Training
Aids for
Dam
Safety

MODULE:

EVALUATION OF EMBANKMENT DAM STABILITY AND DEFORMATION

TADS

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EVALUATION OF EMBANKMENT DAM STABILITY AND DEFORMATION

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PREFACE

There are presently more than 80,000 dams in use across the United States. Like any engineering works, these dams require continual care and maintenance, first to ensure that they remain operational and capable of performing all intended purposes, and then to preclude endangering people and property downstream.

The safety of all dams in the United States is of considerable national, state, and local concern. Given that, the principal purpose of the TADS (Training Aids for Dam Safety) program is to enhance dam safety on a national scale. Federal agencies have responsibility for the safe operation, maintenance, and regulation of dams under their ownership or jurisdiction. The states, other public jurisdictions, and private owners have responsibility for the safety of non-Federal dams. The safety and proper custodial care of dams can be achieved only through an awareness and acceptance of owner and operator responsibility, and through the availability of competent, well-trained engineers, geologists, technicians, and operators. Such awareness and expertise are best attained and maintained through effective training in dam safety technology.

Accordingly, an ad hoc Interagency Steering Committee was established to address ways to overcome the paucity of good dam safety training materials. The committee proposed a program of self-instructional study embodying video and printed materials and having the advantages of wide availability/marketability, low per-study cost, limited or no professional trainer involvement, and a common approach to dam safety practices.

The 14 Federal agencies represented on the National Interagency Committee on Dam Safety fully endorsed the proposed TADS program and have underwritten the cost of development. They have also made available technical specialists in a variety of disciplines to help in preparing the instructional materials. The states, through the Association of State Dam Safety Officials, also resolved to support TADS development by providing technical expertise.

The dam safety instruction provided by TADS is applicable to dams of all sizes and types, and is useful to all agencies and dam owners. The guidance in dam safety practice provided by TADS is generally applicable to all situations. However, it is recognized that the degree to which the methods and principles are adopted will rest with the individual agency, day owner, or user. The sponsoring agencies of TADS assume no responsibility for the manner in which these instructional materials are used or interpreted, or the results derived therefrom.

ACKNOWLEDGMENTS

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MODULE INTRODUCTION

EVALUATION OF EMBANKMENT DAM STABILITY AND DEFORMATION

MODULE INTRODUCTION

OVERVIEW OF THIS MODULE

This module provides information about embankment dam stability, including settlement and deformation. It is suggested that this module be read in its entirety to gain a meaningful perspective of the subject matter. The following topics will be covered:

- The purpose of evaluating the structural stability of existing embankment dams,

- Types of data to be reviewed and evaluated before an actual investigation can be performed,

- Methods that may be used to conduct stability analyses, and

- Potential remedial actions.

Due to the wide variety of geologic materials used in embankment dam construction and the diverse geologic environments in which embankment dams are built, it is not possible to present all conditions individually in this module. However, the basic principles and general considerations for safe and successful performance of existing embankment dams will be described. A realistic understanding and proper application of these ideas should enable site-specific use of the information presented.

Slope instability is one form of stability problems for existing embankment dams. Other conditions endangering the stability of an embankment dam are excessive deformations, excessive stresses, overtopping, and internal erosion. These forms of embankment instability can occur during routine and unusual conditions.

This module is intended to provide general information on quantitative assessment of embankment dam stability under static and dynamic loading conditions. However, it is not the objective of this module to provide specific information on any particular methodology or procedure used in assessing embankment dam stability. For such details, the reader should refer to other technical literature on the subject. Some of this literature is included in Appendix C. The emphasis of this module is to discuss what needs to be done for quantitative assessment of stability of existing embankment dams, rather than on how to do it. It is also not the intent of this module to provide information on analyzing embankment dam failures.

MODULE INTRODUCTION

HOW TO USE THIS MODULE

This module is designed to be used in conjunction with other Training Aids for Dam Safety (TADS) modules. The TADS Learner's Guide lists all of the TADS modules and presents a recommended sequence for completing the modules. You may want to review the Learner's Guide before beginning this module.

CONTENTS OF THIS MODULE

For clarity of presentation, the evaluation of **settlement and deformation**, **static stability**, and **dynamic stability** of existing embankment dams are presented in separate units. The commonalities in an embankment dam stability and deformation evaluation, such as general data review (including instrumentation), modes of failure, and analysis considerations are discussed together in a separate unit.

This module is divided into five units followed by three appendixes:

- **Unit I. Overview:** Presents background information on evaluating embankment dam stability, safety evaluation requirements, the role of seepage in embankment dam stability, embankment dam behavior, and historical perspective on embankment dam incidents.

- **Unit II. Common Considerations:** Presents common considerations for evaluating the stability of an embankment dam, including the types of data to be reviewed and evaluated, modes of failure, and embankment dam analysis considerations.

- **Unit III. Settlement And Deformation:** Discusses how settlement and deformation can affect the stability of an embankment dam, what type of project data should be reviewed and analyzed, methods of analyses, and what remedial actions can be taken to alleviate settlement and deformation problems.

- **Unit IV. Static Stability:** Presents information on how static instability affects an embankment dam, what type of project data should be reviewed and analyzed, methods of analyses, and what remedial actions can be taken to improve static stability.

- **Unit V. Dynamic Stability:** Describes how dynamic instability affects an embankment dam, what type of project data should be reviewed and analyzed, methods of analyses, and what remedial actions can be taken to improve dynamic stability.

CONTENTS OF THIS MODULE (Continued)

- **Appendix A. Glossary:** Defines technical terms used in the module.
- **Appendix B. References:** Lists references that can be used to supplement this module.
- **Appendix C. Additional Reading:** Provides articles that supplement this module.

DESIGN OF THIS MODULE

This module is comprised of text instruction only. There is no accompanying video presentation.

UNIT I

OVERVIEW

I. OVERVIEW: INTRODUCTION

INTRODUCTION

An embankment dam must be able to safely withstand static and dynamic loads that may be imposed upon it during its life. If you are an owner or otherwise have responsibility for an embankment dam, inherent in that responsibility is your obligation to ensure the static and dynamic stability of the dam.

This unit provides an overview for evaluating embankment dam stability, including:

- Safety evaluation requirements
- Effect of seepage on embankment dam stability
- Embankment dam behavior
- Historical perspective on embankment dam incidents

I. OVERVIEW: BACKGROUND INFORMATION

INTRODUCTION

Embankment dams have been built since early times. The general philosophy in design of these dams has been to utilize locally available geologic materials. Design practices have evolved with improved understanding of soil behavior. Construction techniques have evolved with advances in earthmoving and compaction equipment.

Embankment dams are a preferred choice for sites with wide valleys and difficult foundation conditions because of their flexibility. However, soil is a difficult engineering material because of its three-phase nature, diverse composition, and our incomplete understanding of its behavior under all of the stress and boundary conditions usually encountered in the field. Soil behavior under load is, in general, highly nonlinear, time dependent, and strain softening. The geologic past of a damsite significantly affects the in-service performance of the dam, but this information is generally not completely known.

The design and construction documentation of older embankment dams can be very sparse. Similarly, information on dam instrumentation and performance over the years may be incomplete or nonexistent. Even for relatively newer dams, where records are extensive, there may be different levels of completeness of records and different degrees of quality in design and construction.

EVALUATING EMBANKMENT DAM STABILITY

Soil mechanics, as an engineering science, is a relatively young discipline in engineering education and practice. Earthquake engineering of embankment dams is even younger and somewhat in its formative stages. Although a great deal has been learned and put to use in design and construction of newer dams, there exist in the field a large number of dams designed and built without the benefits of modern understanding of soil behavior and improved construction techniques.

When combined, these factors make the stability evaluation of existing embankment dams a difficult and challenging engineering undertaking. Because of uncertainties in problem definition, and an incomplete understanding of soil behavior under all loading conditions encountered, the stability evaluation of existing embankment dams must proceed on a conservative basis.

I. OVERVIEW: BACKGROUND INFORMATION

EVALUATING EMBANKMENT DAM STABILITY (Continued)

Engineers responsible for remedial action usually do not have the full range of options to deal with potential problems that were available at the time the original project was conceived. They must cope with existing conditions, including the presence of the dam itself. Being denied direct access to the foundation under the dam and its appurtenances for inspection and remedial treatment, the engineers must sometimes devise imaginative ways to circumvent the handicap. Often, economics rule against or limit the time available for lowering the reservoir water level to facilitate work on the upstream parts of the dam, the reservoir floor, or on the abutments below the normal water surface. For these reasons, remedial work may be more difficult and more expensive than corresponding categories of work would have been at the outset of the project.

The structural safety of an embankment dam is dependent primarily on the absence of excessive deformations and pore fluid pressure build-up under all conditions of environment and operation, the ability to safely pass floodflows, and the control of seepage to prevent migration of materials and thus preclude adverse effects on stability.

SAFETY EVALUATION REQUIREMENTS

All embankment dams in service, regardless of their age, should be systematically evaluated for their safe performance under all operational conditions. The principal requirement for dam safety evaluation is to protect public safety, life, and property. Hence, all dams must function safely under routine everyday operations as well as under unusual conditions such as floods and earthquakes. The potential for adverse incidents, such as excessive seepage, instability, and major damage during floods and earthquakes, needs to be assessed to ensure that the safety of people and property will not be endangered by the dam. If a risk does exist, corrective actions need to be taken.

EFFECT OF SEEPAGE ON EMBANKMENT DAM STABILITY

All embankment dams are subject to some seepage passing through, under, and around them. If uncontrolled, seepage may be detrimental to the stability of the structure as a result of excessive pore water pressures, or by internal erosion. For existing embankment dams, all seepage records compiled during the existence of the structure should be reviewed for significant trends or abnormal changes. The cause of any abnormality should be determined as accurately as possible. Any record or evidence that seepage flows have removed any significant amount of fine-grained soil must be evaluated through field investigations. Turbid flow issuing from a dam or its foundation may be an indication of internal erosion. Seepage should be effectively controlled to preclude structural damage or interference with normal operations. Refer to the TADS module entitled <u>Evaluation of Seepage Conditions</u> for more information on seepage related issues.

I. OVERVIEW: BACKGROUND INFORMATION

EMBANKMENT DAM BEHAVIOR

All embankment dams in service deform and settle under self-weight and imposed loads. In general, deformations of embankment dams may result in aesthetically unacceptable surficial appearance. However, excessive deformations indicate internal distress of the dam. The static response of a dam is due to the internal stresses caused by the weight of a dam and routine operations of the reservoir, including reservoir drawdown. The flood condition is transitory, but generally treated as a static condition for slope stability analysis purposes. The dynamic response of a dam is due to the internal stresses caused by an earthquake and is in addition to its static response. At the end of an earthquake activity, a new state of the embankment dam prevails and an altered static response continues.

The difference in the state of an embankment dam before and after an earthquake can be small or large, depending on the earthquake characteristics, site geology, and the embankment characteristics. These items will be discussed in later units. It is significant to note that the above-mentioned physical sequence in field response under various loading conditions should be reflected in stability assessment to the extent possible.

HISTORICAL PERSPECTIVE ON EMBANKMENT DAM INCIDENTS

The records of dams indicate that on the average about 10 significant dam failures have occurred somewhere in the world in each decade, and many more damaging near-failures have occurred (Safety of Dams: Flood and Earthquake Criteria, National Academy Press, 1985). Earthfill dams have been involved in the largest number of dam failures, followed in order by gravity dams, rockfill dams, and multiple and single arch dams. That more troubles would occur among the more prevalent dam types should not be surprising. Considering the number of failures compared to total number of dams built for each type, embankment dams show a comparatively good record.

Major embankment dam failures in the United States include: Teton Dam (1976); Buffalo Creek Dam (1972); Baldwin Hills Dam (1963); Lower Otay Dam (1916); Walnut Grove Dam (1890); South Fork Dam (1889); and Mill River Dam (1874). The major embankment dam incidents in the United States include: San Luis Dam (1981); Lower and Upper San Fernando Dams (1971); Fontenelle Dam (1965); Hebgen Dam (1959); and Sheffield Dam (1925). Dams and Public Safety by R. B. Jansen; Advanced Dam Engineering for Design, Construction, and Rehabilitation edited by R. B. Jansen; and Development of Dam Engineering in the United States edited by E. B. Kollgaard and W. L. Chadwick give detailed descriptions on these and other dam incidents.

I. OVERVIEW: BACKGROUND INFORMATION

HISTORICAL PERSPECTIVE ON EMBANKMENT DAM INCIDENTS (Continued)

There are several instances in which potential instability problems were identified during routine safety evaluations and corrective actions taken to avert possible dam incidents from happening. Jackson Lake Dam, Navajo Dam, Wolf Creek Dam, Walter F. George Dam, and Clemson Lower Division Dam are some of the embankment dam examples where major corrective actions were taken to improve their stability in anticipation of their problems becoming serious and potentially leading to significant dam failures in the future.

TABLE I-1. HISTORICAL RECORD OF EMBANKMENT DAM FAILURES AND ACCIDENTS TO 1979 FOR DAMS OF HEIGHTS 50 FEET OR GREATER

CAUSE	FAILURE	INCIDENT
Overtopping	18	7
Flow Erosion	14	17
Slope Protection Damage	--	13
Embankment Leakage, Piping	23	14
Foundation Leakage, Piping	11	43
Sliding	5	28
Deformation	3	29
Deterioration	2	3
Earthquake Instability	--	3
Faulty Construction	--	3
Gate Failure	1	3
TOTAL	77	163

From: Development of Dam Engineering in the United States by E. B. Kollgaard and W. L. Chadwick, (Eds.). Pergamon Press, New York, 1988.

I. OVERVIEW: SUMMARY

SUMMARY: BACKGROUND INFORMATION

Unit I provided some background information that will be helpful when preparing to evaluate the stability of an embankment dam, particularly: safety evaluation requirements, role of seepage on embankment dam stability, and embankment dam behavior.

This unit also provided a historical perspective on embankment dam incidents, and instances where potential instability problems had been identified during routine safety evaluations.

UNIT II

COMMON CONSIDERATIONS

II. COMMON CONSIDERATIONS: OVERVIEW

INTRODUCTION

The analysis of existing dams is usually not as detailed as the procedures involved in the design of new dams. Some critical areas in an embankment and/or foundation may require a detailed review. Primarily, the review is intended to ensure the presence of safe and adequate embankment dams under routine, everyday operations as well as under unusual conditions such as floods and earthquakes. In performing this review, you should examine all available data to determine if problem areas have been recognized, and to identify information that can be used to evaluate the structural adequacy of the existing embankment dam. The need for supplemental information should also be identified during this review.

Unit II provides common considerations for evaluating the stability of an embankment dam, including making a general review and evaluation of project data as a part of a stability or deformation analysis. Your review of project data should include the following types of information:

- Design and as-built construction data, including geologic site conditions, hydrology, structural analyses, factors of safety, geotechnical analyses, design documents, as-built drawings and records, and as-built survey data.

- Inspection reports.

- Field explorations and laboratory testing.

- Instrumentation data.

The as-built design and construction of a dam should be compared with current practice and regulatory requirements.

Unit II will also describe the various modes of failure under nonearthquake and earthquake conditions, as well as common stability problems associated with embankment dams.

Finally, a brief discussion of the developments in embankment dam analysis including analysis limitations, analysis of existing dams versus new dams, and analysis organization is provided.

II. COMMON CONSIDERATIONS: DOCUMENTATION REVIEW

INTRODUCTION

It is essential to know and understand the design considerations for loading conditions, instrumentation, construction details, and reservoir operations for a particular embankment dam under study. This information will help you in several ways while evaluating the stability of the dam. For example, the loading conditions considered in the design stage can be compared with the loading conditions considered relevant for the safety evaluation. In general, the stability of an embankment dam cannot be considered adequate for loading conditions which exceed those considered in the initial dam design without proper verifications.

GENERAL GUIDE TO DATA REVIEW

You may use the following list of items as a general guide in reviewing data for an existing embankment dam:

- Design Data Review
    - Foundation Geologic Data
        - Geologic description
        - Fractures, joints
        - Cementing
        - Previous loadings
        - Material characteristics
        - Permeability tests
        - In situ tests (Standard Penetration Test (SPT), Cone Penetration Test (CPT), Vane Shear)
        - Potential weak zones or materials
        - Faulting, seismicity
        - Aerial photographs
    - Laboratory Test Data
        - Types of tests performed
        - Results of tests
        - Source of material (compare to actual material used in construction)

GENERAL GUIDE TO DATA REVIEW (Continued)

. Field Exploration

- Test pits
- Drill holes
- Trenches

. Design Layout

- Foundation treatment
- Foundation excavation
- Internal zoning of dam
- Methods of controlling seepage

. Stability Analyses Performed

- Sections analyzed (compare with actual as-built geometry)
- Material properties used (compare with laboratory tests for both pre- and post-construction, typical values of similar materials)
- Pore pressure assumptions (how were they derived, are they reasonable, do they compare with observed conditions)
- Loading conditions analyzed (do they agree with actual and future loadings)
- Slope stability failure surface configuration (were all potential problem surfaces analyzed, such as potential wedge failure surfaces through weak layers)
- Analysis method used and factors of safety obtained (are they considered appropriate by modern practice)

• Construction Data Review

. Geologic Data

- Geologic mapping
- Additional explorations
- Note differences between conditions assumed during design and those exposed during construction

II. COMMON CONSIDERATIONS: DOCUMENTATION REVIEW

GENERAL GUIDE TO DATA REVIEW (Continued)

- Foundation Excavation and Treatment
  - Design
  - Construction reports and photographs
- Source of Embankment Materials
  - Were the actual materials used different than assumed in design (if so, what properties, such as strength, durability, permeability, etc. are different and could they adversely affect performance)
- Embankment Construction
  - How was material placed
  - Type of equipment
  - Compaction performed
  - Moisture content and how added
  - Granular material--look at the percentage of fines. Note any zoning changes made during construction.
- Laboratory Tests
  - What tests were performed
  - What material was tested and how representative was it
  - Frequency of tests
  - Results

- Performance Data Review
  - Seepage
  - Deformations
  - Instrumentation

II. COMMON CONSIDERATIONS: DOCUMENTATION REVIEW

DAM AND FOUNDATION MATERIALS

Stresses and deformations in an existing embankment dam are due to self weight, applied static and dynamic loads, and are also influenced by the deformation and strength response of its foundation to the same loads. Therefore, it is essential to know the spatial distribution of materials in the body of the dam and its foundation. This information is generally obtained from the design and construction documents. However, if these documents are incomplete or unavailable, the following information may be helpful in estimating materials within the body of a dam under study.

