seepage. (Middlebrooks, 1953)
Morena, Ca.	1912		ASCE Trans, v. 65 Middlebrooks, (1953)	107 51	R		Overtopped, did not fail. (Middlebrooks, 1953)
Mount Lake State Park, Minn.	1937	1938	ENR, v. 120 Middlebrooks, (1953)				Overtopped. (Middlebrooks, 1953)
Mount Pisgah, Colo.	1910	1928	Lessons, Table IV (1975) Sherard, (1953)	76 23	E	A-1	Sliding embankment upstream slope. (Lessons, 1953) Rapid drawdown failure due to seepage or leakage from outlet tubes causing complete saturation of the inner and lower portions of dam; after a quick drawdown; steep concrete convered slope did not drain quickly enough and slide occurred. (Sherard, 1953)
Mountain Creek, Tx.	1931		Corps of Engrs. Middlebrooks, (1953)	36 11	E rolled		Loss of filter through riprap. (Middlebrooks, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Mud Pond, Mass.	1873	1886	ER, v. 13 Middlebrooks, (1953) Justin, (1932) Hill, (1902) Jorgensen, (1920)	15 5	E-R		Piping, (Middlebrooks, 1953) due to poor construction. (Justin, 1932) Dam was said to have been poorly constructed. (Hill, 1902) (Jorgensen, 1920)
Mud Mountain, Wash.	1948	1969	Lessons, Table IV and p. 254 (1975)	425 130	R	A-1	Flow discharge, trash rack failed, possible due from pounding from bed load of builders in river at tunnel intake (invert at riverbed elevation). (Lessons, 1975)
Murayamakami, Japan	1924	1923	Lessons, Table VI (1975)	24 7	E	A-1	Horizontal movement of main dam due to earthquake. (Lessons, 1975)
Marayamashi, Japan	1927	1923	Lessons, Table VI (1975)	30 9	E	A-3	Cracking due to earthquake. (Lessons, 1975)
Murray Gill Res. Kan.	1965	1967	Lessons, Table IV and p. 257 (1975)	77 23	E	MR	A landslide developed 250 ft (76 m) downstream of axis of the dam. Probable cause reservoir seepage through the porous limestone and water seepage along the overburden shale contact. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Nacimiento, Ca.	1959	1969	Lessons, Table IV and p. 259 (1975)	270 82	E	A-1	After heavy rains, high level outlet slide gate clogged, causing flow discharge in turn causing erosion of dam. (Lessons, 1975)
Narraquinnep, Colo.	1906	1926-51	Lessons, Table IV (1975) Sherard, (1953) Middlebrooks, (1953)	79 24 97 30	E rolled	MR	Sloughing of upstream slope began after first filling; repaired 1946, some leakage right abutment due to channels in abutment rock. (Mid-height); some leakage downstream toe, center of dam. (Sherard, 1953) 1925 Sliding embankment upstream slope. (Lessons, 1975) 1951 Sliding embankment upstream slope. (Lessons, 1975) Sliding embankment, upstream slope. (Lessons, 1975) Continued sloughing of upstream slope and abutment leakage. (Middlebrooks, 1953)
Navajo, N. Mex.	1963	1964	Lessons, Table IV and p. 262 (1975)	404 123	E	MR	Flow discharge; damage to outlet stilling basin floor, wedges and walls. (Lessons, 1975)
Nebraska City, Neb.	1890	1890	EN, v. 47 Middlebrooks, (1953) Hill, (1902) Jorgensen, (1920)	17 5	E-R		Seepage. (Middlebrooks, 1953) Two new reservoirs failed, shortly after being put to use. Soil was porous and to prevent the percolation of water the embankments and bottom were lined with 2 in. plank which was covered with 1 ft (.3 m) of earth. (Hill, 1902) (Jorgensen, 1920)
Necaxa, Mexico	1909	1909	Lessons, Table VI (1975) EN, v. 62, Justin, (1932) ER, v. 60 p. 1 Middlebrooks, (1953) Jorgensen, (1920) ENR, v. 80 Schuyler, (1909) Justin, (1932)	99 18 190 58	E H clay core	DDC	Dam slide. (Lessons, 1975); slide in embankment during construction, (Middlebrooks, 1953) Sloughing during construction. (Justin, 1932) Soft clay core bulged out before dam was finished. (Jorgensen, 1920) Failure in hydraulic dam due to excessive core pressures. The core has high pressure because it is saturated in center. There is a need for water pressure on the dam to equalize internal pressure. (ENR, v. 80, No. 15) There had been a drought prior to failure with sudden rise in water level. A summary of the possible causes of failure are: 1) because of drought the water level was low, because water level was low there was no pressure on the dam to conteract the hydrostatic pressure within the dam; 2) the peculiar quality of the clay, which did not harden in the center, although making both side embankments impervious to water, 3) the narrow crest width of the upstream rock-filling, due to the greater length of that side, and the constant difficulty of keeping the flumes built sufficiently in advance of the work, 4) the use of tepetate, having specific gravity much lower than limestone. This combined with the narrow crest width which existed at the time of the break, where much clay must have been deposited dangerously near the edge, and directly in line of the final break, might have been the most important of all the contributory causes. (Schuyler, 1909) Partial failure due to the uncounterbalanced internal pressure of the plastic, clay core. (Justin, 1932)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Necomah, Wis.		1905	ER, v. 52		E concrete core		Core settled. (Middlebrooks, 1953)
New Bedford, Mas.	1866	1868	ASCE Trans., v. 1 and 2 Middlebrooks, (1953)		E puddled core		Piping along outlet conduit. (Middlebrooks, 1953)
New Bowman, Ca.	1927	1926	ENR, v. 54 Middlebrooks, (1953) Tibbets, (1929)	170 52	R		Break in outlet tunnel, repaired. (Middlebrooks, 1953) Breaks in outlet tunnel due to rock pockets (areas of pressure greater than could be withstood without support). Discovery by excessive leakage and dewatering. (Tibbetts, 1929)
New Don Pedro, Ca.	1971	1969	Lessons, Table IV and p. 265	580 178	E-R	DDC	Due to extremely heavy precipitation, cofferdam was overtopped. (Lessons, 1975)
New Exchequer, Ca.	1967	1972	Lessons, Table IV and p. 266 (1975)	479 146	R concrete face	MR	Slope protection concrete slab badly cracked due to vertical settlement and lateral movements of the rockfill. (Lessons, 1975)
Nishne Tulomskaya, USSR	1938	1936	Lessons, Table VI (1975)	64 20	R	A-3	Dam, slide. (Lessons, 1975)
Nizhne Svirskaya, USSR	1935	1935	Lessons, Table VI (1975)	28 9	E	F-2	Dam, slide. (Blasting). (Lessons, 1975)
North Dike, Wachusset, Mass.	1905	1907	Merriman, (1930) Middlebrooks, (1953)	82 25	E rolled		Slide in upstream slope. (Middlebrooks, 1953)
North Hartland Lake	1961	1969	Lessons, Table IV and p. 270, (1975)	190 58	E	AR	Slides occurred in natural slopes in the reservoir occured during flood control operations with no significant effect on reservoir or dam. Some slides took place as water level rose. Material is water lain fine sands and silts. (Lessons, 1975) Embankment, upstream slope. (Lessons, 1975)
North Scituate, RI		1926	ENR, v. 96 Middlebrooks, (1953)	6 2	E rolled		Overtopped. (Middlebrooks, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
North Springfield Lake, Vermont	1960	1969	Lessons, Table IV and p. 273 (1975)	126 38	E	A-1	Leakage, foundation; leakage downstream toe of embankment right bank. (Lessons, 1975)
Norwich, NY upper Sherburne, lower	1892	1905	EN, v. 54 Middlebrooks, (1953) Justin, (1932)	34 10	E rolled		Overtopped. (Middlebrooks, 1953). Insufficient spillway. (Justin, 1932) Upper dam has puddled core. (Justin, 1932) Two dams. Upper dam overtopped by flood and washed out. Lower dam partially washed out. (Jorgensen, 1920)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURES
Ogayarindo, Japan	1944	1963	Lessons, Table VI (1975)	19 6	E	F-2	Blocking, water discharge. (Lessons, 1975)
Olive Hills	1962	1963	Lessons, Table IV (1975)	100 30	G-E	AR	Sliding reservoir slope. (Lessons, 1975)
Uno, Japan	1913	1923	Lessons, Table VI (1975)	49 15	E	A-1	Cracking; earthquake. (Lessons, 1975)
Untelaunee, Pa.		1936					Newly completed structure; water seeping through fundation (cavernous limestone) and through upstream slope. Began after heavy rains. Caverns were below existing grout curtains. Remedial Measures: 1) blanketing upstream area and 2) extend grout curtin under dam.