Existing dams can be viewed in light of knowledge of studies and reports on similar dams of the same vintage to gain an understanding of probable design and construction methods. Some field explorations may be required to acquire information about the foundation materials and verify the estimated embankment dam materials.

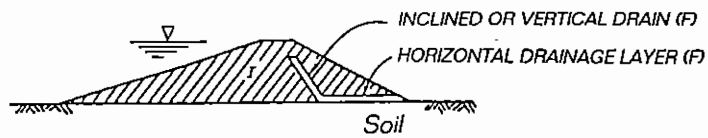
Figure II-1 is a schematic description of typical cross sections of embankment dams (Engineering and Design--Earth and Rock Fill Dams, General Design and Construction Considerations, Army Corps of Engineers, 1982; T. M. Leps, 1978). Depending upon the relative proportion and kinds of materials used, embankment dams are divided into three broad categories:

- Homogeneous Dams
- Zoned Earth and Rockfill Dams
- Rockfill Dams With Impervious Membranes

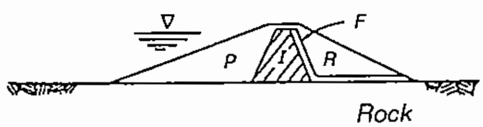
II. COMMON CONSIDERATIONS: DOCUMENTATION REVIEW

DAM AND FOUNDATION MATERIALS (Continued)

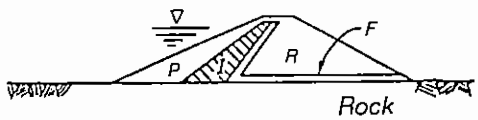
FIGURE II-1(a). TYPICAL CROSS SECTIONS FOR HOMOGENEOUS AND ZONED EMBANKMENT DAMS



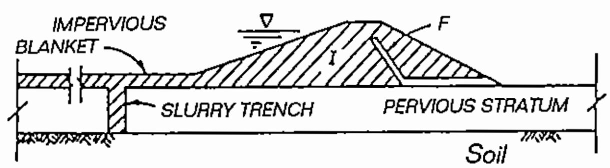
a. Homogeneous dam with internal drainage on impervious foundation



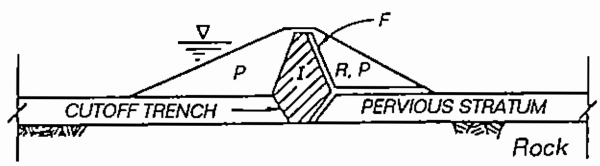
b. Central core dam on impervious foundation



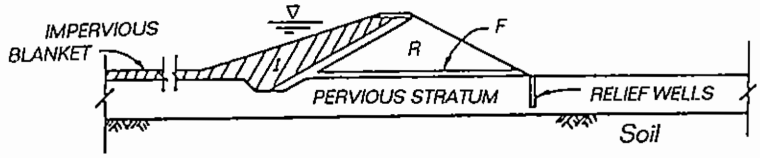
c. Inclined core dam on impervious foundation



d. Homogeneous dam with internal drainage on pervious foundation



e. Central core dam on pervious foundation



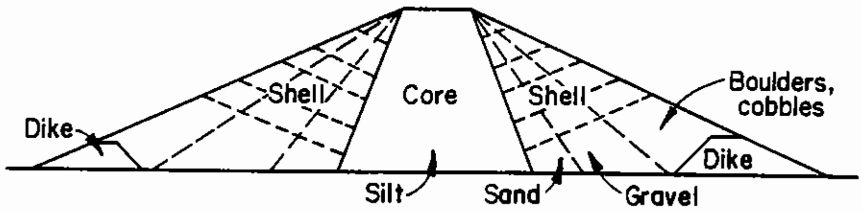
f. Dam with upstream impervious zone on pervious foundation

LEGEND

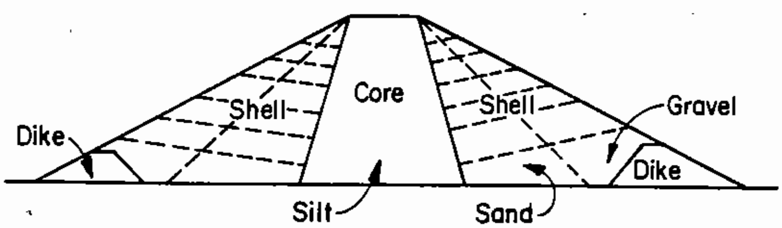
I = Impervious
P = Pervious
R = Random or Rockfill
F = Select Material

DAM AND FOUNDATION MATERIALS (Continued)

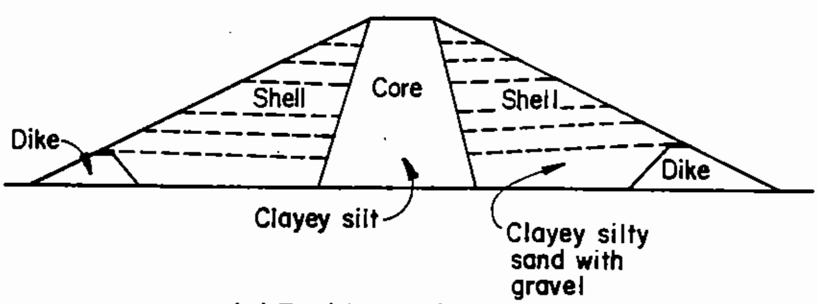
FIGURE II-1(b). TYPICAL CROSS SECTIONS FOR FULL HYDRAULIC FILL DAMS



(a) Glacial borrow



(b) Alluvial borrow



(c) Residual soil borrow

II. COMMON CONSIDERATIONS: DOCUMENTATION REVIEW

Homogeneous Dams

Homogeneous dams are usually constructed entirely or almost entirely of a single impervious soil. Sometimes this type of dam also has an internal drainage system. They are usually used for low to moderate height applications of up to 150 feet. A homogeneous dam is generally more massive and usually has flatter slopes than a zoned embankment of the same height.

Zoned Earth And Rockfill Dams

Zoned earth and rockfill dams are constructed of distinct and separate fill zones. Typically, this type of dam has a central impervious soil core. Adjacent to the core are transition filters between the fine-grained core and coarse rockfill or randomfill shells. The impervious core may be centrally located, placed along the sloping upstream face of the dam, or located in an intermediate position. For applications exceeding the height range for homogeneous dams, and compressible foundation materials, a zoned dam is normally the choice.

Hydraulic Fill Dams

The cores of some older dams have been placed by hydraulic means. These hydraulic fill dams frequently contain large masses of loose to very loose soils in them because of the dumping and sluicing of the soils during construction. Adequate soil data (for example, SPT blow counts, gradation analysis, and phreatic surface) must be available to evaluate the liquefaction potential and stability of these dams.

Rockfill Dams With Impervious Membranes

Rockfill dams with impervious membranes are used where there is a shortage of locally available fine-grained soils needed to form an impervious core. In this situation, the impervious membrane may be an upstream asphaltic concrete or reinforced concrete face slab; or on smaller dams, a steel or plastic liner, or a central asphalt or concrete core wall.

II. COMMON CONSIDERATIONS: INSTRUMENTATION

INTRODUCTION

The instrumentation in a dam may be grouped into two categories:

- The original instrumentation that was planned and installed for monitoring dam performance and safety.

- Any additional instrumentation that may have been installed during the construction or post-construction periods to address specific problems or concerns.

All data, whether from original instrumentation or instruments added later, should be reviewed.

INSTRUMENTATION DATA

Reviewing the instrumentation data from an embankment dam and its foundation helps identify information that could either be used in a stability analysis, or in verifying analysis results by comparing them with the measured responses. For example, measured piezometric data could be used in static slope stability analysis; measured earthquake motion data could be used to verify the correctness of dynamic stability analysis procedure by comparing the calculated results with the corresponding measured responses.

The existing conditions of interest in dam behavior include:

- Pore water pressure and seepage
- Stresses and deformations
- Earthquake motions

The type, number, and location of instruments in the body of an embankment dam and its foundation vary considerably between projects. There has been a need for increased instrumentation of newer projects due to the utilization of damsites having weak foundation strata and the construction of ever higher earth and rockfill dams. Embankment dams designed and built since the 1930s usually have an array of instruments installed in them to measure performance.

II. COMMON CONSIDERATIONS: INSTRUMENTATION

INSTRUMENTATION DATA (Continued)

Commonly used instruments include:

- Piezometers (open or closed systems located in the foundation and embankment)
- Surface monuments (embankment and structural measurement points)
- Baseplates at the dam-foundation contact
- Inclinometers
- Movement indicators (at joints and cracks)
- Internal vertical and horizontal movement devices and strain indicators
- Earth pressure cells
- Accelerometers (in areas of seismic activity)

For more information about these devices, refer to the TADS module, Instrumentation for Embankment and Concrete Dams.

During the inspection of many older dams in the 1970s and 1980s, it was often found necessary to install or replace primitive instruments with modern instruments to measure deformations, pore water pressures, and seepage conditions accurately.

II. COMMON CONSIDERATIONS: MODES OF FAILURE

INTRODUCTION

Every mass of soil located beneath a sloping ground surface has a tendency to move downward and outward under the influence of gravity. Such a mass can fail in several different ways depending upon:

- The stratigraphy of the component materials of the deposit
- The orientation of the foundation on which it rests
- Pore fluid pressure
- Strengths of component materials
- External loading conditions

Figure II-2 on the following page is a schematic description of various modes of failure for embankment dams and the associated operational or loading conditions. Stability failures are sometimes confined only to the embankment but often involve the foundation. Failures have occurred slowly, as continuous or intermittent creep; they have also occurred suddenly without apparent warning. Often rapid failures are preceded by a period of slow movements. Therefore, it pays to review the instrument data periodically to recognize a stability problem in the early stages of its development, perform the required analyses, and implement necessary corrective actions to stop the problem from getting worse.

Failure modes due to excessive deformations, slope instability, internal erosion, and overtopping are the same for static and dynamic loading conditions. Construction related failures are not normally a concern in the safety evaluation of existing dams, and erosion related failures are discussed in the TADS module on <u>Evaluation of Seepage Conditions</u>.

CAUSES OF FAILURE

It is essential to investigate and understand possible causes of various forms of embankment instability so that effective and efficient corrective actions can be designed and implemented to mitigate a problem.

Nonearthquake Conditions

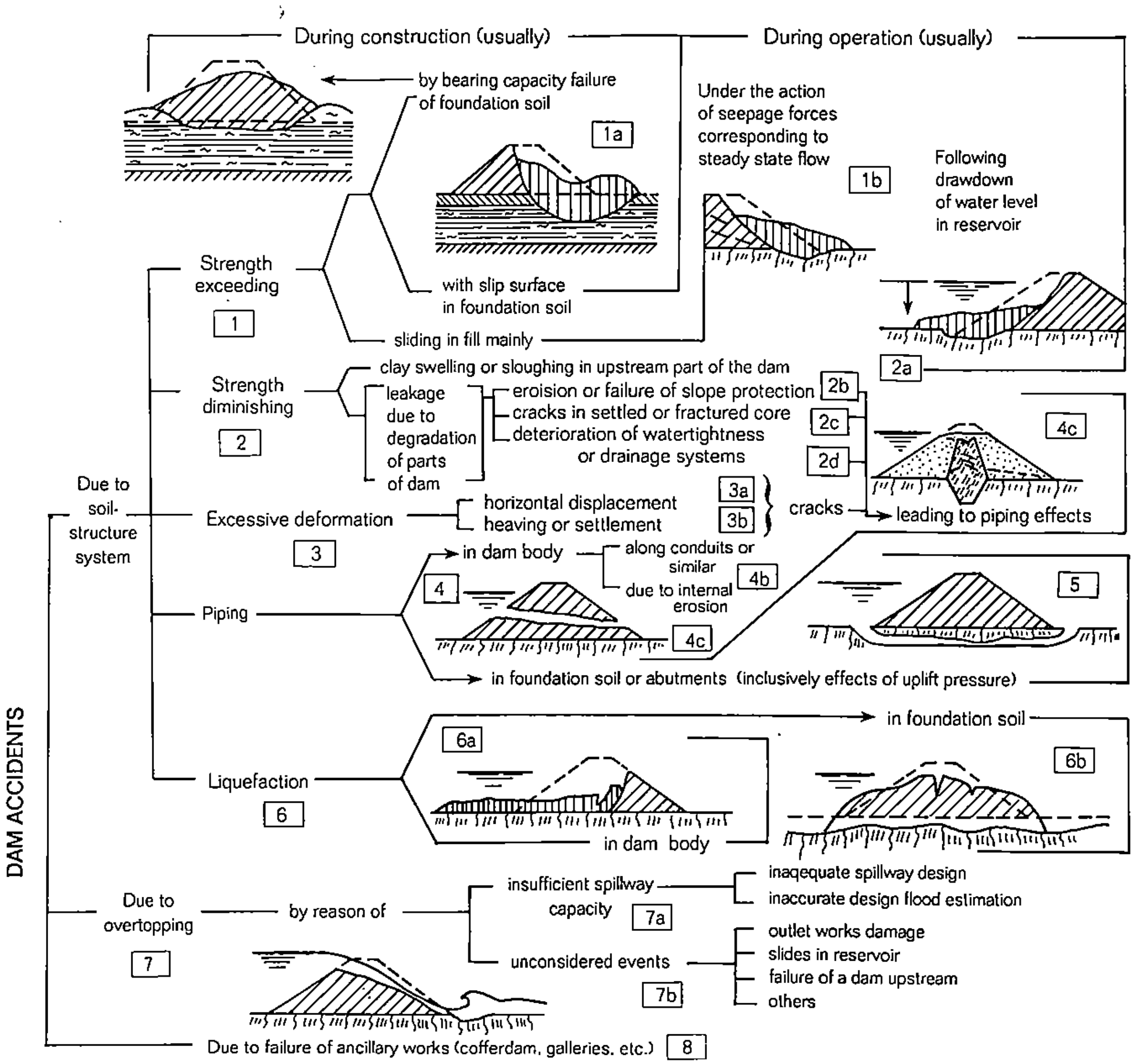
In general, under nonearthquake conditions, an embankment dam may fail due to:

- **Overtopping.** Overtopping of a dam can be due to:
  - Inadequate spillway capacity
  - Large, rapid landslides in the reservoir
  - Too little freeboard

II. COMMON CONSIDERATIONS: MODES OF FAILURE

Nonearthquake Conditions (Continued)

FIGURE II-2. SCHEMATIC DESCRIPTION OF FAILURE MODES IN EMBANKMENT DAMS



Nonearthquake Conditions (Continued)

- **Slope Failure.** Slope failures and failures by spreading are due to:
  - Design deficiencies
  - Neglected remedial actions

- **Piping.** Piping is due to initiation and continuation of internal erosion along paths of least resistance, such as:
  - Along outlet conduits
  - Through cracks across the impervious core of the dam
  - In inadequately compacted core material at its contact with uneven surfaces
  - In zones susceptible to erosion within the subsoil

Failures due to subsurface erosion are catastrophic failures. They occur with little warning, usually at full reservoir, and occasionally many years after the reservoir is first put into operation.

Earthquake Conditions

In general, under earthquake conditions, an embankment dam may fail due to excessive deformations and/or excessive pore fluid pressure buildup. Excessive deformations of a dam may lead to its being overtopped and result in dam failure by erosion. Nonuniform deformations may cause transverse cracks in the dam which allow rapid flow of water through the dam, resulting in dam failure due to internal erosion.

Excessive pore water pressure due to earthquake loading can cause slope instability which may grow in size and lead to dam failure because of reduced sections. Landslides along the reservoir rim can cause waves in the reservoir which may overtop the dam.

The nonuniform distribution of effective stresses in a soil deposit, brought about by the buildup of pore water pressure due to an earthquake, can cause differential settlements that result in transverse and longitudinal cracks in the embankment dam. Flow of water through transverse cracks can lead to dam failure by erosion.

The most serious instability in embankment dams and their foundations caused by earthquake loading is due to sudden densification of loose, saturated, noncohesive soils because of ground vibrations. This densification of the soil skeleton causes rapid buildup of pore fluid pressures. The soil, water, and air matrix forms a liquefied material. The increased share of overburden pressure taken by pore fluids reduces the effective stress between soil grains. Air and water, of course, have no or very little shear strength. This combination of high overburden pressure and low shear strength of liquefied materials, combined with ground vibrations can lead to large, flow-type movements. These flow movements are not calculable; only the potential for their occurrence can be estimated.

II. COMMON CONSIDERATIONS: MODES OF FAILURE

STABILITY PROBLEMS

The stability problems of concern in the safety evaluation of existing embankment dams are those which may cause the dam to fail. An embankment dam can fail under static conditions and/or dynamic conditions due to:

- Excessive deformations
- Excessive stresses
- Excessive loss of materials due to erosion

This module is primarily concerned with the analysis of embankment dam stability problems associated with excessive deformations and excessive stresses. Since deformations and stresses in an embankment dam are significantly affected by the pore pressure response, relevant discussions about water related effects on the stability of an embankment dam are included in this report. However, for detailed discussions on seepage related problems, refer to the TADS module on Evaluation of Seepage Conditions as well as other technical literature on the subject.

II. COMMON CONSIDERATIONS: DEVELOPMENTS IN EMBANKMENT DAM ANALYSIS

INTRODUCTION

In an embankment dam, deformations, stresses, and seepage under static and dynamic conditions are interdependent in the sense that one affects the other; and these responses are time dependent. In performing analyses, however, it is a usual practice to model these responses as separate entities. The interdependent aspects of the physical behavior are accomplished by sequential analysis of separate entities and incorporation of results from the analysis of one entity to the analysis of the next entity.

Significant progress has been made in developing unified analytical and numerical procedures for studying the stability of embankment dams. However, additional work in development, implementation, and verification of theories for true three-phase, time dependent, soil behavior under all stress and deformation conditions prevalent in the field still needs to be done. Thus, unified solution procedures are not commonly used in embankment dam engineering.

Procedures based on the finite element method have been used to analyze embankment dam engineering problems by incorporating soil behavioral aspects in them. Application of particulate mechanics to embankment dam engineering has not yet been possible. Therefore, the stability analysis results should be interpreted and understood with full awareness of the theories used in getting the analytical results.

Recognizing the limitations of theory, embankment dam engineering practice has evolved using simpler analytical approaches combined with observational methods. In this document, both complete methods and simpler approaches commonly used to evaluate embankment dam stability are discussed.

ANALYSIS LIMITATIONS

It should be recognized that, due to the complex nature of soil behavior, all observed behaviors of an embankment dam may not necessarily lend themselves to a precise analysis. In such instances, reliance on engineering judgment, based on professional experience of responsible engineers, is generally considered prudent and acceptable.

II. COMMON CONSIDERATIONS: DEVELOPMENTS IN EMBANKMENT DAM ANALYSIS

ANALYSIS OF EXISTING DAMS VERSUS NEW DAMS

There is no basic difference in methodology for the analysis of existing embankment dams and the analysis of new embankment dams. The attitudes in analysis of existing dams differ from those in new design in a more or less philosophical way. The engineering evaluation of an existing dam can be quite intensive because:

- The dam and the associated structures have already been built.
- The reservoir loading and operations are present.
- The geologic environment has been subjected to the dam and the reservoir effects.

For new designs, however, there are uncertainties in geologic data, material data, material response under inservice conditions, and construction. In spite of the best intentions, interests, and efforts of everybody involved in the design and construction of embankment dams, there is always an element of doubt or wishful thinking present about the successful performance of the projects. However, an existing dam presents the true response of the manmade structures and the geologic environment as a whole. Thus, in the study of existing dams, it is generally not a matter of making predictions; it is a matter of understanding the observed performance and making provisions for safe operations of the dam and reservoir.

The safety assessment of an existing dam may lead to the conclusion that remedial measures, such as adding a stability berm, relief wells, or restricting the reservoir level are needed. Remedial measures for dam safety deficiencies constitute a design. Thus, in this way, the safety evaluation of existing dams and design activities can be interrelated.

In design of new dams, the adopted factors of safety against various modes of failure reflect the extent of uncertainties involved and the associated risks of dam failure. In the evaluation of existing dams, some of these uncertainties might have been resolved or come to pass, and therefore, somewhat lower factors of safety, to reflect the residual uncertainties, may be appropriate. The extent of any reduction in the acceptable factor of safety against a particular mode of failure in existing dams must be made on an individual basis, and should be commensurate with the residual uncertainty and hazards associated with dam failure.

II. COMMON CONSIDERATIONS: DEVELOPMENTS IN EMBANKMENT DAM ANALYSIS

ANALYSIS ORGANIZATION

If evaluation of the stability of an existing embankment dam is for static conditions only, then there is no need to be concerned with the dynamic analysis details. However, if the stability evaluation of a dam is for dynamic conditions, there is a definite need to be concerned with the static analysis also. Since the dynamic response is in addition to the static response, it is essential to know the static response of the dam for input as a set of initial conditions for dynamic response. Therefore, it is suggested that the needs of dynamic analysis be considered while preparing for the static analysis to optimize the analysis efforts. This optimization can be in terms of a common numerical model for static and dynamic analysis, the handling of static analysis results which can be directly input for the dynamic analysis, the evaluation of results and data at every stage of analysis, etc.

It should be understood that performing a dynamic analysis makes sense only if the static stability of the dam is adequate. For dams with marginal static stability, a dynamic loading of any significance is bound to cause stability problems. Therefore, it is essential to ensure a sound static response of an existing embankment dam before continuing with the dynamic analysis.

Whereas a static analysis of an embankment dam can be performed with relative ease, a proper dynamic analysis of an embankment dam can be considerably more difficult, time consuming, and expensive. Therefore, proper planning and judicious use of available resources in all aspects of analytical work for meaningful end results is strongly recommended.

II. COMMON CONSIDERATIONS: SUMMARY

SUMMARY: COMMON CONSIDERATIONS

Unit II provided common considerations for evaluating the stability of an embankment dam, including:

- Documentation review
- Instrumentation
- Modes of failure
- Embankment dam analysis

Documentation Review

Reviewing project data will help you in several ways while evaluating the stability of an embankment dam under routine, everyday operations as well as under unusual conditions such as floods and earthquakes.

This section described . . .

- The types of data for review to determine an embankment dam's ability to perform under static and dynamic loading, and

- The types of dam and foundation materials that might help identify the type of embankment dam under study.

Instrumentation

This section briefly described the types of instruments that may be present or installed and the information they provide relative to assessing the stability of a dam.

Modes Of Failure

Various modes of dam failure under nonearthquake conditions and earthquake conditions were discussed.

Nonearthquake Conditions

Modes of failure related to nonearthquake conditions are due to design deficiencies, poor construction, and/or neglected remedial actions.

II. COMMON CONSIDERATIONS: SUMMARY

Earthquake Conditions

Modes of failure related to earthquake conditions include:

- Excessive deformations and/or excessive pore fluid pressure buildup.
- Sudden densification of loose, saturated, noncohesive soils that causes rapid buildup of pore fluid pressures.

Stability Problems

Stability problems of concern that may cause an embankment dam to fail include:

- Excessive deformations
- Excessive stresses
- Excessive loss of materials due to erosion

Embankment Dam Analysis

This section described analysis considerations for evaluating the stability of an embankment dam, the progress made in analytical procedures, and the limitations of analysis due to the complex nature of soil behavior.

Analysis of Existing Dams Versus New Dams

While there is no basic difference in methodology for analyzing existing dams or new dams, analyzing the performance of a new dam involves making certain predictions about how the dam will perform while analyzing an existing dam requires understanding the observed performance of the dam in order to make provisions for its continued safe operation.

Analysis Organization

Considerations in doing stability analysis for static conditions and for dynamic conditions were discussed.

UNIT III

SETTLEMENT AND DEFORMATION

III. SETTLEMENT AND DEFORMATION: OVERVIEW

INTRODUCTION

This unit discusses how settlement and deformation can affect the stability of an embankment dam. Background information on settlement and deformation includes:

- Types of deformations
- Deformations of interest
- Significance of settlements
- General deformation behavior

In addition to background information, this unit describes:

- What types of investigations and data collection should be made for problem verification.
- What methods may be used to analyze settlement and deformation problems.
- What remedial action can be taken to alleviate problems caused by settlement and deformation.

III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

INTRODUCTION

Failures of embankment dams, except for failures caused by unanticipated catastrophic events such as earthquakes or overtopping, are almost always preceded by warning signals such as increased rate of deformation, strain discontinuities, cracking, leakage, and pore pressure buildup.

These same warning signs may appear, yet be in no way associated with a potential failure. In order to first detect significant changes in the time rate of deformation, and second, to evaluate the probable causes and consequences of such changes, the following steps are necessary:

- Instruments must be present and in the correct locations in order to measure internal deformations.

- Periodic observation data must be available. The frequency of observations depends upon factors such as time rate of movement and rate of reservoir filling.

- The data must be summarized, plotted versus time and versus reservoir water level, and evaluated promptly by experienced engineers familiar with the dam and the general performance of similar dams.

- Any unanticipated anomalies in the data must be critically studied to determine their causes.

It is important to recognize that all embankment dams in service deform and settle. In general, deformations of embankment dams may result in aesthetically unacceptable surficial appearance. However, excessive deformations indicate internal distress of the dam, and can result in:

- Reduction or loss of freeboard, and/or

- Internal and/or external cracks.

Either of these two consequences of settlements and deformations can lead to dam failure. Historically, embankment dam failures have been attributed to sharply nonuniform settlements.

III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

INTRODUCTION (Continued)

Internal deformations in an embankment dam can be measured by measuring devices installed inside the dam. Surface expressions of internal deformations can be measured by surveying surface monuments from a stable benchmark with fixed lines of sight. Internal distress of an embankment dam and/or foundation may appear as muddy seepage, sand boils, ground heave downstream of the dam, and/or a whirlpool in the reservoir.

The purpose of settlement observations is to provide information concerning the amount, rate, and distribution of settlement. In order to gain the full benefit of observations, it is important that records containing the information are kept in an intelligent and conscientious manner. To be useful, records must be kept in such a manner that the data can be understood by any engineer without further inquiry and without the chance of misinterpretation.

TYPES OF DEFORMATIONS

The deformation response of an embankment dam depends upon its own response to load plus the response of the foundation on which it is built. Often, embankment dams are built on sites with compressible foundations. Sometimes the dam foundations are traversed by geologic discontinuities such as faults and shears near the dam-foundation contact.

The movement of a specific point within a dam or its foundation can be resolved into three orthogonal components:

- **Vertical Movement.** Usually called settlement if it is downward, and heave if it is upward.
- **Upstream/Downstream Movement.** Also called lateral movement.
- **Cross-Valley Movement Parallel to the Dam's Axis.** Also called longitudinal movement.

The total movement of a specific point within a dam or its foundation is made up of two parts:

- Elastic or recoverable part
- Inelastic or permanent part

III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

TYPES OF DEFORMATIONS (Continued)

Elastic movement is due to elastic behavior of a soil and occurs almost immediately on application of load. Inelastic movement in a soil deposit is due to the process of consolidation under load application, and readjustment of the soil skeleton.

Consolidation is a relatively slow process and the movement depends on the rate at which the water is able to move out of the soil matrix. Expansive soils, such as fat clays, absorb water and undergo volume increase. Thus, such soils may cause upward movement of the overlying soil or the structure supported by them.

In general, with regard to the usual settlement of soils, elastic movement is considerably less than consolidation movement and it is generally neglected.

DEFORMATIONS OF INTEREST

The three types of deformations of interest in an embankment dam are:

- Uniform or near-uniform movements
- Sharply nonuniform or differential movements
- Lateral movements

Uniform Movements

Uniform or near-uniform movements of points within an embankment dam do not usually cause internal straining or cracking in the soil deposit (see Figure III-1(a) on the next page). Uniform settlements do, however, reduce a dam's freeboard.

Differential Movements

Differential movements are relative movements between neighboring points or sections within an embankment dam, or in the foundation zones (see Figure III-1(b), (c), (d), (e) and (f)). Differential movements cause internal straining in an embankment and can lead to the formation of cracks in the soil mass. Excessive differential movements within the body of an embankment and/or its foundation (see Figure III-2) can lead to failure of the dam (J. L. Sherard, 1973).

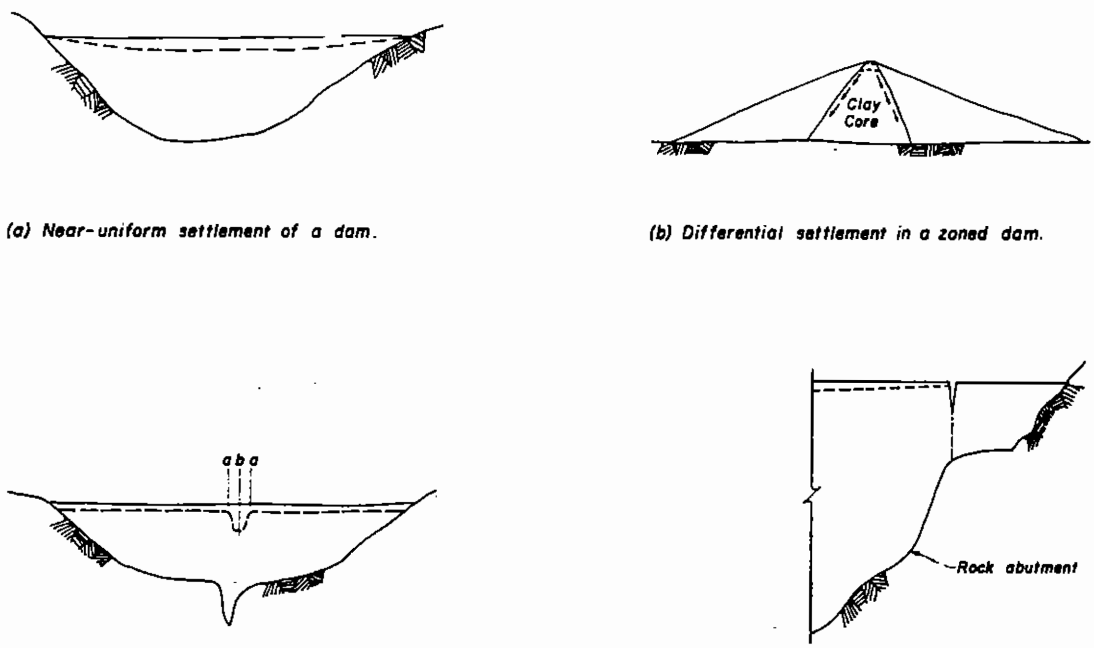
III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

Lateral Movements

Lateral movements may be due to spreading of an embankment dam (see Figure III-1(g)), and/or settlement of a foundation (Figure III-1(h)). Excessive spreading of an embankment may lead to the formation of longitudinal cracks which can lead to slope instability through the introduction of surface water runoff into the cracks. Longitudinal cracks have been observed in many embankments built on clay foundations.

It is instructive to note that the total movement in a soil deposit is a combination of uniform movement, differential movement, and lateral movement. In most cases, the critical movement is the differential movement because it leads to embankment cracking. Generally, compression of the foundation soils is responsible for excessive settlements of embankment dams.

FIGURE III-1. SCHEMATIC ILLUSTRATIONS OF EMBANKMENT DEFORMATIONS



(a) Near-uniform settlement of a dam.

(b) Differential settlement in a zoned dam.

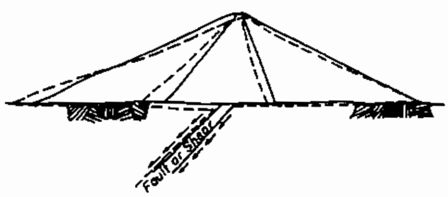
(c) Differential settlement between points a and b.

(d) Differential settlement over a sharp irregularity in rock surface.

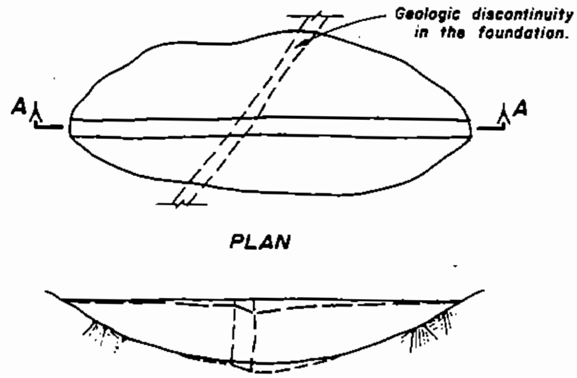
III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

Lateral Movements (Continued)

FIGURE III-1. SCHEMATIC ILLUSTRATIONS OF EMBANKMENT DEFORMATIONS
(Continued)

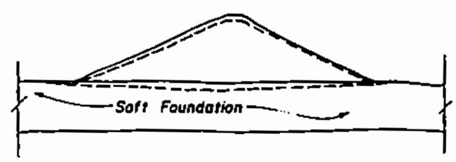


(e) Differential settlement in the foundation due to a geologic discontinuity across the valley.



(f) Differential settlement in the foundation due to a geologic discontinuity along the valley.

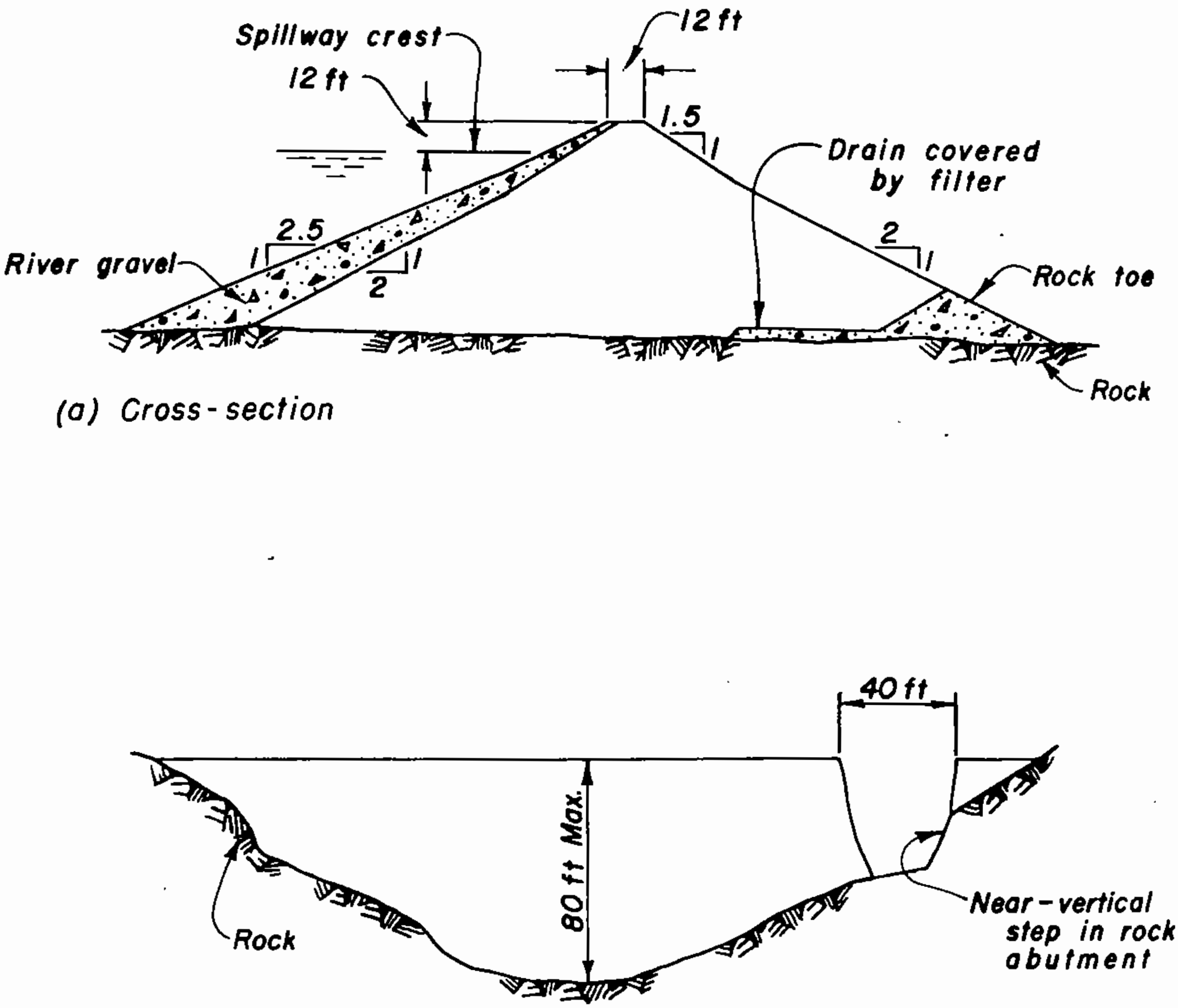
(g) Lateral spreading of a dam.