Oros, Brazil		1960	Lessons, Table VI (1975) Lessons, (1973)	54 16	E-R clay core	A-3	Blowout, overtop. (Lessons, 1975) Due to heavy rains and the fact that construction had been delayed, dam was overtopped. The crest was to be raised to 656 ft (200 m), but due to delays in construction it had been raised to 623 ft (190 m), when heavy rains fell. The dam was overtopped on its entire crest of 2034 ft (620 m) and the depth of overflow was .3 m. Due to overtopping the dam was breached and the width of breach was 656 ft (200 m). (Lessons, 1973)
Otanifke, Japan	1920	1946	Lessons, Table VI (1975)	27 8	E	A-1	Cracking; earthquake. (Lessons, 1975)
Otter Brook	1958	1958	Lessons, Table IV (1975)	138 42	E	DDC	Sliding embankment, upstream slope. (Lessons, 1975)
Otter Creek Watershed, No. 9, Wis.	1970	1970	Lessons, Table IV and p. 277	60 18	E	A-2	During initial reservoir filling seepage noted right abutment. Area downstream of this abutment very wet preventing vegetation. (Lessons, 1975)
Ouachita River. La.			ENR, v. 79, no. 11				Break due to sandy bottom of river. (not sure if this dam is earth) (ENR, v. 79, No. 11)
Ovcar Banja, Yugo.	1952	1965	Lessons, Tabke VI (1975)	27 8	E	F-2	Blowout, overtop. (Lessons, 1975)
Owen	1915	1914	Lessons, Table IV (1975)	57 17	E	DDC	Leakage, embankment. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Palisades, Id.	1957	1964	Lessons, Table IV and p. 281 (1975)	258 79	E	MR	Extensive cavitation damage developed downstream of the gates. (Lessons, 1975)
Panshet, India	1961	1961	Lessons, Table VI (1975)	49 15	E	F-2	Cracking. (Lessons, 1975)
Paris	1940	1939	Lessons, Table IV (1975)	57 17	E-R	DDC	Overtopping, structure under construction. (Lessons, 1975)
Park Reservoir, Wyo.	1909	1909	Lessons, Table IV and p. 282 (1975)	80 24	E rock fill toe	A-1	Sliding embankment upstream and downstream slopes. (Lessons, 1975)
Peapack Brook, NJ		1926	ENR, v. 100 p. 116 Middlebrooks, (1953) Justin, (1932) Critchlow, (1928)	32 10	E rolled concrete core	DDC	Overtopped. (Middlebrooks, 1953) during construction. (Justin, 1932) Failure due to slump on upstream slope because of rising water level after slump, support of center portion of concrete core was removed. The concrete core was then overturned and badly shettered by the impact and subsequent rush of water. (Crtichlow, 1928)
Pecos Eddy, NM	1890	1893	EN, v. 31	45 14	R		An article saying the dam had broken. No details given. (EN, v. 31)
Penn Forest, Pa. 1	1958	1960	Lessons, Table IV and p. 285 (1975) ENR, v. 173, 1964, R33	145 44	E rolled	A-2	When pool was approximately 15 ft (4.5 m) below spillway crest, water was observed seeping out of west bank. (April, 1960); May 1960, when reservoir was 4.5 ft (1.4 m) below spillway crest, A 16 x 18 x 7 ft (4.8 x 5.4 x 2.1 m) sinkhole developed. (Lessons, 1975) Has a problem with leakage through or under the dam when reservoir is at high levels. (ENR, v. 173, No. 24, 1964, p. 33)
Penn Forest, Pa. 2	1959	1960	Lessons, Table IV (1975)	170 52	E	A-2	Leakage, embankment piping. (Lessons, 1975)
Phoenix Lake, Ca.	1908	1968	Lessons, Table IV and p. 288 (1975)	88 27	E	MR	Upstream impervious blanket and downstream pervious blanket and berm added. Existing riprap removed and replaced over newly constructed impervious blanket. (Lessons, 1975)
Piedmont No. 1, Ca.	1903	1905	Lessons, Table IV (1975) Sherard, (1952)	53 16	E concrete core	A-2	Deformation total conduit. (Lessons, 1975) Outlet pipe sheared off at core wall due to settlement of embankment. (Sherard, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Pilarcitos, Ca.	1866 1874 (raised)	1969	Lessons, Table IV and p. 291 (1975)	95 29	E puddled core	A-1	Sliding embankment, upstream slope; due to drawdown. (Lessons, 1975)
Pleasant Valley, Ut.	1927	1928	Lessons, Table IV (1975) Sherard, (1953) Middlebrooks, (1953) ENR, v. 100 p. 826	78 24 63 19	E-R Earth & rock	A-2	Deformation, differential transverse embankment cracks. (Lessons, 1975) Piping through settlement cracks. (Middlebrooks, 1953) In the first few years after construction, a shallow slide occurred on the downstream slope near the center of the dam. An oval shaped area was involved, extending from the downstream edge of the crest to about mid-height, with a width, parallel to the axis, of about 25 ft (i m). It is doubtful if the embankment was affected more than 3-4 ft (.9 - 1.2 m). It is believed to have been caused by saturation by rain and/or melting snow. (Sherard, 1953) A leak had developed through the base of the dam. At the time the article was written it was unknown as to why it was leaking and remedial measures (sand bagging) were being applied. (ENR, v. 100, No. 21, p. 826) Leak began because of transverse settlement cracks near top center of the dam. The crack left a gap 150 ft (46 m) long and extends from water edge to upstream face of rock fill. The hole ultimately reached 180 x 40 ft (55 x 12 m) and nearly to stream bed. Remedial measures included sandbagging and sealing upstream face by dumping sacked cinders and sandy loam into hole. (ENR, v. 100, No. 22, 1928)
Point of Rocks, Co.	1914 1911	1915 1915 1927	Lessons, Table IV (1975) Sherard, (1953)	85 26	E E rolled	A-1	Deformation, total conduct. (Lessons, 1975) Concrete placed on 1 1/2 upstream slope failed because of 5 foot waves 1915, 1927, near failure of crest due to high winds and associated wave action. (Sherard, 1953) Slope protection concrete slab badly cracked. (Lessons, 1975)
Pomona Lake, Kan.	1963	1968	Lessons, Table IV and p. 314 (1975)	110 33	E	MR	Flow discharge; damage to stilling basins transition slab, apron slab and baggles. (Lessons, 1975)
Portland, Me.	1889	1893	ER, v. 28 (8-19-1893) Middlebrooks, (1953) Hill, (1902) Jorgensen, (1920)	45 14	E rolled clay face core		Piping along drain pipe. (Middlebrooks, 1953) Break occurred over a drain pipe overlain by an overflow pipe. Failure due to action of frost, light embankment or piping along the pipes. (Hill, 1902) (Jorgensen, 1920)
Portland, Or.		1894	ER, (2-2-1895) Justin, (1932) Hill, (1902) Jorgense, (1920)				Concrete lining failed. (Justin, 1932) Concrete lining failed. (Joregensen, 1920). The concrete lining of the reservoir cracked badly before water was let in. Water was then turned in and serious leaks and further cracks appeared in the lining. (Hill ,1902)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Portneuf, Id.	1911	1950	Sherard, (1953) Middlebrooks, (1953)	55 17	E rolled		Concrete conduit disintegrated and was replaced in 1950. (Middlebrooks, 1953) Disintegration of concrete due to poor quality of concrete during construction. (Sherard, 1953)
Prairie River, Wis.		1912	EN, v. 68 Middlebrooks, (1953)		E rolled		Overtopped. (Middlebrooks, 1953)
Pratts Fork, Oh.	1934	1938	OCE Files Middlebrooks, (1953) ENR, v. 121, 1938)	21 6			Overtopped. (Middlebrooks, 1953) Small earth dam with concrete spillway; failure due to heavy rains, no details given. (ENR, v. 121, 1938)
Priest Rapid	1959	1964	Lessons, Table IV (1975)	184 56	E-G	A-2	Leakage, embankment. (Lessons, 1975)
Providence, RI	1816	1916	EN, v. 45 Middlebrooks, (1953) Brownell, (1901)	17 5	E-R		Two dams. Upper and lower. Breaking of upper dam caused failure of lower dam. Due to heavy rains on frozen drainage area of upper dam, flow into reservoir was heavy. Reservoir level rose fast. It is thought that ice may have blocked spillway of upper dam causing overtopping and breaking. Water from upper reservoir entered reservoir of lower dam quickly raising water level. The dam was then overtopped and during this time broke. (Brownell, 1901)