(h) Settlement of foundation.

Lateral Movements (Continued)

FIGURE III-2. STOCKTON CREEK DAM FAILURE



(a) Cross-section

(b) Longitudinal section looking downstream

III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

SIGNIFICANCE OF SETTLEMENTS

Of the three components of a general movement of a specific point within an embankment, the vertical component is of practical significance in safety of dams in the following ways:

- Excessive uniform settlements of a dam can lead to the loss of freeboard and danger of overtopping.

- Differential settlements between sections along the axis of a dam can lead to the development of transverse cracks through the embankment, core wall, etc., which could allow passage of water and progressive failure by erosion and piping.

- Differential settlements between different zones within the body of a dam can lead to the development of longitudinal cracks (e.g., the shell being more deformable than the core or vice versa).

- Differential settlements within a material and longitudinal arching action over a more deformable zone can lead to internal horizontal cracks. This differential settlement reduces the magnitude of vertical compressive stresses locally. If the compressive stresses at any location are reduced to values less than the pressure of the water in the reservoir at the same elevation, hydraulic fracturing of the embankment material may occur. This can lead to dam failure by erosion and piping.

GENERAL DEFORMATION BEHAVIOR

It is instructive to know the generally acceptable deformational response of an embankment dam in order to be able to detect deviations from the normal and identify possible problems. These deviations may or may not be significant since normal variations occur as a result of reservoir operations, temperature changes, earthquakes, etc. Also, there may be false "movements" attributable to bench mark disturbances, survey errors, or other instrument problems.

No two dams or damsites are alike. Therefore, one should not expect an exact repetition of distress signs from one dam to another even for the same problematic feature. However, when deviations from normal behavior are observed that cannot be readily accounted for, it is better to bring the observations to the attention of an experienced and qualified engineer. It may become essential to increase the frequency of instruments readings, install additional instruments, and undertake other investigations in order to understand the cause(s) of the observed deformations and seek effective remedies for the problem. A general description of acceptable deformational behavior of a typical embankment dam is given on the next page.

III. SETTLEMENT AND DEFORMATION: BACKGROUND INFORMATION

Acceptable Deformational Behavior

During construction of an embankment dam, internal deformations take place due to changes in total stresses and pore pressures and due to creep or secondary time effects. Foundation movements, load transfers between zones, and other factors influence the deformations. The time rate of deformations depends upon the rate of load increase and the type of material being used to construct the dam.

In general, an embankment spreads in the upstream-downstream direction and cross-valley movements take place towards the deepest part of the valley. The settlements occurring during the construction of a dam are absorbed as additional fill is placed to reach the required elevation. Camber is provided to account for any additional expected settlement. Thus, the as-built dam survey provides a good reference geometry for measuring dam and foundation movements for in-service conditions.

After the completion of a dam, the rate of load increase becomes zero and the settlement rate diminishes rapidly, only to increase again as the reservoir fills for the first time. The first filling of the reservoir may cause the crest to move upstream while at the same time the lower portion of the impervious core may deflect downstream. Subsequently, many embankments continue to settle indefinitely at a diminishing rate with respect to time, except for variations associated with periodic raising or lowering of the reservoir and with earthquakes.

The magnitude, time rate, and direction of movement of a specific point within a dam may change during different phases of construction of the dam and operations of the reservoir. Horizontal and vertical crest movements that develop in the first few months following completion of a dam are likely to be greater than the movements that occur over the next decade. The crest movements during the first few months following completion of a dam may be less than 25 percent of the movements that developed at midheight of the dam during construction.

For dams on relatively incompressible foundations, cambers of about one percent of the height of the dam are commonly provided to accommodate settlements of the foundations. A higher percentage of camber is provided for dams constructed on sites where the foundations may yield substantially under the embankment loads.

III. SETTLEMENT AND DEFORMATION: INVESTIGATION AND DATA COLLECTION FOR PROBLEM VERIFICATION

INTRODUCTION

At the time of design, estimates are often made of expected deformations. The records pertaining to design should be reviewed for this information, and if located, compared to the performance data relative to embankment movement.

If measured deformations significantly exceed the engineering design estimates, then it becomes essential to supplement the available performance, geologic, and material test data with additional investigations. The fact that the actual performance of the dam did not conform to the expected behavior could be due to incomplete geologic data and/or material behavior information available at the time of initial design and construction. To prepare an effective investigation and additional data collection program, the design engineers and geologists should review the following documents in addition to making a site inspection of the dam. It is generally advantageous to review these documents prior to making a site visit.

- Design and construction data
- Surveying data
- Instrumentation data
- Inspection reports and visual inspections

Based on a data review and site inspection, a program to obtain needed additional data can be developed and may include:

- Field explorations for additional geologic data and sample collection.
- Laboratory testing of soil and rock samples.
- Installation of instrumentation for additional performance data and/or more frequent observations of existing instruments.

It is prudent to make preliminary estimates of all probable cause(s) of the excessive settlements and their locations; i.e., within the body of the dam and/or foundation. Based on this estimate, the locations of sample collection, number of samples, and their types are specified. Similarly, the laboratory testing conditions in terms of loads and drainage conditions are specified to match the field conditions. Preferably the test samples should be undisturbed. The drainage conditions could be drained or undrained depending upon the type of deformation response to be investigated.

The type of test data to be collected should be consistent with the needs of analysis procedure(s) to be used for analyzing the settlement problems. Since relatively softer members in an embankment or foundation are responsible for excessive deformations, they should be sampled and tested in appropriate laboratories; i.e., soil mechanics and/or rock mechanics laboratories.

III. SETTLEMENT AND DEFORMATION: INVESTIGATION AND DATA COLLECTION FOR PROBLEM VERIFICATION

PROBLEM IDENTIFICATION

As mentioned earlier, all embankment dams deform and settle. It is only when the actual deformations exceed the design estimates that they need to be studied for possible cause(s).

The simplest physical manifestation of embankment deformation is that it renders the dam geometry somewhat irregular, which may or may not be noticeable to an untrained eye. However, through survey and other dam instrumentation data, one can determine the magnitude and direction of these movements at their respective locations.

If the deformations become excessive, they may cause:

- Visible cracks on the crest and/or faces of the dam.
- Hidden cracks within the body of the dam where they remain invisible.

Table III-1 on the following pages lists observations of different cracks, possible causes of their occurrence, seriousness, and suggested actions. It is imperative to treat the appearance of cracks or their consequences seriously and study them in the detail needed. It is better to come to the conclusion, after an investigation, not to do anything substantial about a crack, than to ignore its existence. If cracks are left unattended, the underlying problem may worsen and threaten the safety of the dam. Historically, as noted in Table III-1, all cracks in embankment dams are not necessarily serious--some cracks only need to be sealed at the surface to prevent the infiltration of water. However, a thorough evaluation of their cause(s) and possible consequences should be undertaken.

While individual modes of deformation in embankment dams and their foundations are shown schematically in Figure III-1, page III-5, actual deformations are generally a combination of several of these individual modes. Identifying the principal cause(s) of excessive deformations may require combined evaluation of site geology, dam design and construction, instrumentation data, and mathematical analyses.

TABLE III-1. CRACKING OF EMBANKMENT DAMS

Crack Types	Observation	Probable Cause	Seriousness	Suggested Action
I(a)[1]	Transverse vertical cracks, visible at crest, developing over steep abutments and extending from upstream to downstream.	Excessive differential settlement between adjacent x-sections of the dam probably due to abrupt changes in the abutment slopes and/ or longitudinal stretching of the embankment.	Potentially very serious as water can pass through these cracks and may lead to progressive failure by erosion and piping.	Immediate action: Impose reservoir level restrictions. Conduct a thorough investigation to determine cracks' depths and to identify their cause. Design and construct effective remedial fix.
(b)	Longitudinal vertical cracks, visible at the crest or above water level on the upstream face.	Settling and tilting of the upstream shoulder towards the reservoir and/or differential settlement between the core, filter zones, transitions, and rockfill. Incipient embankment slide.	Generally not dangerous. However, they could potentially lead to hydraulic fracturing.	Immediate action: Impose reservoir level restrictions. Identify the cause of cracks. If not attributed to slope instability, routine maintenance and restoration of surface may be O.K.

TABLE III-1. CRACKING OF EMBANKMENT DAMS
(Continued)

Crack Types	Observation	Probable Cause	Seriousness	Suggested Action
II(a)[2]	Transverse horizontal hidden cracks, possibly extending from upstream to downstream especially in central core dams. Excessive seepage through the dam may be the only sign of internal distress. Concentrated leaks are possible.	Differential settlement and longitudinal arching action between steep abutments in narrow canyons, or locally over a closure section, an old river channel, a fault channel, or a buried conduit.	Very serious. Potential for hydraulic fracture exists. Water can pass through these cracks and may lead to progressive failure by erosion and piping.	Immediate action: Impose reservoir level restrictions. Identify the cause of the cracks. Design and construct effective fix.
(b)	Transverse horizontal hidden cracks in thin central core dams extending from upstream to downstream. Excessive seepage through the dam may be the only sign of internal distress. Concentrated leaks are possible.	Differential settlement between the narrow core and the shoulders. The lower portion of the core settles by consolidation under self weight (when placed too wet) or by wetting compression (when placed too dry).	Very serious. Potential for hydraulic fracture exists. Water can pass through these cracks and may lead to progressive failure by erosion and piping.	Immediate action: Impose reservoir level restrictions. Identify the cause of the cracks. Design and construct effective fix.

TABLE III-1. CRACKING OF EMBANKMENT DAMS
(Continued)

Crack Types	Observation	Probable Cause	Seriousness	Suggested Action
(c)[3]	Horizontal upstream to downstream local hidden cracks within a homogeneous embankment dam.	Saturation collapse settlement due to reservoir filling. The critical zone or layer can result from variations in borrow material, shrinkage cracking of the construction surface during work stoppage; dry placement water content; and inadequate compaction control.	Very serious.	Immediate action: Impose reservoir level restrictions. Identify the cause of the cracks. Design and construct effective fix.
(d)	Horizontal upstream to downstream hidden cracks adjacent to steep abutments or overhangs and abrupt irregularities.	Consolidation or wetting compression.	Very serious.	Immediate action: Impose reservoir level restrictions. Identify the cause of the cracks. Design and construct effective fix.

TABLE III-1. CRACKING OF EMBANKMENT DAMS
(Continued)

Crack Types	Observation	Probable Cause	Seriousness	Suggested Action
III	Loss of drilling fluid in bore holes.	Suspected crack.	Serious.	Quit drilling with fluid under pressure as it may cause crack extensions by hydraulic fracturing mechanism.

[1] Type I cracks by themselves have not led to any serious problem or erosion failure. They have maximum width at the surface, and can be observed and repaired.

[2] Type II cracks have been responsible for serious damage and failure by internal erosion. Type II cracks develop from the inability of an upper portion of embankment to follow the settlement of a lower portion. Thus, they can best be examined in terms of compressibility and deformation characteristics of embankment, foundation and abutment materials, and settlement calculations.

[3] Saturation collapse settlement involves water penetration and settlement occurring instantaneously from upstream to downstream along pervious zones and layers of dry and loose material, before the denser material arching over has a chance to become saturated and collapse. Compacted soils most susceptible to internal erosion and brittle cracking are also most likely to experience saturation collapse in the loose, dry lifts that they contain. See the discussion by G. Mesri and S. Ali to the paper entitled Hydraulic Fracturing In Embankment Dams, published in the ASCE Geotechnical Engineering Journal.

III. SETTLEMENT AND DEFORMATION: ANALYTICAL METHODS

INTRODUCTION

Actual settlement response data can be analyzed to predict final total settlement and the settlement rate for an embankment dam using the graphical procedure discussed under Graphical Evaluation of Settlement Records on page III-19. Alternatively, numerical analysis procedures may be used to study settlement response of an embankment dam.

ANALYSIS OF SETTLEMENT PROBLEMS

Mathematical analyses for the settlement response of an embankment dam and its foundation can be made to gain additional information about the dam behavior for the imposed conditions. The objectives of making a mathematical analysis can be:

- **To provide a check on the adequacy of an analytical model.** To properly meet this objective, it is essential to know the site geology, composition, physical properties of the materials in the foundation and embankment, and field pore pressure data prior to making a mathematical analysis of the problem.

- **To estimate material property values used in an analytical model to obtain a comparable match between the calculated results and observed performance data.** To properly meet this objective, it is essential to know the site geology. In addition, laboratory and field test data and historical data for similar geologic settings and material descriptions should be used to check the reasonableness of numerically estimated values of material parameters. It is better to keep the numerical models simple and manageable. The paper entitled **Analysis of Foundation Settlements at Ridgeway Dam**, included in Appendix C, illustrates this point.

- **To predict future settlement behavior.** To properly meet this objective, it is advisable to keep in mind that while interpolation for results may be reasonable, extrapolation for results is generally not reasonable. It may be advantageous to use the graphical procedure discussed under Graphical Evaluation of Settlement Records on page III-19.

ANALYSIS OF SETTLEMENT PROBLEMS (Continued)

Theoretical analyses are performed to predict, usually ahead of construction and observations, anticipated behavior to arrive at an acceptable design. The embankment and its foundation, including abutments, function together and thus constitute the problem for engineering analysis and design. The available geologic information is used to define the foundation part of the problem; the selection of materials and their placement within the body of a dam defines the embankment part of the problem. Since the load-deformation behavior of soils and most other geologic materials is generally nonlinear, strain softening, and time dependent, the construction sequence and rate of reservoir filling play a significant role in the buildup of deformations, and hence stresses, in the dam and its foundations. It is an objective of design to make all this "inservice" behavior occur in a predictable manner and within anticipated limits.

All theoretical analyses make simplifying assumptions in the problem definition, material behavior, load simulation, and boundary conditions. Therefore, the results of theoretical analyses are estimates which need to be verified by the actual response data.

PRINCIPAL STEPS IN A SETTLEMENT ANALYSIS

The necessary information required for a settlement prediction is summarized in Table III-2 on the following page. A prediction capability consists of three components:

- A model to describe soil behavior;
- Suitable methods to evaluate the required soil parameters; and
- Computational procedures for applying the model to practical problems.

So far, it has not been possible to describe soil response by a single model for all loading and drainage conditions. Therefore, prediction of settlements is more of an art than a routine procedure. There are several methods and practices available for use in predicting settlements of structures. Their use in engineering practice is a matter of individual or organizational preference and past experience. Brief descriptions of a conventional method and of a more modern method are included here. For details of these and other methods of settlement calculations, consult the publication <u>Soft Clay Engineering</u> by A. S. Balasubramaniam and R. P. Brenner, (1981).

PRINCIPAL STEPS IN A SETTLEMENT ANALYSIS (Continued)

TABLE III-2. COMPONENTS OF A SETTLEMENT ANALYSIS

Determination of Subsoil Section (1) Vertical and lateral extent of soils; location of compressible soils, drainage surfaces, and any special boundary conditions. (2) Variation of initial pore pressure with depth.
Stress Analysis (1) Initial effective stress versus depth. (2) Magnitude, distribution, and time rate of application of surface load, including any shear stress between ground surface and applied load. (3) Stress distribution theory compatible with boundary conditions; effect of rigid boundaries or layers. (4) Variation of principal stresses σ_1, σ_2, and σ_3 with consolidation; influence of arching, change in Poisson's ratio.
Selection of Soil Parameters (m_v, C_c, C_r, C_α, σ'_{vc}, k, E_u, E', K_o, υ', A, C_v)* (1) Representativeness of samples tested. (2) Sample disturbance. (3) Environmental factors. (4) Testing technique.
Estimation of Settlement and Pore Pressures (1) Method of analysis. (2) Rotation of principal planes. (3) Variation of m_v, k, C_v with consolidation. (4) Secondary compression.

*Definitions of these parameters are:

m_v--coefficient of volume compressibility; C_c--compression index; C_r--recompression index; C_α--coefficient of secondary compression; σ'_{vc}--preconsolidation pressure; k--coefficient of permeability; E_u--elastic undrained Young's modulus; E'--drained Young's modulus; υ'--drained Poisson's ratio; K_o--coefficient of earth pressure at rest; A--Skempton's pore pressure coefficient; C_v--coefficient of consolidation.

III. SETTLEMENT AND DEFORMATION: ANALYTICAL METHODS

GRAPHICAL EVALUATION OF SETTLEMENT RECORDS

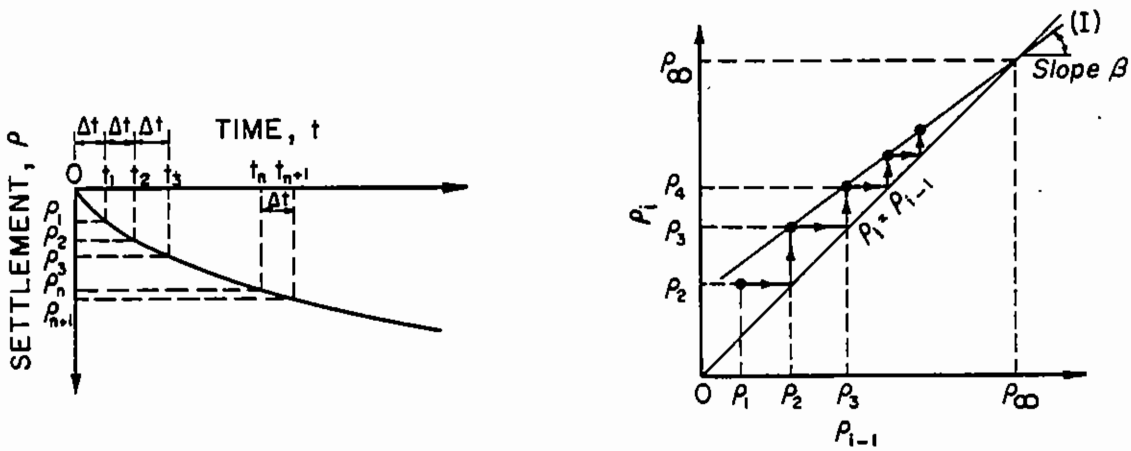
A practical approach by which to estimate final total settlement and settlement rates from settlement data obtained during a certain time period involves the following steps (A. S. Balasubramaniam and R. P. Brenner, 1981):

- The observed time-settlement curve plotted to an arithmetic scale is divided into equal time intervals, Δt, (usually Δt is between 30 and 100 days). The settlements ρ_1, ρ_2, . . . corresponding to the times t_1, t_2, . . . are read off and tabulated (see Figure III-3 below).

- The settlement values ρ_1, ρ_2 . . . are plotted as points (ρ_{i-1}, ρ_i) in a coordinate system with axes ρ_{i-1}, and ρ_i, as shown in Figure III-3(b). Draw the 45°, $\rho_i = \rho_{i-1}$, line.

- A straight line, I, is fitted through the data points. The point where this line intersects the 45° line gives the final consolidation settlement, ρ_∞. The slope β of line I is related to the coefficient of consolidation, C_v, and therefore indicates the rate of settlement. The slope, β, depends on the time step, Δt, selected and decreases when Δt increases.

FIGURE III-3. STEPS FOR THE USE OF GRAPHICAL METHOD



(a) Partition of settlement record into equal time intervals.

(b) Plot of settlement values and fitting of straight line.

III. SETTLEMENT AND DEFORMATION: ANALYTICAL METHODS

CONVENTIONAL ONE-DIMENSIONAL METHOD

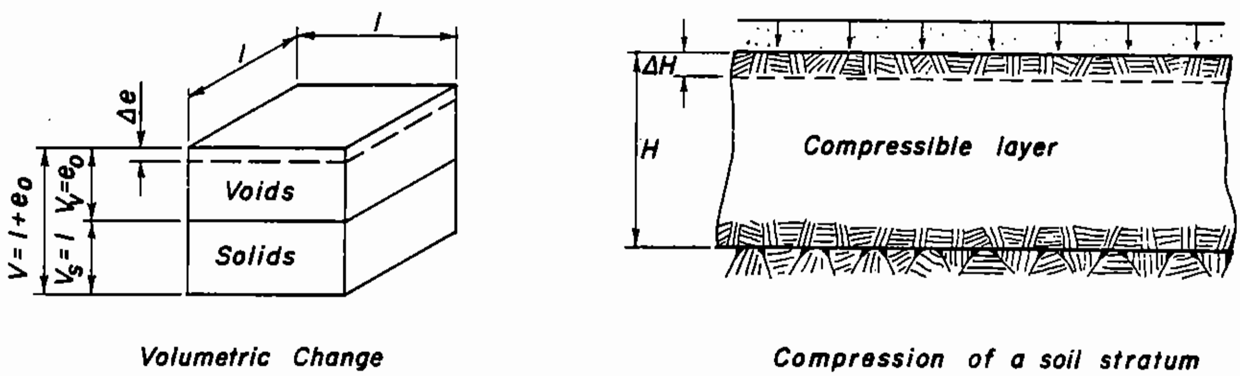
The analytical model for this method of settlement calculations is shown on Figure III-4 below. The settlement calculations by this method involve two steps:

- An assessment of the stress increment in the compressible layer by the surface load, and

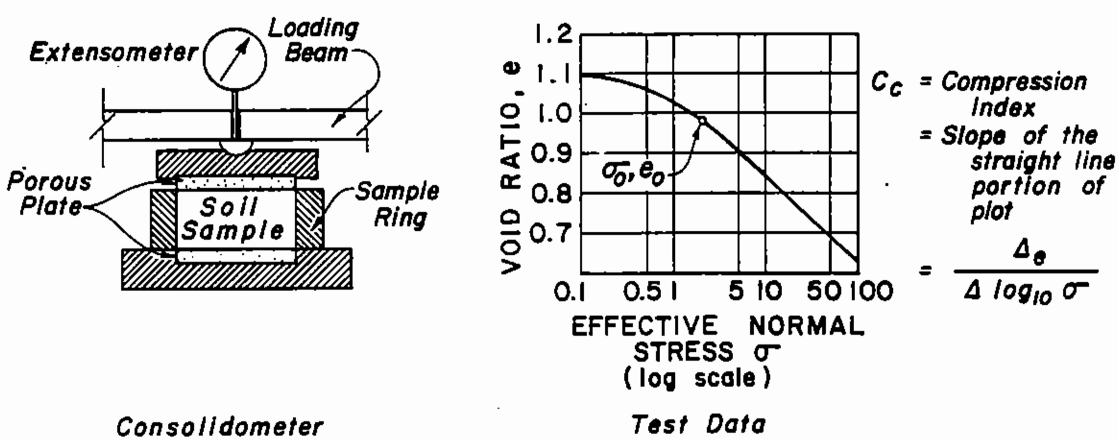
- An evaluation of the settlement caused by the stress increment using an appropriate stress-strain relationship from odometer tests on the compressible soil (see Figure III-4).

The paper entitled **Analysis of Foundation Settlements at Ridgeway Dam**, included in Appendix C, illustrates the use of this method.

FIGURE III-4. CONVENTIONAL CONSOLIDATION SETTLEMENT METHOD



(a) Conventional one-dimensional analytical model.



(b) Laboratory test for one-dimensional consolidation.

III. SETTLEMENT AND DEFORMATION: ANALYTICAL METHODS

FINITE ELEMENT METHOD

The basis for the finite element method is the representation of a body or a structure by an assemblage of its subdivisions called finite elements. These elements are interconnected at nodal points to form a finite element model. Solutions are obtained in terms of displacements at these nodal points and average stresses in the elements. The stress-strain relationship of the material in each element is used to formulate its stiffness matrix.

Individual element stiffness matrices are assembled to form an overall stiffness matrix for the entire soil mass with specified boundaries. The stiffness matrix relates the nodel displacements to the nodal loads. Such a procedure can take into account various types of stress-strain behavior, nonhomogeneity, irregular geometry, and complex boundary conditions, as well as time-dependent loading.

Among the material models which describe the stress-strain behavior, the linear elastic one is of the simplest form, and this gives useful results when the elastic parameters are evaluated over the appropriate stress range. Other models commonly used are:

- The bilinear elastic model
- The elasto-plastic model
- Hyperbolic relationships
- The critical state model

Except for the linear elastic model, all other models need the shear strength of the soil, and initial geostatic stress values. Thus, estimates of the in situ K_o value, that is, coefficient of earth pressure at rest, must be made. In order to obtain the initial settlement, the finite element analysis is carried out assuming completely undrained conditions and using undrained stress-strain deformation properties. The final settlement is obtained from an analysis employing drained deformation properties and effective stresses.

The paper entitled **Analysis of Foundation Settlements at Ridgeway Dam**, included in Appendix C, illustrates the use of linear elastic analysis by the finite element procedure.

III. SETTLEMENT AND DEFORMATION: REMEDIAL ACTIONS

INTRODUCTION

It is imperative to take settlement issues seriously and take action quickly. These actions may be temporary in nature, as permanent solutions to the problem will usually involve elaborate and time consuming investigative, design, and construction activities.

EMERGENCY AND TEMPORARY MEASURES

The objective of the following emergency-type remedial actions is to safeguard an embankment against a potential overtopping or piping failure.

- Sand bagging to restore freeboard.
- Restricting of the reservoir water level.
- Visual monitoring of the dam and evaluation of the instrumentation data pertaining to deformations.

Emergency action planning should account for potential overtopping if there has been a loss of flood surcharge freeboard.

LONG-TERM MEASURES

Unfortunately, there are no simple and quick long-term remedial fixes for settlement-related dam safety problems since they are internal to the embankment and/or foundation. Dams that have suffered damage or failed from internal erosion were deficient in certain aspects of design or construction leading to transverse cracking and uncontrolled erosion. Therefore, if type II cracks (Table III-1) develop, or a potential for their occurrence exists, an embankment modification to provide adequate transition and filter zones is needed to prevent erosion through a transverse crack. Once a through crack is formed, it is virtually impossible to contain the resulting seepage and erosion without lowering the reservoir to a level below the base of the crack. It is generally agreed that all embankment dams in service either crack or have the potential to develop cracks. Therefore, there is no real substitute to an adequate design of the original constructed facility.

III. SETTLEMENT AND DEFORMATION: REMEDIAL ACTIONS

LONG-TERM MEASURES (Continued)

A thorough investigation to understand the cause(s) and extent of a settlement problem should be undertaken. Depending upon the findings of this investigation, the following remedial measures, shy of reconstructing portions of the dam, may be adequate:

- Restore the dam to design crest elevation.
- Place weighted filters over areas where muddy seepage discharges occur or sand boils appear.
- Provide relief wells to relieve excess pore pressures at depth.
- Install additional instruments to closely monitor the dam performance and increase the frequency of data collection and its prompt evaluation.
- Install an early warning system to alert the dam attendant and engineers of a potential problem developing.

SUMMARY: SETTLEMENT AND DEFORMATION

Unit III described how settlement and deformation can affect the stability of an embankment dam, including information on:

- Types of deformations
- Deformations of interest
- Significance of settlements
- General deformation behavior

Also described were modes of failure related to settlement and deformation.

Settlements can lead to:

- Loss of freeboard and potential for overtopping
- Development of transverse cracks
- Development of longitudinal cracks
- Development of internal cracks

Normal deformations of an embankment dam occur as a result of reservoir operations, temperature changes, earthquakes, etc. This Unit described acceptable deformation behavior that may occur in an embankment dam.

Investigation And Data Collection

Unit III also described the types of documents to be reviewed and how to identify problems during a site visit, including a description of different types of cracks, their probable cause, seriousness, and suggested action.

Analytical Methods

Objectives of making a mathematical analysis can be:

- To provide a check on the adequacy of an analytical model.

- To estimate material property values used in an analytical model to obtain a comparable match between the calculated results and observed performance data.

- To predict future settlement behavior.

Theoretical analyses are performed in the design stage to predict anticipated behavior. However, results of theoretical analysis need to be verified by the actual response data.

III. SETTLEMENT AND DEFORMATION: SUMMARY

Principal Steps In A Settlement Analysis

The components of a settlement analysis consist of determination of subsoil section, selection of soil constitutive model and soil parameters, and estimation of pore pressures and settlement.

The following methods to analyze and evaluate settlement were briefly described:

- Graphical evaluation of settlement records
- Conventional one-dimensional method
- Finite element method

Remedial Actions

Temporary and long-term measures to alleviate the effects of settlement and deformation were stated.

UNIT IV

STATIC STABILITY

IV. STATIC STABILITY: OVERVIEW

INTRODUCTION

This unit discusses the static stability concerns in embankment dams. Background information is presented on significant events that can cause static instability and indicators of static instability.

In addition to background information on static instability and its effects, this unit describes:

- What types of project data must be reviewed and analyzed for static stability evaluations.
- What tests are used to obtain soil shear strength values.
- What methods may be used to analyze the static stability of an embankment dam.
- What remedial action can be taken to alleviate static instability.

IV. STATIC STABILITY: BACKGROUND INFORMATION

INTRODUCTION

If design stability analyses have been performed for an existing embankment dam, they should normally be sufficient if they were performed by acceptable methodologies. Additional stability analyses should be performed if . . .

- Existing analyses are not in agreement with the current accepted methodologies,
- Existing conditions have deteriorated,
- The hazard potential of the project has increased,
- The embankment has been or will be subjected to loading conditions more severe than designed for, or
- The assumed design parameters cannot be satisfactorily justified.

Satisfactory behavior of an embankment under loading conditions not expected to be exceeded during the life of the structure should be considered indicative of satisfactory stability, provided that adverse changes in the physical condition of the embankment do not occur.

SIGNIFICANT EVENTS FOR STATIC INSTABILITY

Significant events that can cause static instability of an existing embankment dam after years of satisfactory service include:

- An unusually severe drawdown of the reservoir. The severity of drawdown can be in terms of a more rapid rate or to a lower level than before.
- An unusually high and perhaps sustained reservoir water level.
- A prolonged dry period followed by rain. The dry period can cause desiccation cracks to develop in some dams; subsequent rain can fill the cracks with water and precipitate slides.
- Gradual development of an adverse seepage pattern through the dam and/or its foundation.
- Gradual loss of strength in clay shales or overconsolidated clays due to swelling.

IV. STATIC STABILITY: BACKGROUND INFORMATION

STATIC INSTABILITY INDICATORS

A need for evaluating the static stability of an existing embankment dam and its foundation is indicated if:

- There is an apparent slope stability failure.
- There are longitudinal cracks on the dam crest or slopes.
- There are wet areas on the downstream slope or toe portion of the dam.
- There is erosion or sloughing near the downstream toe of the dam resulting in local oversteepening of the downstream slope.
- Surface measurement points indicate movements.
- Internal instrumentation indicates movement.
- Internal instrumentation indicates excessive pore pressures in the dam and/or foundation.
- There are bulges in the ground surface beyond the toes of the slopes.
- There is a need for dynamic stability analysis.
- Review of design and construction records indicate the presence of previously unrecognized but potentially harmful geologic conditions.

Sometimes performing a static slope stability analysis of an existing embankment dam and its foundation may be required prior to raising the dam, or for changed reservoir operations. But these are not safety of dams issues in the conventional sense. Thus, they are not covered in this module.

IV. STATIC STABILITY: REVIEW AND EVALUATION OF PROJECT DATA

INTRODUCTION

If, for whatever reason, the need for slope stability evaluation of an existing dam is established, it becomes essential to reassess the site geology and material property data for the dam and foundation materials for the following reasons:

- If the performance of the existing dam is not commensurate with the design intent, confidence in the previous understanding of the site geology and/or material property data becomes less than desirable.

- If there are time-dependent influences causing actual performance to deviate from the design intent, the new developments need to be explored, and their effects on material properties assessed.

- If a dam safety modification has to be designed, the properties of borrow area materials available for construction must be determined.

For a proper use of mathematical or judgmental approach, some testing of in situ materials will be required to determine the current values of material properties, as discussed later in this unit. Additional field exploration may be warranted to gain additional site geologic data.

DESIGN, AS-BUILT CONSTRUCTION, AND PERFORMANCE DATA

Prior to undertaking analytical work for stability assessment of an embankment dam, it is essential to review the design, construction, and performance records on the project. The objectives in reviewing these reports should be to:

- Become knowledgeable about the site geology and geologic materials in the dam and its foundation.

- Seek to identify the cause and effect in the observed response.

- Identify the need for additional data in terms of geologic investigations and material testing.

The design documents that should be reviewed include:

- Geologic reports and geologic logs
- Laboratory test reports and laboratory data
- Design calculations and design assumptions

IV. STATIC STABILITY: REVIEW AND EVALUATION OF PROJECT DATA

DESIGN, AS-BUILT CONSTRUCTION, AND PERFORMANCE DATA (Continued)

The construction documents that should be reviewed include:

- Construction specifications
- As-built drawings
- Construction quality control test reports
- Correspondence that may highlight design changes or problems
- Construction incidents

The performance reports that should be reviewed include:

- Instrumentation reports (including piezometer response data, movement device data, and corresponding reservoir level data)
- Reports on any adverse incidents

Dam safety inspection reports should be reviewed to correlate visual observations with the available geologic information and material test data.

After an office review of the available design and construction information on the embankment dam under study, a visit to the damsite by those involved in the stability assessment of the dam is recommended. The field visit allows you to gain a visual perspective of the size, scale, and proportion of the problem under study in relationship to the geologic environment and other components of the project.

LOADING CONDITIONS

Stability problems in embankment dams are almost always preceded or accompanied by seepage problems. It is, therefore, essential to understand the seepage occurring through the dam and its foundation prior to doing stability analysis. The loading conditions for stability analysis of an existing embankment dam include:

- Steady-state seepage conditions
- Reservoir operating conditions
- Unusual conditions

The following is a brief description of these loading conditions.

IV. STATIC STABILITY: REVIEW AND EVALUATION OF PROJECT DATA

Steady-State Seepage Conditions

The annual reservoir operation plan should be examined to determine the appropriate reservoir water surface elevation for use in estimating the location of the steady-state phreatic surface. Usually, the appropriate elevation represents the water surface elevation that prevails most of the time. However, under certain reservoir operations, the average elevation is reached for only a small fraction of time each year or it is reached in an oscillatory cycle with the effective reservoir elevation near the midpoint of the cycle. The condition of steady seepage throughout an embankment may be critical for downstream slope stability.

Reservoir Operation Conditions

The following reservoir operation conditions are considered for static stability evaluation:

- **Maximum reservoir level.** A phreatic surface should be estimated for the maximum reservoir level which may occur in a surcharge pool that drains relatively quickly or in a flood control pool that is not to be released for several months. If the phreatic surface is significantly different from that of the steady-state condition, then the downstream stability under this condition should be analyzed.

- **Rapid drawdown conditions.** During the steady-state condition, embankments become saturated by seepage. Subsequently, when the reservoir is drawn down faster than pore water can drain from the soil voids, excess pore water pressure and unbalanced seepage forces result. In general, rapid drawdown analyses are based on the conservative assumptions that:

    - Pore pressure dissipation does not occur in impervious material during drawdown; and

    - The phreatic surface on the upstream slope coincides with the upstream slope of the impervious zone and originates from the top of the lowest drawn down water surface level. However, the critical elevation of drawdown with regard to upstream stability of an embankment may not necessarily coincide with the minimum reservoir elevation, and thus, intermediate drawdown levels should be considered.

Unusual Conditions

Unusual conditions considered are:

- Inoperable internal drainage, and
- Unusual drawdown.