Puddingstone, Ca.	1928	1926	Lessons, Table IV (1975) Middlebrooks, (1953) ENR, v. 96, p. 665 p. 913	182 55	E concrete face and core H	DDC	Overtopping, structure under construction. (Lessons, 1975) Due to clogged outlet. (Middlebrooks, 1953) Dam did not have a spillway, as it was planned to raise dam to greater height, there was a 4 x 7 ft (1 x 2 m) tunnel for flood runoff. As water level rose, tunnel became plugged with debris. All efforts to remove debris failed. As a result, overtopping occurred, inflicting major damage to structure. (ENR, v. 96, No. 22)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Raven, Ga.	1954	1954	Safety of Small Dams, 1974	50 15	E	F-1	Failure due to several reasons; 1) owner changed height without changing base width, steepening slopes, 2) contractor did not remove debris from site and did not use compaction on the soil. (Safety, 1974)
Rector Creek, Ca.	1946	1947	Lessons, Table IV (1975) Sherard, (1953)	170 52 150 46	E E rolled	A-1	Deformation, differential transverse embankment cracks , compacting. Cracking due to settlement; caused by rolling dry earth. (Sherard, 1953)
Red Mountain Reservoir	1949	1956	Lessons, Table IV (1975) Sherard, 1963)	60 18	E	A-2	Leakage, foundation. (Lessons, 1975) The internal downstream sand drain flowed from under the dam due to liquefication of the drain. (Sherard, etal., 1963)
Red Rock	1910	1910	Lessons, Table IV (1975)	106 32	E	DDC	Overtopping, structure under construction. (Lessons, 1975)
Red Rock, Io.	1969	1969	Lessons, Table IV and p. 319 (1975)	92 28	E	A-2	During first filling, wave wash made several benches in riprap on right embankment. Seepage emerged along downstream toe near the left abutment. (Lessons, 1975)
Riverside	1902	1910	Lessons, Table IV (1975)	42 13	E	A-1	Slope protection concrete slab badly cracked. (Lessons, 1975)
Roanoke, Va.							Earth bottom settled and caved in. (Jorgensen, 1920) City reservoir caved in. (Hill, 1902)
Roberts Pond. Mas.		1922	Latimer, (1922)		E puddled core	F-1	Failure due to failure of Longwalds Pond dam. (Latimer, 1922)
Rocky Ford, Ut.	1914	1915-50	Sherard, (1953) Middlebrooks, (1953)	70 21	E rolled		High saturated line, reservoir level limit in 1950. (Middlebrooks, 1953), leakage through foundation; foundation founded on gravel. Saturation line function of reservoir level. (Sherard, 1953)
Ross Barnett Res., Miss.	1962	1969	Lessons, Table IV and p. 321 (1975)	68 20	E	A-2	During first filling, abutment seepage developed along the downstream toe of the embankment at the left abutment. Seepage probably travelled along broken and weathered limestone surface and through glacial pervious material. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Roxborough Res. Pa.	1894	1894	ER, v. 30 (1-12-1895) Justin, (1932) Middlebrooks, (1953) Hill, (1902) Jorgensen, (1920)		E rolled		Piping. (Middlebrooks, 1953); due to settlement. (Justin, 1932) The bottom was lined with 4 in concrete upon a clay puddle 2 ft (.6 m). (Jorgensen, 1920) The rock underlying is gneiss and mica, and upper portion of which is more or less disintegrated. (Hill, 1902) A leak was discovered after a year of use. A depression was found in the embankment about 3 ft (.9 m) above the bottom. Underneath the brick lining the clay had been washed away, and below there was a fissure in the rock 6-8 in in width. Through this fissure the water escaped, coming to the surface 1,200 f5 (366 m) away. On first filling (1893) when depth of 20 ft (6 m) increased flow was detected in a spring nearby. The leak was not located for nearly a year after. (Hill, 1902)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Sallisaw Creek Watershed 29	1964	1964	Lessons, Table IV (1975)	62 19	E	A-2	Leakage, embankment. (Lessons, 1975)
Saluda, S. Car.	1930	1930	Lessons, Table IV (1975 ENR, v. 104 Middlebrooks (1953)	208 63	E-H H	DDC	Sliding embankment downstream slope. (Lessons, 1975) Core pool lost during construction. (Middlebrooks, 1953)
Sampna, Tank, India	1956	1961 1964	Lessons, Table VI (1975)	23 7	E	A-1	Dam, slide. (Lessons, 1975) Dam, slide. (Lessons, 1975)
San Andreas	1870	1906	Lessons, Table IV (1975)	105 32	E	A-1	Deformation, differential. (Lessons, 1975)
San Antonio, Ca.	1965	1969	Lessons, Table IV and p. 832 (1975)	224 68	E	A-1	Butterfly valve malfunctioned and 40 ft (12 m) of the 84 in (2.1 m) discharge pipe collapsed. (Lessons, 1975)
Sandy Run, Pa.	1914	1977	ENR, v. 199, No. 4, 1977	28 9			Overtopped and washed out. (ENR, v. 199)
San Pablo, Ca.	1920	1936 1921	Lessons, Table IV (1975) Corps of Engineers (1949) Middlebrooks, (1953)	220 67	E-H H	A-1	Deformation of total conduit. (Lessons, 1975) Fill loss through riprap; no filter. (Corps of Engineers, 1949) Slope protection inadequate. (Lessons, 1975) Fill loss through riprap, no filter. (Middlebrooks, 1953)
Santee	1941	1941	Lessons, Table IV (1975)	60 18	E-B	A-1	Slope protection concrete slab eroded. (Lessons, 1975)
Santee Cooper, SC	1942	1942-46	CE, v. 18 Middlebrooks, (1953)	80 24	E-H	A-1	Disintegration of porous concrete slope protection. (Middlebrooks, 1953)
Santo Amaro, Brazil		1907	Merriman, (1930) Middlebrooks, (1953)	63 19	H	DDC	Failed during construction due to slide. (Middlebrooks, 1953) Slip of blanket over core-wall of section east of hydraulic fill occurred during construction. Excess of clay in blanket did not allow good drainage outward. (Merriman, 1930)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Sarda Sagar, India	1961	1963	Lessons, Table VI (1975)	16 5	E	A-2	Foundation, boiling. (Lessons, 1975)
Schaeffer	1911	1921	Lessons, Table IV (1975)	100 30	E	F-1	Sliding embankment downstream slope. (Lessons, 1975)
Schenectady, NY		1916	EN, v. 76, p. 817 Justin, (1932)		E rolled timber core		Overtopped. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) Failed for third time, overtopped due to insufficient spillway. (Justin, 1932) Overtopped due to heavy rains and lack of spillway, there was a waste gate to empty flood waters but it was closed at the time. The dam had failed on two previous occasions but no details were given except to say that all three failures were due to faulty design. (EN, v. 76, No. 17)
Schofield, Utah	1926	1927	ENR, v. 100 (7-5-1925) Middlebrooks, (1953) Justin, (1932)		E-R concrete reinforced core		Transverse cracking and piping into rock. (Middlebrooks, 1953) due to insufficient freeboard. (Justin, 1932) When water level was 1 ft (.3 m) from crest water began seeping from toe. Three days later a depression was discovered in the crest and upstream slope above the high water line. Very early the next morning a general leakage was discovered emerging over a length of about 90 ft (27 m) over the downstream rock toe. Five hours later a section 90 ft (27 m) long parallel to the axis of the dam, and 35 ft (11 m) wide had completely caved in at the water surface and water from the reservoir was pouring through the downstream rock section. Within a few hours, the embankment had caved in for a length of 180 ft (55 m). Soundings indicated that the crater-like hole extended nearly to the original streambed. Fortunately, there was no yield of the downstream rock section. For two days sand bags were dumped into the hole. The cavity was then refilled with earth. (Sherard, 1953)
Scottdale, Pa.		1904	EN, v. 52 (9-4-1904) Middlebrooks, (1953)	60 18			Piping. (Middlebrooks, 1953) Water working through dam at one end. Faulty design, no core wall. (Jorgensen, 1920) Built 1901 rolled E. The failure was not due to flood. Leakage through embankment was discovered same day as failure. Failure caused by water working through seams in the natural rock at the end of the dam close to the earthwork, thus starting a leak which rapidly enlarged. (Whited, 1904)
Seefield, Utah		1925	ENR, v. 79 Middlebrooks, (1953)	130 40	E rolled		Overtopped. (Middlebrooks, 1953)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Sepulveda Canyon (Lessons) Sepulrida Canyon, Ca. (Justin)	1914	1914	ER, v. 74 p. 357 Lessons, Table IV (1975) Bennett, (1916)	65 20	E concrete core	F-1	Overtopped; completed structure; insufficient spillway. (Lessons, 1975) Due to heavy rains, and insufficient spillway the dam failed. Middlebrooks summarizes R. Bennett, in Engineering Record, v. 74 as overtopping. I don't think the whole dam was overtopped but a notch cut 26 ft (8 m) deep and 12 ft (4 m) wide in the center was overtopped by some extent and because of this the fill behind the concrete core was washed away causing the core to break and wash away. (Bennett, 1916)
Shavers Lake	1925	1954	Lessons, Table IV (1975)	198 60	E	MR	Deterioration of concrete due to severe climate. (Lessons, 1975)
Sheep Creek, ND	1969	1970	Lessons, Table IV and p. 333 (1975)	60 18	E	F-2	Cause uncertain; probably cause leaks from joints in spillway pipes after heavy rain. (Lessons, 1975)
Sheffield, Ca.	1925	1925	ENR, v. 95 Middlebrooks, (1953) Sherard etal. (1963) Shaughnessy, (1925)	30 9	E rolled		Earthquake slide. (Middlebrooks, 1953) Failure of the dam after an earthquake shock was probably caused by liquefication of the lower part of the dam or the foundation. (Sherard etal., 1963) Failure caused by combination of earthquake and leakage. Due to earthquake and saturation a slide occurred on downstream slope, lowering crest. (Shaughnessy, 1925)
Shell Oil Co.	1946	1947	Lessons, Table IV (1975)	78 24	E	A-2	Sliding embankment upstream. (Lessons, 1975)
Shelton, Conn.	1901	1903	ER, v. 49, No. 9 Middlebrooks, (1953) Jorgensen, (1920)	20 6	E-R		Piping. (Middlebrooks, 1953) Two dams failed. Upper dam earth with sheet-piling and clay puddling. Lower dam was "stone" grouted. The failure of the upper dam is attributed to burrowing muskrats. The lower structure was swepth out by flood from the upper dam. (EN v. 49, No. 9) Earth-fill with masonary downstream wall. Failed near spillway. Muskrats burrowing in embankment and water finding its way under the masonry wall. (Jorgensen, 1920)
Sherburne, NY	1892	1905	EN, v. 54 Middlebrooks, (1953) Musson, (1905)	34	E rolled		Overtopped. (Middlebrooks, 1953) Due to heavy rains. Two dams involved. One above the other. The upper dam was overtopped and washed away while at the lower dam the partition between spillways was carried out. (Musson, 1905)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Sherburne Lake, Mont.	1916		Corps of Engineers Middlebrooks, (1953) Sherard, (1953)		E rolled		Floating logs displaced hand-placed riprap. (Middlebrooks, 1953) The excavation for the spillway initiated a slow slide of considerable extent on the left abutment above the spillway. Over the years after completion of the dam the hillside moved slowly but continually toward the dam, deforming and lifting the spillway structure. The slide has never endangered the dam and the reservoir has been regulated so the spillway has never been used. (Sherard, 1953)
Sherman, Neb.	1962	1964-68	Lessons, Table IV and p. 339	134 41	E	MR	From 1964-1968 beaching occurred in that the smaller sized riprap moved downstream to form a berm. Bedding was moved in some areas. (Lessons, 1975)
Short Creek, Ark.		1939	ENR, v. 122 Middlebrooks, (1953)	57 17	E-R	DDC	Overtopped during construction. (Middlebrooks, 1953) Due to heavy rains and rapid rise in water level. (ENR, v. 122, No. 17)
Sidie Hollow	1965	1965	Lessons, Table IV (1975)	52 16	E	A-2	Leakage, foundation. (Lessons, 1975)
Silver Jack	1971	1969	Lessons, Table IV and p. 341 (1975).	173 53	E	DDC	Massive slide occurred downstream of right abutment; slide triggered by excavation for spillway. (Lessons, 1975)
Sinker Creek, Id	1910	1943	Middlebrooks, (1953) Sherard, (1953) Lessons, Table IV (1975)	70 21	E-H	F-1	Leakage embankment. (Lessons, 1975) Seepage slide. (Middlebrooks, 1953) Sloughing occurred when embankment was saturated. Began soon after completion, because of this reservoir level was kept as low as possible. The reservoir was kept full for about six months and was under frequent inspection because of its history of sloughing. Early in the evening on the day of failure, considerable seepage and sloughing was noticed on downstream toe. The sloughing worked its way progressively upstream. By midnight the sloughing had worked back to the downstream edge. Just after midnight, the reservoir broke through the thin remaining section of embankment. (Sherard, 1953)
Six Mile Creek, NY	1905	1905	EN, v. 53 (6-25-1905) Justin (1932) Middlebrooks, (1953)	15 5	E rolled		Overtopped. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) Failed during great flood. (Jorgensen, 1920) Dam was carried away by a flood caused by heavy rainfall. (EN, v. 53, no. 26)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Snake Ravine	1893	1898	EN, v. 40 p. 242 Middlebrooks, (1953) Lippincott, 1898	64 19	E	F-1	Leakage embankment. (Lessons, 1975) Poor compaction. (Middlebrooks, 1953) Dam failed because of improper use of the hydraulic method. Dam was built 2 times first in 1893 and failed at the 30 ft (9 m) level. Second dam failed when tested June 1898. Second dam was built on wreck of the first. The dam began leaking on both sides and foundation and was moved en masse down the ravine, a distance over 1,000 ft (over 300 m). Failure was blamed on contractor who rejected the location and method of construction of the officers of the district. (Lippincott, 1898)
South Fork	1852	1889	Lessons, Table IV (1975)	72 22	E rock	F-1	Flow discharge, larger than spillway capacity. (Lessons, 1975)
Spartanburg, Pa.		1892	EN, v. 27 Middlebrooks, (1953) Justin, (1932) Hill, (1902) Jorgensen, (1920)	10 3	E-R		Overtopped. (Middlebrooks, (1953) Due to insufficient spillway. (Justin, 1932) Failure by overtopping due to inadequate spillway. (Hill, 1902) (Jorgensen, 1920) Dam overtopped due to excessive rains, causing part of dam to be washed out. (ENR, v. 27, June 16, 1892)
Spring Lake, RI	1887	1889	EN, v. 20 (9-31-1899) Middlebrooks, (1953) Hill, (1902) Jorgensen, (1920)	18 5	E-R		Piping along an outlet. (Middlebrooks, 1953) The portion washed away was just above the waste pipe. (Hill, 1902) (Jorgensen, 1920)
Staffordville, Conn.		1887	EN, v. 4 Middlebrooks, (1953) Hill (1902) Jorgensen, (1920)	20 6	E-R masonary face		Piping along an outlet. (Middlebrooks, 1953) Piping along outlet laid through embankment. (Hill, 1902) (Jorgensen, 1920)
Standley Lake, Colo.	1911 1912	1916 1912	ENR, v. 78 Lessons, Table IV (1975)	113 34	H E	A-1	1916 Core too large, slides during and after construction. (Lessons, 1975) 1912 Sliding embankment downstream slope. (Lessons, 1975)
Standley Lake, Colo.	1909	1967-71	Lessons, Table IV and p. 343 (1975) Sherard, 1953 Gelder, 1917 Fellows, 1917 Hayes, 1917	118 36	E	A-1	Small slide 1912; larger slip in 1916, (Hayes, 1917) Settlement in dam due to the existence of trestle. Sliding due to: 1) Two solid and compacted embankments, between which was a pool of water, into which was dumped dry, loose material, which never dried out; 2) great snowstorm in winter but ground never froze; saturation occurred upon melting; 3) reservoir level quickly lowered leaving saturated embankment standing; 4) slide occurred in saturated material. (Fellows, 1917)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Standley Lake, Colo. (continued)							A partial failure of the back of the dam near the center of the embankment occurred. About 300 ft (91 m) in length of the rear, one-half of the embankment bulged at the lower face to some extent, the line of breakage extending to the inner edge of the crest of the dam. Water level was immediately lowered and dam placed under constant watch. (EN, v. 72, No. 6)
							1912 A small slip occurred on the downstream slope affecting an area approximately 60 ft (18 m) by 60 ft (18 m). The depth of the material was only a few feet.