Appropriate estimates of internal pore pressures in the embankment and foundation materials should be made to reflect the severity of the unusual conditions and the stability of the dam evaluated. If questions arise as to the proper functioning of the internal drains, they should be assumed inoperable for analysis.

IV. STATIC STABILITY: SOIL SHEAR STRENGTH

INTRODUCTION

It is necessary to know or estimate the shear strength of embankment and foundation soils in order to perform static stability analysis.

SHEAR STRENGTH TESTS

Shear strength values are generally based on laboratory tests performed under three conditions of test specimen drainage. Tests corresponding to these drainage conditions are: (Engineering and Design Geotechnical Investigations, Army Corps of Engineers, 1984; **Embankment Dam Design Standard for Static Stability Analyses,** Bureau of Reclamation, 1987; Engineering Guidelines for the Evaluation of Hydropower Projects, Federal Energy Regulatory Commission, 1988).

- Unconsolidated-undrained (Q) test in which the water content is kept constant during the test.

- Consolidated-undrained (R) test in which consolidation or swelling is allowed under initial stress conditions, but the water content is kept constant during application of shearing stresses.

- Consolidated-drained (S) test in which full consolidation or swelling is permitted under the initial stress conditions and also for each increment of loading during shear.

The appropriate Q, R, and S tests should be selected to reflect the various prototype loading cases and drainage conditions. Normally, shear strength tests are made with triaxial compression apparatus. However, S tests on fine-grained soils are usually made with direct shear apparatus. When impervious soils contain significant quantities of gravel sizes, S tests should be performed on triaxial compression apparatus using large-diameter specimens.

Strength Tests For Steady-State Seepage Condition

- The consolidated-drained triaxial shear test or the consolidated-undrained triaxial shear test with pore pressure measurements is appropriate.

- The direct shear test is appropriate for sands and sandy or silty clays. It could be used for plastic clays; however, the required rate of shearing would be very slow and may not, therefore, be very practical.

- The unconsolidated-undrained triaxial shear test is appropriate for very soft clays.

IV. STATIC STABILITY: SOIL SHEAR STRENGTH

Strength Tests For Rapid Drawdown Condition

- The consolidated-undrained triaxial shear test with pore pressure measurements is appropriate for impervious and semipervious soils because such tests provide both effective stress shear strength parameters as well as undrained shear strength as a function of consolidation stress. Sufficient back pressures should be used to effect 100 percent saturation to ensure accurate pore pressure measurements.

- The consolidated-drained triaxial shear test or direct shear test can be used if the material is highly permeable, that is, permeability greater than 10^{-4} cm/s.

- For overconsolidated clay shales, consolidated-undrained triaxial shear tests with pore pressure measurements, consolidated-drained triaxial shear tests, or direct shear tests may be used. Where potential slip surfaces follow existing shear planes, residual shear strengths from repeated direct shear tests or rotational shear tests are appropriate.

Selection Of Shear Strength Values

The selection of the proper soil parameters and their correct use in a stability analysis are generally of greater importance than the method of stability analysis used. When shear strength values are selected from shear strength test data, the shape of the stress-strain curves for individual soil tests is considered.

Where undisturbed foundation soils and compacted soils do not show a significant drop in shear or deviator stress after peak stresses are reached, the shear strength value can be chosen as the peak shear stress in S direct shear tests, the peak deviator stress, or the deviator stress at 15 percent strain where the shear resistance increases with strain.

For each soil type, a shear strength value should be selected such that two-thirds of the test values exceed the chosen shear strength value.

☞ Sometimes stability analyses of an existing embankment dam and its foundation are made using estimated values for their material properties. These estimates for material property values are based on:

- Past laboratory test reports on the project under study.

- Past experiences in testing similar materials on other projects.

Selection Of Shear Strength Values (Continued)

While this may be an acceptable practice for preliminary work in a dam safety evaluation process, it is essential that final evaluations and recommendations for remedial work be based on material property values obtained from appropriate laboratory and field tests on a site-specific basis.

It is always a good idea to compare the tested values with the historical data on similar materials or empirical relations, and to resolve the differences. The end objective is to get the best representative property values for the materials involved.

IV. STATIC STABILITY: ANALYTICAL METHODS

INTRODUCTION

In general, two different analysis approaches are available to assess embankment dam stability (K. Terzaghi and R. B. Peck, 1967). These are:

- Effective stress analysis
- Total stress analysis

EFFECTIVE STRESS VERSUS TOTAL STRESS ANALYSIS

In effective stress analysis, the shear strength of a soil is evaluated on the effective normal stress basis and explicit account is taken of the pore pressures in the stability analysis calculations. In total stress analysis, the shear strength of soil includes the effect of pore pressure. The two approaches are expected to yield identical factor-of-safety results for a shear surface provided the correct shear strength and the corresponding pore pressure data are used in the calculations. Thus, the choice of analysis approach may be based on:

- Convenience of use
- Convenience of testing and data collection
- Availability of computational procedure

In embankment dam engineering, however, effective stress analysis is commonly used because it facilitates proper understanding of the relative response of each constituent in the soil matrix. Thus, to properly perform an effective stress stability analysis of an embankment dam, it is necessary to know:

- Pore pressures in the dam and foundation materials.

- Forces exerted by the water as it seeps through the dam and foundation materials.

The determination of these two items is discussed on the following page.

IV. STATIC STABILITY: ANALYTICAL METHODS

Pore Pressure

The ideal means of knowing pore pressures in an existing embankment dam and its foundation is by the piezometric data. This requires that there be:

1. A sufficiently large number of piezometers installed at appropriate locations within the body of the dam and its foundation;

2. A reliable record of piezometer readings and the corresponding reservoir water levels, preferably in plotted form, over an extended period of time; and

3. A reliable means available to calculate pore pressure at a desired location from the discrete pore pressure data (A. K. Chugh, 1981).

In the absence of piezometric data, seepage analysis can be performed using a numerical model for the problem (A. K. Chugh and H. T. Falvey, 1984). The pore pressures can be defined by the calculated phreatic line, or by the calculated pore pressure values at discrete locations in the dam and its foundation.

Seepage Force

The seepage force on an element of soil is calculated by multiplying the volume of the soil element, unit weight of water, and the hydraulic gradient. The seepage forces in an embankment dam and foundation materials can be calculated from either the piezometric data or the seepage analysis results.

☞ **NOTE:** Sometimes for expedience, seepage analyses are not performed. Instead, a high phreatic line is drawn on the embankment cross section under study, and pore pressures along the shear surface are calculated on the basis of hydrostatic pressure distribution. However, this practice for defining pore pressures for stability analysis is neither suggested nor recommended. Also, explicit inclusion of the seepage force on the slide mass in slope stability analysis is generally not made. For proper stability analysis, it is suggested that it be included in the calculations.

IV. STATIC STABILITY: ANALYTICAL METHODS

STATIC STABILITY ANALYSIS METHODS

There is no basic difference in methodology between static stability analyses of new and existing dams. However, the analysis efforts should be commensurate with the quality and quantity of input data available. The commonly used static stability analysis methods are:

- Limit equilibrium method
- Finite element method

The limit equilibrium method is generally used to perform slope stability analyses. The finite element method is more versatile and is used for complete analysis of the stresses and movements in embankment dams under static conditions. The two methods generally give similar average factors of safety for a shear surface.

For more information on the limit equilibrium method of analysis, consult the following references listed in Appendix B:

- **The Use of the Slip Circle in Stability Analysis of Slopes** by A. W. Bishop
- **Variable Factor of Safety in Slope Stability Analysis** by A. K. Chugh
- **Suggestions for Slope Stability Calculations** by A. K. Chugh and J. D. Smart
- **Analytical Methods for Slope Stability Analysis** by D. G. Fredlund
- <u>Embankment Dam Engineering</u> edited by R. C. Hirschfield and S. J. Poulos
- **The Analysis of the Stability of General Slip Surfaces** by N. R. Morgenstern and V. E. Price
- **A Method of Analysis of the Stability of Embankments Assuming Parallel Interslice Forces** by E. Spencer
- **Static Analysis of Embankment Dams**, International Commission on Large Dams
- <u>Soil Mechanics in Engineering Practice</u> by K. Terzaghi and R. B. Peck

For more information on the finite element method of analysis, consult the following references listed in Appendix B:

- **Analysis of Embankment Stresses and Deformations** by R. W. Clough and R. J. Woodward, III
- **Elastic-Plastic Stability Analysis of Mine-Waste Embankments** by E. L. Corp, R. L. Schuster, and M. M. McDonald
- **Analytical Methods for Slope Stability Analysis** by D. G. Fredlund
- <u>Advanced Dam Engineering for Design, Construction and Rehabilitation</u>, edited by R. B. Jansen
- **Static Analysis of Embankment Dams**, International Commission on Large Dams

STATIC STABILITY ANALYSIS METHODS (Continued)

Seepage analysis is performed separately and its results, in the form of pore pressures and seepage forces, are included in stability analysis calculations. It is a common practice to analyze static stability of embankment dams in two dimensions.

LIMIT EQUILIBRIUM METHOD

In this method, a qualitative estimate of factors of safety can be obtained by examining the conditions of equilibrium when incipient failure is postulated, and comparing the available shear strength with the shear force in the soil. The factor of safety is thus defined as the ratio of the total shear strength available on a failure surface to the total shear force along the failure surface required to reach a condition of limiting equilibrium.

There are several slope stability analysis procedures developed based on the limit equilibrium method. Each procedure subscribes to a different set of assumptions in order to make the slope stability problem statically determinate. Some procedures do not satisfy all conditions of equilibrium. Table IV-1 on the next page lists the knowns and the unknowns of a slope stability problem and some of the commonly used procedures and their assumptions.

It is suggested that the adopted procedure for the slope stability analysis of an embankment dam should satisfy all conditions of statics, that is, force and moment equilibrium. Proper use of these methods requires information about layout of different soils in the embankment and foundation zones, soil properties in terms of unit weight and shear strength, pore water pressures, and shear surface. The slide mass is divided into slices to properly account for different soil properties and pore water pressure conditions.

Care should be taken in properly using soil properties data. When pore water pressures are taken into account explicitly, the . . .

- Soil unit weights should be total unit weights,
- Soil shear strengths should be in terms of effective stress strength parameters, and
- Pore water pressure information should be available.

When pore water pressures are considered implicitly, the . . .

- Soil unit weights should be total unit weights,
- Soil shear strengths should be in terms of total stress strength parameters, and
- Pore water pressure information is not used.

TABLE IV-1. DETAILS OF UNKNOWNS, ASSUMPTIONS, AND EQUATIONS FOR THE SLOPE STABILITY PROBLEM AS POSED TO THE LIMIT EQUILIBRIUM METHOD

No	Method	Normal Forces		Shear Forces	Interslice Forces			F.S.	Total Number of Unknowns	Total Number of Equations				Total Number of Equations	Do the Number of Equations Equal the Number of Unknowns
		MAGNITUDE	LOCATION		INTERSLICE SHEAR	INTERSLICE THRUST	LOCATION OF INTERSLICE THRUST			EQUILIBRIUM			FAILURE EQN		
		①	②	③	④	⑤	⑥	⑦	Σ ① to ⑦ ⓧ	ΣF_x ⑨	ΣF_y ⑩	ΣM ⑪	⑫	Σ ⑨ to ⑫ ⓟ	⑬
	For the slope stability problem as posed to the limit equilibrium method	n	n	n	n−1	n−1	n−1	1	6n−2	n	n	n	n	4n	No
1	Bishop's	n	A	n	A	n−1	n−1	1	4n−1	n	n	n	n	4n	No
2	Janbu's	n	A	n	n−1	n−1	A	1	4n−1	n	n	n	n	4n	No
3	Kenney's	n	A	n	n−1	n−1	A	1	4n−1	n	n	n	n	4n	No
4	Sarma's	n	1 (n)	n	A	n−1	n−1 (A)	1	4n	n	n	n	n	4n	Yes
					RESULTANT	LOCATION OF THE RESULTANT	INCLINATION OF THE RESULTANT								
5	Morgenstern and Price's	n	A	n	n−1	n−1	1	1	4n	n	n	n	n	4n	Yes
6	Spencer's	n	A	n	n−1	n−1	1	1	4n	n	n	n	n	4n	Yes

A = *Assumed*; n = *Number of slices = Number of unknowns.*

IV. STATIC STABILITY: ANALYTICAL METHODS

Trial Shear Surfaces

The shear surface selection is in terms of its shape and location as discussed below.

Shear Surface Shape

Three commonly used shear surface shapes are of:

- Circular geometry
- Noncircular or wedge geometry
- Log-spiral geometry

Circular slip failures have been observed in homogeneous soil deposits. Noncircular or wedge failures have been observed in nonhomogeneous soil deposits. In analytical work, log-spiral shear surfaces in homogeneous soil deposits are considered to give lower factors of safety than circular shear surfaces. In an embankment dam analysis, therefore, all shear surface shapes should be tried to locate the paths along which shear sliding failure may occur.

The selection of potential slide surface geometry in slope stability analysis by the limit equilibrium method deserves careful consideration. The paper entitled **Suggestions for Slope Stability Calculations**, included in Appendix C, provides some suggestions in this regard.

Shear Surface Locations

Slope failures have been observed on the:

- Downstream slopes of dams
- Upstream slopes of dams during reservoir drawdown

The extent of slope instability varies from localized failures at the toe of a dam to large failures involving the top of the dam and foundation materials. Therefore, it is essential to analyze shear surfaces which are local to the toe of a dam, intermediate size shear surfaces which involve one-half to three-quarters of one slope of the dam, and large shear surfaces which encompass one slope, the dam crest, and the opposite slope.

Figure IV-1 on the following page illustrates the shape and location of two actual shear surfaces along which static slope failures have occurred in embankment dams. These shear surfaces were reconstructed from observational and instrumentation data.

Shear Surface Locations (Continued)

FIGURE IV-1(a). SCHEMATIC OF PARK DAM SLOPE FAILURE

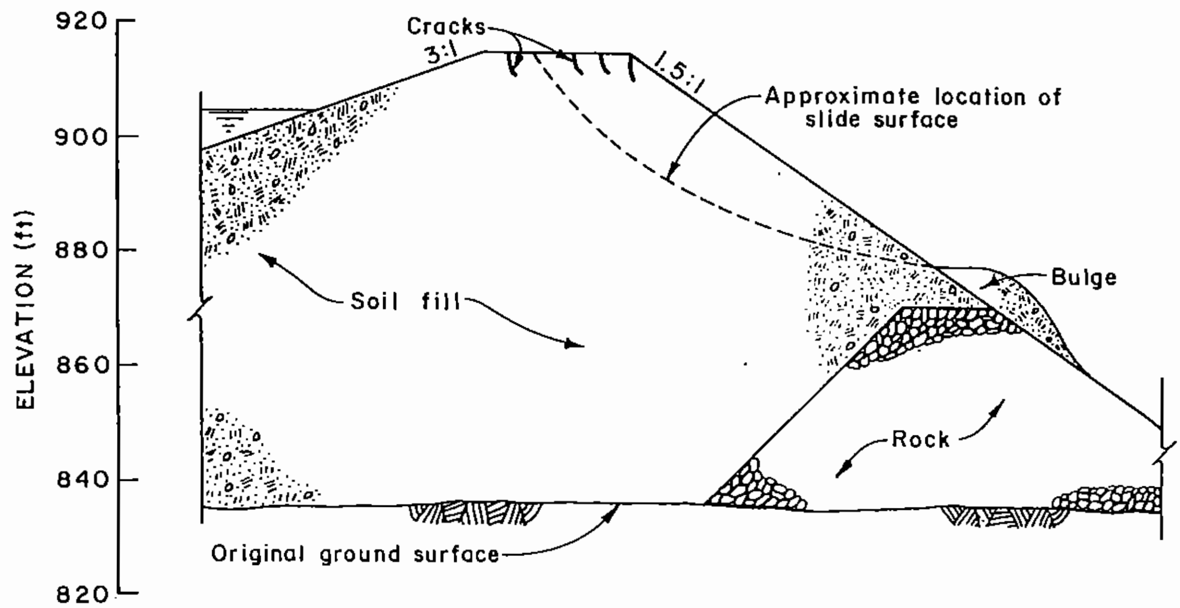
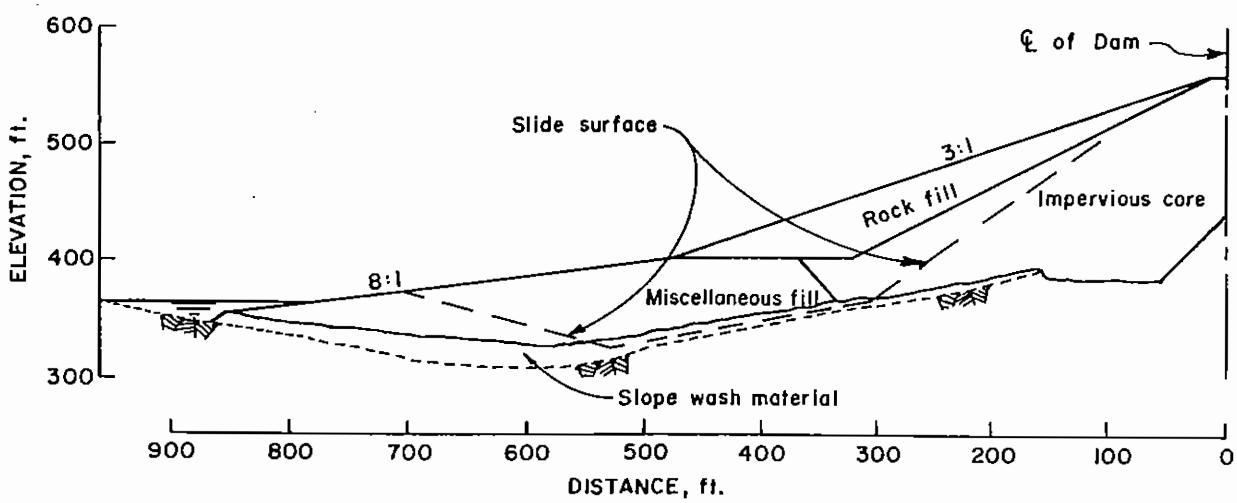


FIGURE IV-1(b). SCHEMATIC OF SAN LUIS DAM SLOPE FAILURE



IV. STATIC STABILITY: ANALYTICAL METHODS

Slope Stability Analysis Strategy

Computer programs are available for various analysis procedures including the simplified methods wherein the lateral forces on the sides of slices are omitted (A. W. Bishop, 1955). Simplified methods are advantageous when many shear surfaces are to be analyzed to locate the most critical one. However, after the critical surface has been determined, it is best to make a stability analysis using reasonable directions for forces on the sides of the slices and plotting the force polygon for each slice. Such plotting is essential for the engineer to review the reasonableness of the solution. This graphical check can be a substitute for a numerical check on the static equilibrium of forces on each slice.

For cohesionless materials, the critical surface of sliding is a surface at shallow depth parallel to the face of the dam. For cohesive materials, that is, $\phi = 0$, the critical surface of sliding is a circular surface at large depth. When layers of weak material occur in the dam or foundation, the critical surface of sliding is a wedge with a large portion of the shear surface located in a weak layer at shallow depth.

Principal Steps In Limit Equilibrium Analysis

The following are the principal steps in limit equilibrium analysis:

1. Select the embankment dam section for static slope stability analysis. Generally, it will be the maximum section.

2. Draw the dam cross section and include the embankment and foundation material boundaries.

3. Mark the pore pressure data and/or the phreatic line estimate. Include the shear strength data for each material for the appropriate loading condition.

4. Draw shear surfaces along which static slope stability analysis need to be performed. Follow the suggestions given in the section on Trial-Shear Surfaces on page IV-16.

5. Prepare the input data following the computer program user instructions.

6. Submit the input data of Step 5 for computer analysis.

IV. STATIC STABILITY: ANALYTICAL METHODS

Analysis Results

The results of slope stability analysis by the limit equilibrium method are the factor of safety, normal and shear stresses along the shear surface, and the normal and shear stresses along the interslice boundaries. Before accepting the computed factor of safety, the results should be scrutinized for reasonableness, that is, normal stresses do not indicate tension across the shear surface, the directions of shear stresses are consistent with the direction of possible sliding movement, and the resultants of interstice forces lie within the slide mass. However, the magnitude of these stresses are not the same as those obtained in the finite element analysis because the deformable nature of the soils is neglected in the limit equilibrium method.

FINITE ELEMENT METHOD

For static slope stability analysis of embankment dams, the linear elastic material mode and the elastic-plastic material models have been used (E. L. Corp, R. L. Schuster, and M. M. McDonald, 1975). If dynamic stability analysis is planned, consider the requirements for dynamic analysis by the finite element method discussed in the section on Seismic Analysis Methods in Unit V.

A brief description of the finite element method was provided in Unit III, Settlement And Deformation. Its use for static stability analysis is described below.

As a general approach, at least one cross-section of the embankment, for example, the maximum section, is analyzed. Irregular abutments, variable foundation materials, or diverse material properties may necessitate additional sections or a three-dimensional analysis. The numerical model is extended a sufficient distance past the upstream and downstream toes of the embankment dam so that the computed stresses in the dam and foundation are not influenced by the proximity of the mesh boundary (R. W. Clough and R. J. Woodward, III, 1967; International Commission on Large Dams, 1986).

Principal Steps In Finite Element Analysis

The following are the principal steps taken when conducting a finite element analysis.

1. Select the location of the embankment dam section for static stability analysis. Generally, it will be at the maximum section.

2. Draw the dam cross-section and include the embankment and foundation material boundaries. Include the stress-strain and strength properties of each material for the appropriate loading conditions.

Principal Steps In Finite Element Analysis (Continued)

3. Divide the cross section into finite elements. Each element must completely lie in one material. The proportioning of element sizes should follow the finite element discretization requirements for static analysis (R. W. Clough and R. J. Woodward, III, 1967). If dynamic stability analysis is planned, consider the requirements for dynamic analysis discussed in the section on Seismic Analysis Methods in Unit V.

4. Number the elements and nodes following the conventions of a particular computer program to be used.

5. Prepare input data following the computer program user instructions.

6. Submit the input data from Step 5 for computer analysis.

Analysis Results

The results of a finite element analysis of an embankment dam and its foundation include stresses and deformations for each element for the loading conditions under study. The computed shear stresses are compared to the corresponding shear strengths to determine the factor for safety on an element-by-element basis. This information is used to assess an average factor of safety along a selected shear surface by taking an average of the calculated factor of safety values for the elements along the shear surface. Similarly, potentially critical shear zones are identified by connecting the elements with low factor of safety values.

IV. STATIC STABILITY: REMEDIAL ACTION

INTRODUCTION

The engineering evaluation of an existing embankment dam and its foundation provides the engineer with quantitative information about dam safety deficiencies. The objectives of evaluating this quantitative information should be to:

- Identify areas of dam safety deficiencies and magnitudes of these deficiencies.

- Identify the causes of dam safety deficiencies.

Once the answers to these two objectives are known and understood, there is often more than one way to improve the safety of the dam. The means to rectify a particular deficiency may be conventional or quite innovative; however, the relative merits of each alternative should be evaluated and cost estimates prepared. The final choice of a particular scheme depends upon:

- The relative merits of possible solution schemes
- Economic considerations
- Organizational preferences
- Past experiences

REMEDIAL MEASURES

Features which have been used to improve the static stability of embankment dams include:

- Repairing oversteepened embankment slopes
- Buttressing unstable embankment slopes with additional fill
- Sealing cracks in embankments to prevent rainfall infiltration
- Sealing the upstream slope with a membrane or other seepage barrier
- Removing and replacing weak embankment material
- Adding drainage zones
- Rehabilitating existing toe drains
- Adding toe drains

SUMMARY: STATIC STABILITY

Unit IV discussed static stability of embankment dams, and provided background information on significant events that can cause static instability and indicators of static instability.

Review And Evaluation Of Project Data

The types of project data to be reviewed when evaluating static stability include:

- Site geology and/or material property data
- Design, as-built construction, and performance data
- Loading conditions

Analytical Methods

The two general approaches to assessing embankment dam static stability were discussed:

- Effective stress analysis, and
- Total stress analysis

The commonly used analysis methods were briefly presented:

- Limit equilibrium method
- Finite element method

Remedial Action

Measures to alleviate potential static instability were stated.

UNIT V

DYNAMIC STABILITY

V. DYNAMIC STABILITY: OVERVIEW

INTRODUCTION

This unit discusses the dynamic stability of embankment dams. Background information is presented on:

- When dynamic analysis is warranted
- Modes of failure related to dynamic instability
- Objectives of dynamic stability analysis

In addition to background information on dynamic instability and its effects, this unit also describes:

- What types of project data should be reviewed and analyzed at the outset of evaluating the dynamic stability of an embankment dam.

- What dynamic properties of soils are of interest in a dynamic stability analysis and how these values are obtained.

- What approaches and methods may be used to analyze the dynamic stability of an embankment dam.

- What remedial actions can be taken to alleviate potential dynamic instability.

V. DYNAMIC STABILITY: BACKGROUND INFORMATION

INTRODUCTION

Earthquakes may affect embankment dams in various ways. Seismic forces may be transmitted directly from the foundation to the dam. Reservoir waves overtopping the dam may be generated by earthquake-induced landslides or oscillation of the reservoir or sudden movement of the dam foundation.

For well-built embankments on stable foundations, that is, of and on densely compacted soils, required dynamic analysis may be minimal if estimated peak ground accelerations would not exceed 0.2 g. Where ground motions are more severe, engineers often use dynamic displacement analysis for dams constructed of and on soils which would not lose strength under seismic impact. When embankment or foundation soils might be weakened substantially by strong shaking, dynamic analyses are made to determine liquefaction potential.

Since the failure of Sheffield Dam in 1925 and the historic incident at Lower San Fernando Dam in 1971, the dynamic stability of hydraulic fill dams especially has become suspect. These concerns are for potential liquefaction of loose saturated sands, gravels, or silts having a contractive structure when subjected to shear deformations with high pore pressures developing, resulting in a loss of resistance to deformation (R. B. Jansen, 1980).

It should be understood that performing a dynamic analysis of an embankment dam makes sense only if the static stability of the dam is adequate. For dams with marginal static stability, a dynamic loading of any significance is bound to cause stability problems. Therefore, it is essential to ensure a sound static response of an existing embankment dam, by corrective actions if needed, before continuing with the dynamic analysis. The dynamic loads are in addition to the static loads. Thus, the static analysis results need to be combined with the dynamic analysis results in order to estimate the response of an embankment dam to an earthquake.

V. DYNAMIC STABILITY: BACKGROUND INFORMATION

WHEN DYNAMIC ANALYSIS IS WARRANTED

The need for dynamic analysis of an existing embankment dam and its foundation is indicated if:

- The dam was designed and built prior to 1970 when the significance of earthquake effects on embankment dams and their foundation materials was not well recognized or well understood, and only pseudostatic methods of stability analysis were used for dams in seismic areas.
- Recent seismic activity and/or seismotectonic studies indicate potential for earthquakes which may be more damaging than the ones estimated at the time the dam was designed.
- Review of geology and construction records or sampling and testing indicates the presence of potentially liquefiable materials in the foundation.
- The dam was constructed by hydraulic fill methods.
- A high potential exists for loss of life and property in the event of failure.
- A new hazard assessment of the area is needed due to demographic changes.

One or more of these factors may be of enough significance to warrant undertaking a dynamic analysis of an existing embankment dam and its foundation. Sometimes dynamic analysis of an existing embankment dam may be required for raising the dam, or for changed reservoir operations. These are not safety of dams issues in the conventional sense and are, therefore, not covered in this document.

V. DYNAMIC STABILITY: BACKGROUND INFORMATION

MODES OF FAILURE

Figure II-2 on page II-12 includes a schematic description of the most serious modes of failure of an embankment dam due to earthquake loading; namely, liquefaction in the dam body, and the liquefaction in the foundation soil. Other modes of failure of an embankment dam due to earthquake loading include (R. B. Jansen, 1988):

- Slope failures induced by ground motions.
- Sliding of the dam on weak foundation materials.
- Disruption of the dam by major fault movement in the foundation.
- Loss of freeboard due to differential tectonic ground movements.
- Loss of freeboard due to slope failures or soil compaction.
- Piping failure through cracks induced by the ground motions.
- Overtopping of the dam due to seiches in the reservoir.
- Overtopping of the dam due to large slides or rockfalls into the reservoir.
- Overtopping of the dam due to failure of the spillway or outlet works.

Localized dam incidents during an earthquake can quickly become large incidents, threaten the integrity of an embankment dam, and lead to uncontrolled release of reservoir water. Therefore, dynamic stability evaluation of an embankment dam should include a study for these possible modes of failure.

The upper part of an embankment is especially vulnerable to seismic forces. It is susceptible to cracking and to separation at the contact with the abutment. Since seepage paths are short near the top of the dam, and because the internal embankment pressures are generally too low to close cracks, the potential for dangerous leaks is considerable.

V. DYNAMIC STABILITY: BACKGROUND INFORMATION

OBJECTIVES OF DYNAMIC STABILITY ANALYSIS

The principal objectives of dynamic stability analysis of existing embankment dams and their foundations are to assess:

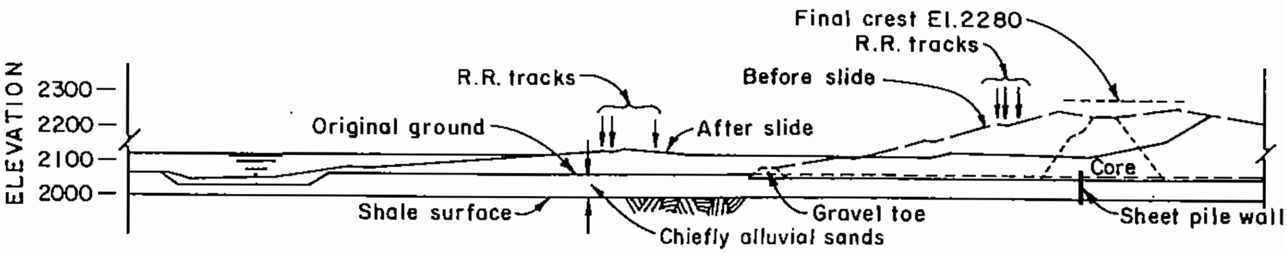
- Liquefaction potential of susceptible materials. The dam becomes unstable as a result of loss in soil strength in the dam or foundation. Typical examples are liquefaction slides in the Lower San Fernando Dam and Fort Peck Dam (see Figure V-1(a)).

- Extent of permanent deformations. The dam remains stable during and after the earthquake; however, deformations accumulate. The accumulated deformation needs to be estimated and evaluated with respect to its effects on the likelihood of an uncontrolled release of water from the reservoir (see Figure V-1(b) on the following page).

- Potential for cracking. An estimate of effective stresses in the embankment and its foundation during and following an earthquake is of interest in evaluating the potential for cracking.

The possible effects of fault movement on embankment dam stability also need to be considered in analysis.

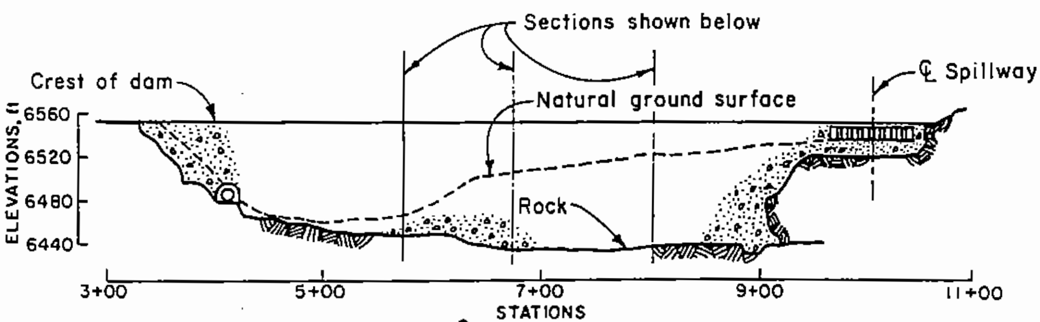
FIGURE V-1(a). SCHEMATIC OF SLIDE AT FORT PECK DAM



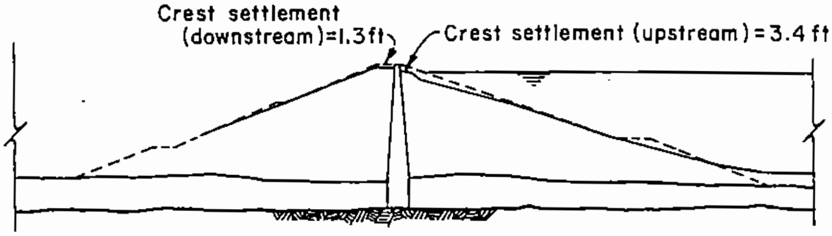
V. DYNAMIC STABILITY: BACKGROUND INFORMATION

OBJECTIVES OF DYNAMIC STABILITY ANALYSIS (Continued)

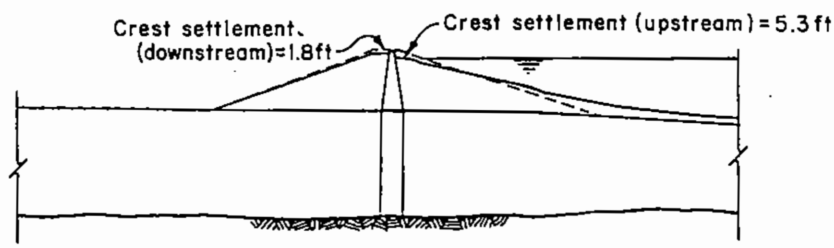
FIGURE V-1(b). SCHEMATICS OF CROSS SECTIONS THROUGH HEBGEN DAM BEFORE AND AFTER EARTHQUAKE



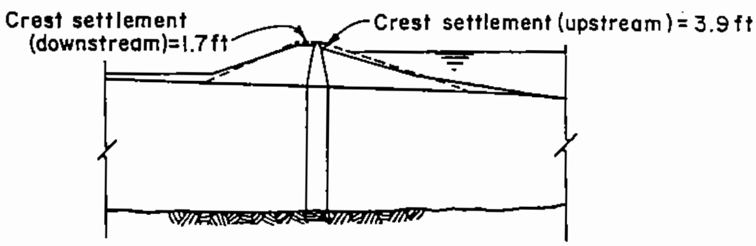
(i) Profile of Hebgen Dam



(ii) Station 5+75



(iii) Station 6+75



(iv) Station 8+00

--- Cross section before earthquake
— Cross section after earthquake
0 50ft

V. DYNAMIC STABILITY: REVIEW AND EVALUATION OF PROJECT DATA

INTRODUCTION

Prior to doing a dynamic analysis of an embankment dam, it is essential to review the available information on:

- Site geology
- Dam design and construction
- Reservoir operations and dam performance
- Static analysis results
- Instrumentation data
- Landslide activity along the reservoir

CONSIDERATIONS FOR PERFORMANCE UNDER DYNAMIC LOADING

During the project data review, the presence or absence of favorable and unfavorable factors affecting the embankment dam's ability to perform satisfactorily during an earthquake should be identified.

Favorable Factors

- Large freeboard.
- Wide transition zones.
- Adequate compaction of materials in the foundation and embankment.
- A high level of quality control in construction.
- Continuous surveillance and monitoring of the dam.
- Foundation of competent bedrock.
- No static slope instability or excessive settlement problems during dam construction and operations.
- No excessive seepage or high pore pressures in the dam body or foundations.
- High static factor of safety for steady-state seepage condition.
- No large landslides around the reservoir rim.

Unfavorable Factors

- Proximity to active faults.
- Unconsolidated sediments in the foundation.
- Low-density materials in the embankment and/or foundation.
- Low or marginal static stability.
- Poor quality control during construction.
- Zones of high pore pressure in the dam body and/or foundation.
- Uniform fine-grained, cohesionless materials in foundations.
- Unstable reservoir rim.