							1913 Cracks appeared on the crown, running parallel with the axis near the center of the dam. The cracks continued to open and after a year they were a foot in width and extended for lengths of 300 ft (91 m).
							1914 The crest had settled a maximum of 13 ft (4 m) since the end of construction. During the same year a slide, confined to the outside slope, occurred on the downstream slope. The vertical displacement at the top was 13 ft (4 m) and the horizontal movement at the toe was 60 ft (18 m).
							1917 As the reservoir was being drawn down for irrigation purposes, a small crack appeared on the upstream slope parallel to and about 30 ft (9 m) below the crest. This crack was the first indication of what was to be a general slide on the upstream slope.
							The movement was quite rapid the first week. Twenty four hours later where this crack had been, a nearly vertical shear, 10 ft (3 m) in height existed. The earth behind the slide was dry giving no indication of having been saturated.
							Later and after further movement, which increased the height of the vertical face, there appeared a pronounced seepage at elevation 5450.
							After the first rapid movement, the material continued to creep slowly down the slope. No major repairs were made and creep contined each time the reservoir was drawn down.
							1922 As the reservoir was being drawn down, a slide very similar to the 1917 slide, occurred on the upstream slope. The maximum vertical displacement was about 20 ft (6 m). (Sherard, 1953)
							1967 Cracking occurred on crest with settlement towards the upstream.
							1968-1970 Further cracking and settlement towards upstream.
							1971 Severe cracking and settlement by spreading. Sliding in foundation towards downstream; movement stopped when water level kept below gates. (Lessons, 1975)
Stanislaus Forebay, Ca.	1908	1970	Lessons, Table IV and p. 345 (1975)	60 18	E	A-1	After heavy rains a number of relatively shallow slides occurred in downstream embankment slopes. (Lessons, 1975)
State Park, Geo.	1955	1975	ENR, v. 195 No. 12	80 24	E		Drawdown before failure occurred. Leaks found in concrete spillway. The largest seepage was through weepholes, with some leakage through minor cracks. (ENR, v. 195)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Stockton Creek, Ca.	1949	1950	Sherard, (1953) Middlebrooks, (1953) Lessons, Table IV (1975)	95 29	E rolled	F-2	Failed at abutment, probably along contact or crack. (Middlebrooks, 1953) Leakage, embankment piping. (Lessons, 1975) Shortly after construction and after a heavy rainstorm, the level of the reservoir rose rapidly to the spillway for the first time, a section of the dam washed out near the right end. The dam had been under observation up until the night before and nothing unusual had been seen. It is believed that the most probable cause of failure is piping in an embankment crack due to idfferential settlement. (Sherard, 1953)
Sublette, Id.	1915	1916 1937	Sherard, (1953) Lessons, Table IV (1975)	40 12 51 16	E rolled R	A-1	Conduit cracked as result of settlement; no loss of embankment. (Sherard, 1953) Can find no reference to the 1937 deformation of conduit described in Table IV. Crest raised 1940.
Summer Lake, Or.	1925	1925	Lessons, Table IV (1975) ASCE Trans. v. 94 Middlebrooks, (1953)	60 18	E rolled	A-2	Foundation slide (Middlebrooks, 1953), sliding embankment downstream slope leakage foundation. (Lessons, 1975)
Suputrida Canyon, Ca.		1914	Middlebrooks, (1953)	65 20	E concrete core		Overtopped. (Middlebrooks, 1953)
Surry Mountain	1942	1943	Lessons, Table IV (1975)	91 28	E	A-1	Sliding abutment slopes. (Lessons, 1975)
Swansen, Wales Swansea, SW Wales (Justin, 1932)	1867	1879	SE, v. 3, p. 437 Middlebrooks, (1953) Justin, (1932)	80 24	E-R puddle core		Piping. (Middlebrooks, 1953)
Swift	1914	1964	Lessons, Table IV (1975)	189 58	R-E	F-1	Overtopping completed structure. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Stockton Creek, Ca.	**1949**	1950	Sherard, (1953) Middlebrooks, (1953) Lessons, Table IV (1975)	95 29	E rolled	F-2	Failed at abutment, probably along contact or crack. (Middlebrooks, 1953) Leakage, embankment piping. (Lessons, 1975) Shortly after construction and after a heavy rainstorm, the level of the reservoir rose rapidly to the spillway for the first time, a section of the dam washed out near the right end. The dam had been under observation up until the night before and nothing unusual had been seen. It is believed that the most probable cause of failure is piping in an embankment crack due to idfferential settlement. (Sherard, 1953)
Sublette, Id.	**1915**	1916 1937	Sherard, (1953) Lessons, Table IV (1975)	40 12 51 16	E rolled R	A-1	Conduit cracked as result of settlement; no loss of embankment. (Sherard, 1953) Can find no reference to the 1937 deformation of conduit described in Table IV. Crest raised 1940.
Summer Lake, Or.	**1925**	1925	Lessons, Table IV (1975) ASCE Trans. v. 94 Middlebrooks, (1953)	60 18	E rolled	A-2	Foundation slide (Middlebrooks, 1953), sliding embankment downstream slope leakage foundation. (Lessons, 1975)
Suputrida Canyon, Ca.		1914	Middlebrooks, (1953)	65 20	E concrete core		Overtopped. (Middlebrooks, 1953)
Surry Mountain	**1942**	1943	Lessons, Table IV (1975)	91 28	E	A-1	Sliding abutment slopes. (Lessons, 1975)
Swansen, Wales Swansea, SW Wales (Justin, 1932)	**1867**	1879	SE, v. 3, p. 437 Middlebrooks, (1953) Justin, (1932)	80 24	E-R puddle core		Piping. (Middlebrooks, 1953)
Swift	**1914**	1964	Lessons, Table IV (1975)	189 58	R-E	F-1	Overtopping completed structure. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURES
Table Rock Cove, SC Table Rock Cave, SC	1927	1928	Lessons, Table IV (1975) Justin, (1932) ENR, v. 100 (12-12-1929) p. 935	140 43	E rolled E No core	A-1	Total downstream valve changer. (Lessons, 1975). Failure of valve changer due to weight of overburden. Blew out as a geyser. (Justin, 1932) Failure of blow-off pipe. Blow-off pipe valve was on downstream end. Settling caused break of pipe inside embankment. Since valve was on downstream end water could not be stopped. Fortunately, the break did not reach the water line so failure was only partial. (Justin, 1932) Slump due to failure of a 42 in class D cast-iron drainage pipe passing through it in a trench along the bank of the stream. The pipe had a valve at downstream end which was closed. The pipe had been damaged due to differential settlement and joints in the pipe were opened at the top by as much as two inches. Water was running through the pipe and out through the opened joint creating piping along the pipe. As the piping progressed inside the dam, material above the blowout began falling down, causing the slump. (Henry, 1929) and (ENR, v. 100 No. 19, 1928)
Tacoma, Wa.		1892	Hill (1902)	17 5	E timber		Undermined due to construction on porous or yielding foundation. (Hill, 1902)
Tappan, Oh.	1935	1934	Middlebrooks, (1953)	52 16	E rolled		Slide in foundation. (Middlebrooks, 1953)
Tater Hill Lake, NC		1977	ENR, v. 199, No. 19	22 7	E		Dam swept away by flood. (ENR, v. 199)
Tecumseh, Ala.		1894	EN, v. 32 Middlebrooks, (1953)				
Telluride, Colo.		1909	Colorado State Engr. Middlebrooks, (1953)	30 9	E-R		Overtopping. (Middlebrooks, 1953)
Terrace Reservoir	1912	1957	Lessons, Table IV (1975)	157 48	E	A-1	Leakage, embankment. (Lessons, 1975)
Teton, Id.	1976	1976	ENR, v. 197, No. 3	305 93	E	DDC	Leakage abutment and foundation. Massive leakage through grout curtin. Massive leakage around the end of grout curtin Leakage at the dam's contact with the north abutment wall Leakage into the dam at some other point, and emergence of water at the north end of dam's downstream side. Filled too fast Leakage through a crack near the dam's north abutment. (ENR, v. 197, No. 3)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Throttle, N. Mex.	1912	1942	Sherard, (1953) Middlebrooks, (1953)	65 20	E rolled		Overtopped, but did not fail. (Middlebrooks, 1953). Overtopping due to reduction of freeboard by settling, an erosion of crest, combined with high precipitation. (Sherard, 1953)
Tiber, Mont.	1956	1956-67	Lessons, p. 348 and Table IV (1975)	205 62	E	A-1	Shortly after beginning operation, cracking of gate structure was noticed. Excessive gate settlement occurred through the years and in 1967 declared inoperable. Settlement caused by collapse of joint cracks due to solution and removal of gypsum. (Lessons, 1975)
Tiffin, Oh.	1914?	1915	EN, v. 75 p. 1121 Justin, (1932)	18 5	E upstream and downstream slopes concrete covered.		Prior to 1915, settlement of earth caused cracking on face, leaks appeared on cutoff (January). February brought high water overtopping structure and then broke through the dam. (Justin, 1932) Because of a previous washout the whole dam was concrete covered, making the whole dam a spillway. The earth settled causing the concrete face to settle and crack. The dam was then overtopped and a large section was washed away. (Justin, 1932)
Tittesworth, G. Britain	1962	1962	Lessons, Table VI (1975)	31 9	E	A-3	Dam, slide. (Lessons, 1975)
Toa Vaca, Puerto Rico	1971	1970	Lessons, Table IV and p, 350 (1975)	215 65	E-R	UDC	Overtopping, structure under construction, due to flood waters. (Lessons, 1975)
Tongue River, Mont.	1939	1964	Lessons, Table IV and p. 351 (1975)	141 43	E-R	MR	A hole was found in the apron of the spillway, about 200 ft (60 m) upstream of the spillway crest. (Lessons, 1975)
Toreson	1896	1953	Lessons, Table IV (1975)	49 15	E	F-2	Chemical drainage corrosion outlet pipe. (Lessons, 1975)
Toronto, Ontario Canada		1912	ER, v. 65 (4-27-1912) Justin, (1932) Middlebrooks, (1953) Jorgensen, (1920)	35 11	E rolled concrete core		Overtopped. (Middlebrooks, 1953); due to insufficient spillway. (Justin, 1932) Dam was overtopped and washed out for 130 ft (40 m) (Jorgensen, 1920)
Townshend Lake, Vt.	1961	1969	Lessons, Table IV and p. 352 (1975)	138 42	E	A-1	Leakage, foundation, leakage at downstream toe of the junction of the embankment and downstream riprap slope. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Trout Lake, Colo.	1894	1909	ER, v. 60 Middlebrooks, (1953) Justin, (1932) Fedler, (1909)	25 8	E rolled stone upstream slope		Overtopped. (Middlebrooks, 1953); insufficient spillway, overtopped due to dam failure upstream, quickly filling reservoir. (Justin, 1932) Due to the washout of an upstream dam, the sudden rise of water proved too much for the two plank flumes at the west end of the dam, which served as the spillway. (Justin, 1932) Enormous floods due to "cloud bursts" in the drainage area entered the reservoir, filling it and causing the dam to overflow. The overtopping caused a "notch" to remain in the crest of the dam, which continued to discharge after the flood had subsided. The action of this stream undermined the foundation at the downstream toe with the result that a day after the flood a small section of the dam began to settle and was pushed out of place. Due to this, what was left of the reservoir was emptied. (Fedler, 1909)
Tupper Lake, NY	1906	1906	EN, v. 57 Middlebrooks, (1953)	18 5	E rolled		Piping along an outlet. (Middlebrooks, 1953)
Turkey Creek, Colo.		1910	Colorado State Engr. Middlebrooks, (1953)	22 7	E rolled	DOC	Slide during construction. (Middlebrooks, 1953)
Turlock Irrigation, Ca.		1914	Middlebrooks, (1953) Jorgensen, (1920)	56 17			Leakage around an outlet. (Middlebrooks, 1953) After first filling; piping along an outlet. (Jorgensen, 1920)
Turtle Creek, Tx.		1851	EN, v. 25 Middlebrooks, (1953)	29 9	E rolled		Foundation settlement. (Middlebrooks, 1953)
Tuttle Creek, Kan.	1962	1967	Lessons, Table IV and p. 358 (1975)	157 48	E	MR	Outlet gate works. Corrosion of structural steel gate components and cavitation of concrete. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Union Bay, B.C. Canada		1912	EN, v. 67 Middlebrooks, (1953) Jorgensen, (1920) Mitchell, (1912)	20 6	E timber core		Overtopped. (Middlebrooks, 1953) Very poor design and construction. There were no trenches in foundation. Dam was overtopped and disintegrated. (Jorgensen, 1920) Due to heavy rains and obstructed spillway, dam was overtopped washing away fill on backside of dam, exposing timber core. Timber core was unstable, poorly jointed and anchored. Failed by combination of overturning and sliding. (Mitchell, 1912)
Unknown, Venezuela	1965	1965	Lessons, Table VI (1975)	33 10	E	A-2	Cracking. (Lessons, 1975)
Upper Barnes Creek, Tex.	1956	1966	Lessons, Table IV and p. 361 (1975)	64 19	E clay core	A-1	After heavy rains; sliding embankment downstream slope due to abnormally high pore pressures. (Lessons, 1975)
Upper Elk Creek, No. 22, Okla.	1970	1971	Lessons, Table IV and p. 362 (1975)	61 19	E	A-1	Leakage abutment, underseepage by-passed the cutoff and the shallow foundation drain. (Lessons, 1975)
Upper Highline Res. Colo.	1966	1967	Lessons, Table IV and p. 363 (1975)	88 26	E	A-1	Leakage, foundation. (Lessons, 1975)
Upper San Fernando, Ca.	1921	1971	Lessons, Table IV and p. 364 (1975)	82 25	E-H	A-1	Stability earthquake. February 9, 1971 earthquake (6.6 Richter) jolted both dams. The earthquake caused the dam to move downstream a maximum of 5 ft (2 m) at the crest and settled by as much as 3 ft (1 m). (Lessons, 1975)
Utica Reservoir, NY	1874	1902	ER, v. 46, v. 48, p. 226, 290 Lessons, Table IV (1975) Middlebrooks, (1953) Justin, (1932) EN, v. 49	70 21	E rolled	F-1	Sliding embankment downstream slope. (Lessons, 1975) Insufficient compaction. (Middlebrooks, 1953) Steep slopes; poor construction. (Justin, 1932) Dam built of two types of material, impervious overlain by a pervious light light material described as "slightly better than sand." There was no compaction to speak of on some sections, as the earth was dumped from wheelbarrows. Failure occurred in the light upper material (composing the upper 20-25 ft (6-8 m) of the dam.). After the break occurred and the water had left the reservoir the remaining portion of the dam slumped due to settling of the inner or wet portion of the dam, so as to materially flatten the upper slope and leave a pronounced crack, or series of cracks, along the whole, or nearly the whole, crest. (EN, v. 49)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Union Bay, B.C. Canada		1912	EN, v. 67 Middlebrooks, (1953) Jorgensen, (1920) Mitchell, (1912)	20 6	E timber core		Overtopped. (Middlebrooks, 1953) Very poor design and construction. There were no trenches in foundation. Dam was overtopped and disintegrated. (Jorgensen, 1920) Due to heavy rains and obstructed spillway, dam was overtopped washing away fill on backside of dam, exposing timber core. Timber core was unstable, poorly jointed and anchored. Failed by combination of overturning and sliding. (Mitchell, 1912)
Unknown, Venezuela	1965	1965	Lessons, Table VI (1975)	33 10	E	A-2	Cracking. (Lessons, 1975)
Upper Barnes Creek, Tex.	1956	1966	Lessons, Table IV and p. 361 (1975)	64 19	E clay core	A-1	After heavy rains; sliding embankment downstream slope due to abnormally high pore pressures. (Lessons, 1975)
Upper Elk Creek, No. 22, Okla.	1970	1971	Lessons, Table IV and p. 362 (1975)	61 19	E	A-1	Leakage abutment, underseepage by-passed the cutoff and the shallow foundation drain. (Lessons, 1975)
Upper Highline Res. Colo.	1966	1967	Lessons, Table IV and p. 363 (1975)	88 26	E	A-1	Leakage, foundation. (Lessons, 1975)
Upper San Fernando, Ca.	1921	1971	Lessons, Table IV and p. 364 (1975)	82 25	E-H	A-1	Stability earthquake. February 9, 1971 earthquake (6.6 Richter) jolted both dams. The earthquake caused the dam to move downstream a maximum of 5 ft (2 m) at the crest and settled by as much as 3 ft (1 m). (Lessons, 1975)
Utica Reservoir, NY	1874	1902	ER, v. 46, v. 48, p. 226, 290 Lessons, Table IV (1975) Middlebrooks, (1953) Justin, (1932) EN, v. 49	70 21	E rolled	F-1	Sliding embankment downstream slope. (Lessons, 1975) Insufficient compaction. (Middlebrooks, 1953) Steep slopes; poor construction. (Justin, 1932) Dam built of two types of material, impervious overlain by a pervious light light material described as "slightly better than sand." There was no compaction to speak of on some sections, as the earth was dumped from wheelbarrows. Failure occurred in the light upper material (composing the upper 20-25 ft (6-8 m) of the dam.). After the break occurred and the water had left the reservoir the remaining portion of the dam slumped due to settling of the inner or wet portion of the dam, so as to materially flatten the upper slope and leave a pronounced crack, or series of cracks, along the whole, or nearly the whole, crest. (EN, v. 49)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Valentine, Nebr.	1911	1911	EN, v. 63	30 9	E rolled clay core		Settlement of spillway. (Middlebrooks, 1953). Reservoir filled to within inches of spillway crest; earth below spillway settled and dam went out, spillway was concrete covered earth (Justin, 1932) Failure due to settling of the concrete covered earth spillway, first filling, when reservoir level rose to within inches of the spillway. (Jusin, 1932)
Valentine, La.		1937	EN, 1937		E		After heavy rains spillway brought into use; large amount of water going over spillway resulted in erosion of lining. Lining consisted of creosoted timber planking. (ENR, Nov. 4, 1937)
Val Marie, Sask, Canada	1939	1952	ENR, v. 148 Middlebrooks, (1953) Lessons, Table VI (1975)		E rolled E	A-1	Overtopped. (Middlebrooks, 1953) Main dam; overtopping. (Lessons, 1975)
Valparaiso, Chile		1888	E and BR, v. 18, p. 270	56 17	E rolled		Slopes too steep. (Middlebrooks, 1953)
Van Norman, Ca. Upper Lower		1972 1972	Safety of Small Dams (1974)		E H		Due to earthquake both dams partially failed. Lower dam: Upstream face slid inward with portions of the toe moving upstream under water for almost three-fourth of a mile. Upper dam: A significant portion of the embankment moved bodily downstream. No water lost but outlet works severely damaged. (Safety, 1974)
Victor, Colo.		1901	ER, v. 43 p. 550 Middlebrooks, (1953) Justin, (1932)	25 8	E rolled		Overtopped. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) Failure due to inadequate spillway. (Hill, 1902) (Jorgensen, 1920)
Victor Braunig, Tex.	1962	1969	Lessons, Table IV and p. 371 (1975) ASCE VI part 1, 1972	90 27	E	A-1	Deformation, differential transverse embankment cracks. Leakage foundation. (Lessons, 1975)
Vir, India	1961	1962 1963	Lessons, Table VI (1975)	24 7	E	A-1	Cracking. (Lessons, 1975) Cracking. (Lessons, 1975)
Virgin River, Nev.	1925	1929	ENR, v. 103 Middlebrooks, (1953)	120 37	E-R rolled	DDC	Poor design and construction. (Middlebrooks, 1953) According to observations seepage through the dam became so rapid that a large slide on lower slope occurred and thereafter disintegration of the fill continued rapidly. (ENR, v. 103, No. 14, p. 526)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Wachusett North Pike Wacnusett, Mass.	1904	1907	Lessons, Table IV (1975) Schuyler (1908) Saville, (1907) Merriman, (1930)	82 25	E rolled	A-1	Upstream slope slide, water surface 40 feet below top. (Schuyler, 1908) Sliding embankment, upstream slope. (Lessons, 1975) The slip happened at the point of maximum section, where the embankment is about 80 ft (24 m) and where there was 42 ft (13 m) of water. The portion that slipped into the water was about 675 ft (206 m) with thickness normal to the slope of about 35 ft (11 m). The slide did not affect safety of the dam. (Saville, 1907) On April 11, 1907, a portion of the upstream face 675 ft (205 m) long, of a thickness of 35 ft (11 m) normal to the slope, slid down the bank, causing little damage but indicating that 1:2 slope was too steep for such fine material under water. (Merriman, 1930)
Waco	1965	1961	Lessons, Table IV (1975)	140 43	E	LDC	Sliding embankment downstream slope. (Lessons, 1975)
Wagner Creek, Wa.	1918	1913	ENR, v. 120 Lessons, Table IV (1975)	50 15	E and hydraulic	F-1	Spillway failure. (Lessons, 1975) Failure due to spillway failure; heavy snow melt filled reservoir for first time; did not overtop; effort to strengthen dam several days ahead of time failed. (ENR, v. 120) Unusually heavy snow with melt filled reservoir top of dam; did not overtop, failure due to fault in concrete spillway. (ENR, April 1938)
Wahlawa	1905	1921	Lessons, Table IV (1975)	136 41	E-R	A-1	Flow discharge, damaged spillway. (Lessons, 1975)
Walnut Grove, Az.	1886	1890	Wegmann, (1927) Lessons, Table IV (1975) Hill, (1902) Jorgensen, (1920) ENR, v. 100	110 33	Rock	F-1	Overtopped; flow discharge greater than spillway; washout of dam. (Lessons, 1975) Failure by overtopping. (Hill, 1902) (Jorgensen, 1920) Failure ascribed to insufficient spillway, which could not discharge the flood waters, and to carelessness in the execution of the work. (Wegmann, 1927) Failure due to too small a waste weir, causing a backing up of water and overtopping. (ENR, v. 100, No. 12, p. 472)
Wanapum	1964	1964	Lessons, Table IV (1975)	186 57	G-E	A-1	Gate control failed. (Lessons, 1975)
Wanship, Ut.	1957	1969	Lessons, Table IV and p. 376 (1975)	174 53	E	NR	Flow discharge through left gate, creating an unsymmetrical load caused damage to the outlet to the stilling basin. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCES	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Washita, Ok.		1914	Jorgensen, (1920)	12 4	E concrete core		Concrete core undermined and washed out. Break 35 ft (11 m) Third time dam washed out. (Jorgensen, 1920)
Massy, France	1861-63	1883	Sherard, (1953)	54 16	E brick face		Due to rapid drawdown, a slide occurred on the upstream slope. The dam was repaired by "carefully compacting" the earth. (Sherard, 1953)
Weisse, Czech. Weisse Passe, Bohemia		1916	EN, v. 77, p. 139 Middlebrooks, (1953)	42 13	E rolled		Piping along an outlet. (Middlebrooks, 1953) Failure due to percolation along an outlet. Began as a small stream of clear water just above the top of the conduit and in less than a quarter of an hour it had become a stream of dirty water several inches through. Collapse of the dam over the conduit followed soon. Author stated dam was of bad design and failed due to bad design and incompetence. (EN, v. 77, no. 4)
Wesley E. Seale	1958	1965	Lessons, Table IV (1975)	114 35	E	F-2	Flow discharge, spillway gate failure. (Lessons, 1975)
Westbranch	1966	1964	Lessons, Table IV (1975)	93 28	E	DDC	Sliding embankment upstream slope. (Lessons, 1975)
West Julesburg, Co.	1905	1910	ER, v. 63 Justin, (1932) Middlebrooks, (1953)	55 17	E rolled		Piping. (Middlebrooks, 1953) Seepage along ledge rock. (Justin, 1932)
West River Providence (2)		1901	Jorgensen, (1920)		E		Both composed of fine gravel and sand. Water over crest close to wooden sluice way. (Jorgensen, 1920)
Wheatland No. 1, Wy.	1960	1969	Lessons, Table IV and p. 377 (1975)	45 14	E	F-2	Sliding embankment, downstream slope exact cause unknown possibly 1) piping through embankment along conduit, 2) wave action by unusual winds. (Lessons, 1975)
Whitewater Brook, upper, NH	1943	1972	Lessons, Table IV and p. 374 (1975)	62 19	E	F-1	During heavy rains earth embankment was breached adjacent to spillway. (Lessons, 1975)
Wichita Falls, Tx. Holliday Creek	1901	1901	EN, v. 45		E	DDC	One leak occurred, was stopped by 5:00 PM by 7:00 PM another leak occurred, could not be stopped and dam broke at 1:30 AM. The break is said to have been some 200 ft (61 m). (EN, v. 45, No. 21)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Williams	1912	1947	Lessons, Table IV (1975)	40 12	G-E	F-2	Leakage built up high pressures. (Lessons, 1975)
Wilmington, Del.	1864 1867	1900	ER, v. 42 Middlebrooks, (1953) Justin, (1932) Hill, (1902) Jorgensen, (1920)	12 4	E rolled brick face core		Piping along an outlet. (Middlebrooks, 1953) Piping along outlet. (Hill, 1902) (Jorgensen, 1920)
Winston, NC	1902	1912	EN, v. 67, p. 667 Middlebrooks, (1953) Justin, (1932) Ambler, (1912) Jorgensen, (1920)	24 7	E rolled mason core		Overtopped. (Middlebrooks, 1953) And breached due to insufficient spillway. (Justin, 1932) Insufficient spillway and poorly constructed. (Jorgensen, 1920) Due to heavy rains, and inadequate spillway, dam was overtopped and breached. (Justin, 1932) Due to excessive rainfall dam was breached 70 ft (21 m) long and 24 ft (7 m) deep. Rapid filling of reservoir by rains caused the dam to overflow. Overflow of dam caused backside of dam to wash away exposing core. Core failed bit by bit. (Ambler, 1912)
Wisconsin Dells Dells and Hatfield, Wis.	1909 1910	1911 1911	Lessons, Table IV (1975) ER, (10-10-1911) EN, v. 66, p. 483	59 18 34 10	E E concrete core	F-1	Overtopping, completed structure. (Lessons, 1975)
Wise River, Mont.		1927	ENR, v. 99 Middlebrooks, (1953)		E rolled		Overtopping. (Middlebrooks, 1953) Failure caused by high water cutting through the bank at one end of the dam. (ENR, v. 99)
Wister, Okla.	1949 1951	1949	Lessons, Table IV (1975) Middlebrooks, (1953)	90 27	E rolled	A-2	Leakage embankment piping. (Lessons, 1975) Piping. (Middlebrooks, 1953)
Woodhead embankment, England		1850	Walters, (1962) p. 50		E		Built on alternating grits and clays; failure due to percolation through the grits. (Walters, 1962)
Woodmoor Res. Colo.	1968	1972	Lessons, Table IV and p. 384 (1975)	60 18	E	A-1	Leakage, foundation. (Lessons, 1975)

NAME LOCATION	BUILT	FAILED	REFERENCE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Woodrat	1956	1961	Lessons, Table IV (1975)	84 26	E	A-1	Leakage embankment; sliding embankment downstream slope. (Lessons, 1975)
Worcester, Colo.	1912	1951	Sherard, (1953) Middlebrooks, (1953)	68 21	E rolled		Concentrated seepage. (Middlebrooks, 1953) In 1951 concentrated seepage appeared at toe near middle of the dam. Appears to be a long-standing problem. No threat to safety. (Sherard, 1953)
Worcester, Mass.	1871	1876	ASCE Trans., v. 5 and 6 Middlebrooks, (1953)	41 12	E rolled		Leakage in culvert. (Middlebrooks, 1953) Water leaked around culvert passing through embankment. (Jorgensen, 1920) Culvert passed through base of embankment, making a breach 200 ft (61 m) long. (Hill, 1902) There had been leakage into a culvert since dam was completed. The culvert, in the bed of the brook, through the dam, contained supply pipes. The beginning of the failure was two years prior, when water was first observed in considerable quantity into the pipe culvert, about 20 ft (6 m) above the central wall, and passing out through the arch and lower gate-house. It was proposed to clean out the culvert, find the leakage, and stop it. The culvert was cleaned out but leak was never found. Two days before final failure, large quantities of muddy water were observed to be flowing from the culvert below the lower gate-house. An examination was made and found to be entering at or near the old leak site. The dam then failed. Failure was due to a stratum of porous material underlying the gate-house and upper end of pipe vault, partaking the nature of quicksand. The water found its way through the porous material and leaked through the interstices of the masonary (culvert) until it wore a passage large enough to carry earth with it. (Ellis, etal., 1876)
Wyndotte County	1941	1937	Lessons, Table IV (1975)	91 28	E	DDC	Embankment downstream slope failure. (Lessons, 1975)
Wyandote County Lake, KN (probably same as Wyndote County above)		1938	ENR, v. 120	84 26	E puddle core	DDC	The dam failed by slumping of the downstream bank for half the length of the structure. Dam failure due to pockets of a plastic jelly-like material 30-35 ft (9-11 m) below natural surface of the ground squeezed out under weight of dam, allowing dam to collapse and bulge in downstream direction. These bulges caused a rise in elevation of as much as 14 ft (4 m) at points below downstream face. (ENR, v. 120, No. 8)

NAME LOCATION	BUILT	FAILED	REFERECNE	HGT FT M	TYPE	USCOLD INC.	CAUSE OF FAILURE
Yorba	1907	1930	Lessons, Table IV (1975)	50 15	E	A-1	Leakage, foundation. (Lessons, 1975)
Yuba, (Old dam). Ca.		1907	EN, v. 58, (8-8-1907) Middlebrooks, (1953) Justin, (1932) Jorgensen, (1920) Murphy, (1907) Sherard, (1953)	630 192	E rolled		Overtopped. (Middlebrooks, 1953) Due to insufficient spillway. (Justin, 1932) 630 ft (192 m) was carried away in a great flood. Water was 7 ft (2 m) above crest. (Jorgensen, 1920)
Yuba, (New dam). Ca.	1949	1951	Sherard, (1953) Middlebrooks, (1953)	25 8	E		Seepage slide, downstream slope enlargement. (Middlebrooks, 1953) Slide triggered by earthquake combined with increasing pore pressure in the embankment due to a year of full reservoir. (Sherard, 1953) Due to heavy rains. Some 630 ft (192 m) of the barrier extending 80 ft (24 m) from the south abutment to 540 ft (164 m) from the north abutment was completely swept away. 70 ft (21 m) or more of the north part remaining was undermined and badly damaged. The failure was probably due to undermining, caused by the backlash, the latter having been aided by weakening of the concrete surface by the scour of the debris (from flood waters). (Murphy, 1907)
Zuni, N. Mex.	1907	1909	EN, v. 62 Middlebrooks, (1953) Justin, (1932) Saville, (1916) Jorgensen, (1920)		E-R hydraulic		Piping through abutment. (Middlebrooks, 1953) Piping between hydraulic fill and rock. (Justin, 1932) Dam was undermined. (Jorgensen, 1920) Undermining due to gravel layer under lava flow. (Saville, 1916)