CONSIDERATIONS FOR PERFORMANCE UNDER DYNAMIC LOADING (Continued)

These two lists of favorable and unfavorable factors may not be complete; the order of entries is not important and the presence or absence of all factors at a particular site is not implied. Not all of the factors included in these two lists can be subjected to quantitative assessment in terms of numerical values. Therefore, their merits must be evaluated in qualitative terms, case-by-case. The following comments on these factors may be of general interest in dynamic stability problem(s) identification:

- The presence of bedrock, instead of alluvial deposits, along the dam-foundation contact essentially eliminates the concerns associated with foundation liquefaction. Thus, the dynamic analysis efforts can be concentrated on the dam body.

- A high level of quality control during construction is a good indicator that the embankment dam was constructed as designed. If the design appears adequate, then there can be greater confidence in satisfactory performance of the embankment dam during service.

- The review of static analysis results should help identify zones of relative weakness and higher vulnerability during an earthquake. These will be the zones with low static factors of safety. High pore pressure zones and excessive seepage under steady-state conditions get worse under earthquake loading and may cause embankment cracks and lead to piping failure. High pore pressures cause a decrease in effective stresses in soils and can lead to liquefaction failures. Identification of these zones in the data review warrant closer examination in dynamic analysis.

- A wide transition zone on the upstream side of the core is considered to provide material which may fill a crack in the core should one develop. A downstream filter or transition zone should preclude migration of soil particles from the core and, thus, inhibit the tendency for continuous piping of core material. The two transition zones, one upstream and one downstream of the core, work together to promote satisfactory performance of the embankment dam during an earthquake.

- Stable reservoir rim slopes as indicated by a lack of landslide activity is a good sign. However, if there is an ancient landslide in the reservoir area, it should be evaluated for satisfactory behavior for the earthquake loading. The presence of a large freeboard provides the additional reservoir capacity to contain the increased volume; more importantly it provides a barrier to prevent the water wave, generated by a sudden landslide plunge, from overtopping the embankment dam.

CONSIDERATIONS FOR PERFORMANCE UNDER DYNAMIC LOADING (Continued)

- Continuous surveillance and monitoring in itself does not change the performance of an existing dam. However, it does provide information about developing problems. Coupled with an early warning system and emergency action plans, this information can be used effectively in safely evacuating people if failure of the embankment dam seems imminent.

An ideal embankment with an ability to adjust safely to differential movements would be the one which has . . .

- An impervious zone composed of a well-graded mixture of clay, silt, sand, and gravel,
- Ample transitions and drains,
- Thoroughly compacted gravel or quarried rock shells, and
- Liberal freeboard.

One of the designs least resistant to seismic loading would be a dam with a thin, sloping core of silt or other easily erodible soil, thin transition zones, and dumped rockfill shells. Dumped rockfill may have questionable merit in a high dam because it is susceptible to considerable settlement under severe shaking.

V. DYNAMIC STABILITY: DYNAMIC PROPERTIES OF SOILS

INTRODUCTION

The dynamic properties of embankment and foundation soils of interest in a dynamic stability analysis are:

- Shear stress - shear strain - damping characteristics of soils in the dam and foundation under cyclic loading.
- Shear strength of soils under cyclic loading.
- Susceptibility of soils to liquefaction.

The following is a brief discussion of the commonly used field and laboratory tests to obtain these properties (Dynamic Analysis of Embankment Dams, Bureau of Reclamation, 1976).

GEOPHYSICAL IN SITU INVESTIGATIONS

Geophysical explorations are used to obtain the in situ shear wave velocity from which the shear modulus, corresponding to small shear strain ($< 10^{-4}$ percent) is calculated. There are uncertainties in the geophysical techniques for determining damping characteristics of soils in situ. Therefore, laboratory tests are preferred for damping value determinations.

There are a number of different geophysical exploration methods for determining in situ shear wave velocity. These methods can be grouped in two categories: (a) the steady-state vibratory source methods, and (b) the impulsive transient source methods. All these methods involve an energy source to generate seismic waves and an array of geophones to measure the arrival of seismic waves. With the distance between the energy source and the receiving station, and the time of travel being known, the seismic wave velocity is calculated. Only low-yield explosives or nonexplosive sources, such as impact or impulse devices, are used to generate waves. They induce very small amounts of energy into the soil. A brief description of these methods is given on the next page.

GEOPHYSICAL IN SITU INVESTIGATIONS (Continued)

- **Steady-State Vibratory Source Methods** which include:
  - The Rayleigh wave method in which the energy source is an electromagnetic vibrator or a mechanical vibrator with counter-rotating eccentric masses.
  - The shear wave method in which the energy source for generating shear waves is vibriosis, which vibrates in the horizontal plane at constant frequency for a specific number of cycles.
- **Impulsive Transient Source Methods** which include:
  - The Rayleigh wave dispersion method in which low energy explosives are used to generate Rayleigh waves. For all practical purposes, the Rayleigh waves are considered to be equivalent to shear waves. However, Poisson's ratio of the material has a slight influence in relating Rayleigh wave and shear wave velocities.
  - The shear wave - refraction method in which an impact energy source is used to generate shear waves. No borehole is required.
  - The shear wave - downhole method in which an impact device and a wooden plank or steel plate placed near a borehole is used to generate shear waves. One borehole is required.
  - The shear wave - crosshole method in which a small explosive in a fluid-filled hole or a mechanical impulse device located in a dry hole is used to generate shear waves. More than one borehole is required.
  - The shear wave - uphole method which is usually performed along with the crosshole method by placing the geophones on the ground surface near the shot hole. One borehole is required.

The selection of any particular method for determination of shear wave velocity in an embankment dam and its foundation is a matter of the past experience of the personnel, availability of equipment, and organization preference. In general, the most reliable results are obtained from borehole methods, primarily the downhole and crosshole methods; between these two, the crosshole method is generally preferred. See <u>Dynamic Analysis of Embankment Dams</u>, U.S. Bureau of Reclamation, for additional details on geophysical in situ investigations for embankment dams and their foundations.

GEOPHYSICAL IN SITU INVESTIGATIONS (Continued)

The geophysical exploration methods used are able to induce only small strains in the soil (less than 10^{-4} percent). The shear wave velocity and the calculated shear modulus are for small shear strain. Strains produced under earthquake loading are usually greater than 10^{-3} percent; therefore, laboratory tests are required to determine the shear modulus variation for these larger strains. Also, laboratory tests are required for determination of reliable damping values and their variation with strains.

LABORATORY TESTING

Laboratory tests are performed to evaluate the soil behavior under dynamic loadings, namely: the dynamic properties and cyclic strength tests.

The dynamic property tests provide parameters for use in determining the shear stresses induced in an embankment and its foundation. The cyclic strength test results are used to evaluate the soils' ability to withstand these shear stresses safely. The evaluation is on the basis of loss of strength (liquefaction) or deformation.

Prior to laboratory testing, a field exploration and sampling program is conducted. The scope and techniques employed to determine the in situ conditions and to obtain the samples needed in the laboratory depend on the type of structure involved, for example, rolled earth or hydraulic fill, the type and condition of soil encountered, and existing data. In addition to these factors, laboratory and analysis requirements are considered in planning and performing the field investigations.

Dynamic Property Tests

The two dynamic properties of interest in performing the dynamic response analysis of an embankment dam and its foundation are:

- The shear modulus versus shear strain.
- The damping ratio versus shear strain.

Dynamic Property Tests (Continued)

The following tests are commonly performed in the laboratory to determine dynamic response properties in soil. Figure V-2 on the following page shows the approximate strain range of each of these tests.

- Resonant column test in which one end of a cylindrical specimen is forced to vibrate in either the torsional or longitudinal mode.

- Cyclic simple shear test in which shear strains are applied to the specimen and the shear modulus is calculated from the ratio of the shear stress to the shear strain.

- Cyclic triaxial test in which a cylindrical specimen is subjected to a series of repetitive axial compression and extension loads while the vertical deformation is monitored.

The cyclic simple shear test most nearly duplicates the loading conditions thought to occur during an earthquake. However, the cyclic triaxial test is the most widely used test to evaluate the dynamic shear modulus and damping ratio for earthquake analysis. Other test methods listed in Figure V-2 are not commonly used.

Synthesis Of Dynamic Property Test Results

To obtain the variation in shear modulus and damping ratio over the range of shear strain required for the dynamic analysis, the resonant column, and the cyclic simple shear or cyclic triaxial test results are combined.

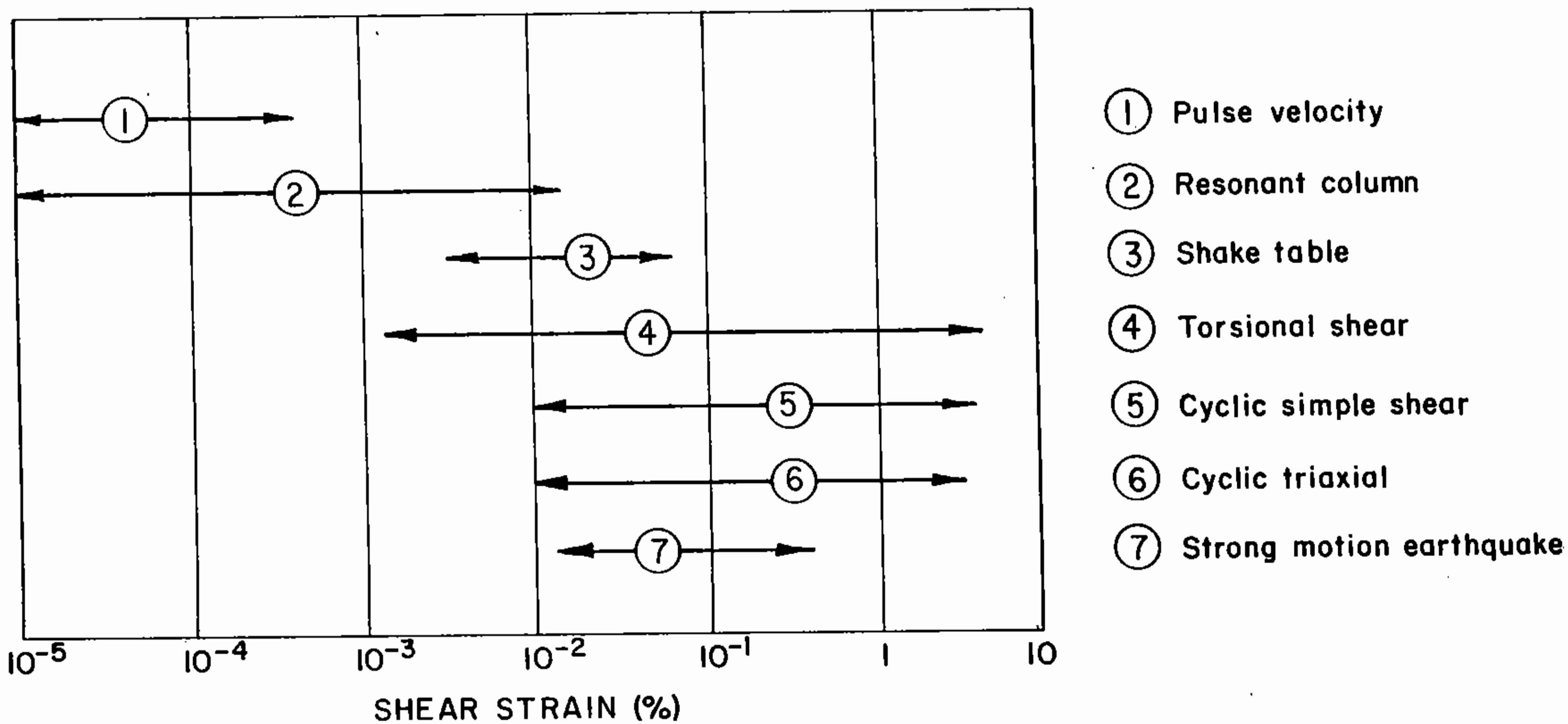
After reducing the data to a function of the mean principle stress, the results of individual tests are combined on a single plot of shear modulus and damping ratio versus single amplitude shear strain.

The site-specific test results are generally compared to those available in the literature for shear modulus and damping ratio versus strain; any differences in results must be resolved to the satisfaction of engineers involved in the seismic evaluation work prior to their use in dynamic analysis.

Cyclic Strength Tests

The cyclic strength tests are the second group of dynamic tests required for the dynamic analysis of an embankment dam and its foundation. The cyclic strength tests are carried out to large shear strain levels to enable evaluation of the shear strength of soil at large strains. Also, the cyclic strength is significantly affected by the static stress conditions. Therefore, the stresses applied in the laboratory tests must simulate those existing in or beneath the embankment dam prior to the occurrence of earthquake.

FIGURE V-2. APPROXIMATE STRAIN RANGE OF TESTS USED TO OBTAIN DYNAMIC SOIL PROPERTIES



V. DYNAMIC STABILITY: DYNAMIC PROPERTIES OF SOILS

Cyclic Strength Tests (Continued)

The following test methods are commonly used:

- Shake table,
- Cyclic simple shear,
- Cyclic torsional shear, and
- Cyclic triaxial.

Because of the wide range in stress conditions existing in an embankment and the need to test undisturbed as well as remolded specimens, the cyclic triaxial test is commonly used in the dynamic analysis of embankment dams.

In the cyclic triaxial test, the pre-earthquake static stresses are simulated by consolidating a cylindrical specimen under isotropic or anisotropic stresses; a series of uniform, load-controlled axial compression and extension stresses are applied; and the load, axial deformation, and pore pressure are monitored.

Many of the procedures followed in conducting isotropic and anisotropic tests are similar. However, significant differences exist between the two types of tests, particularly in analyzing the results. See Dynamic Analysis of Embankment Dams by the U.S. Bureau of Reclamation for additional details on cyclic triaxial test and data analysis.

The results of the isotropic and anisotropic tests are summarized to show the effect of combined static and cyclic stresses on the cyclic strength. These presentations show:

- The cyclic stresses required to cause a selected percent strain in a specific number of cycles.

- The effect of confining pressure on cyclic stresses as a function of number of cycles.

- The effects of static normal stress and the cyclic shear stress on cyclic stresses as a function of number of cycles.

V. DYNAMIC STABILITY: DYNAMIC PROPERTIES OF SOILS

Soil Liquefaction Tests

The method commonly used for liquefaction analysis of a soil deposit subjected to earthquake loading and the resulting generation of shear stresses on horizontal planes requires the following information (H. B. Seed, P. P. Martin, and J. Lysmer, 1975):

- The liquefaction characteristics of the materials under undrained conditions.
- The permeability of the soils.
- The compressibility of the soils.

The following tests are performed on the representative samples of the soils under investigation to determine these characteristics:

- **Liquefaction Characteristics.** These are normally determined by means of undrained cyclic simple shear or triaxial compression tests (the latter with appropriate correction factors applied to convert the results to equivalent simple shear conditions).

- **Permeability Characteristics.** Permeability is an extremely variable soil characteristic and it is not easy to relate it to simple index properties. Some approximate relationships between grain-size characteristics and permeability coefficients are available in the literature (H. B. Seed, P. P. Martin, and J. Lysmer, 1975).

- **Compressibility Of Soil.** Very few studies have been made on the settlement and compressibility of saturated sands due to dissipation of pore pressures produced by cyclic loading. For pore pressure dissipation effects, the coefficient of volume compressibility at low pore pressure is needed. This characteristic is generally assessed from a knowledge of grain size and relative density using the available test data (H. B. Seed, P. P. Martin, and J. Lysmer, 1975).

See the reference The Generation and Dissipation of Pore Water Pressures During Soil Liquefaction by H. B. Seed, P. P. Martin, and J. Lysmer, 1975 in Appendix B for additional details on soil liquefaction tests, available test data, and approximate relationships.

V. DYNAMIC STABILITY: ANALYTICAL METHODS

INTRODUCTION

When analyzing the dynamic stability of an embankment dam, the following must be addressed:

- Loading conditions
- Analytical approach
  - Pseudostatic method versus dynamic response method
  - Effective stress versus total stress analysis
  - Liquefaction evaluation

LOADING CONDITIONS

Dynamic analysis of an embankment dam is generally performed for the steady-state reservoir water level. A dynamic analysis is seldom necessary in conjunction with sudden drawdown of the reservoir. However, if earthquake loading is possible during reservoir drawdown associated with a pumped storage project where frequency of drawdown occurs on a daily cycle, earthquake effects during sudden drawdown should be investigated.

Site-specific seismic evaluations identify earthquake source areas, the maximum credible earthquake, and the distance from the site of each relevant source area. A time-acceleration record is suggested by the responsible seismologist on the dam safety evaluation team for use in dynamic analysis of the embankment dam under study.

It is generally considered that the dynamic response of embankment dams does not require consideration of the vertical component of ground motion and the hydrodynamic effects of the reservoir. However, these considerations are open for discussion for embankment dams with steep slopes, such as rockfill dams (Selecting Seismic Parameters for Large Dams, International Commission on Large Dams, 1989). Also, only one horizontal component of ground motion, acting in the upstream-downstream direction, is considered for seismic evaluation for embankment dams, unless a three-dimensional analysis is warranted. It is suggested that these issues be resolved and agreed upon between all parties involved in the seismic safety evaluation for a particular embankment dam.

PSEUDOSTATIC VERSUS DYNAMIC RESPONSE METHODS

The pseudostatic method of analyzing dynamic stability incorporates the seismic force as a static external force applied to a soil deposit, and a static slope stability analysis is carried out. The static seismic force is designated by a seismic coefficient. The seismic coefficient is assigned a numerical value based on experience.

The dynamic response methods incorporate dynamic properties of soils and use earthquake time-acceleration data. The pseudostatic method is used to calculate the acceleration at which sliding will begin to occur. This result is used in dynamic displacement calculations.

PSEUDOSTATIC VERSUS DYNAMIC RESPONSE METHODS (Continued)

The pseudostatic method of analysis is considered inadequate to reliably predict the safety of embankment dams subjected to earthquake loading because the method could not be used to properly explain the following events (Safety of Dams: Flood and Earthquake Criteria, National Academy Press, 1985; H. B. Seed, K. L. Lee, I. M. Idriss, and F. Makdisi, 1973):

- Slope failures that occurred at many places in Alaska in the 1964 Alaska earthquake, magnitude 8.3.
- A near failure of the Lower San Fernando Dam, and significant sliding in the Upper San Fernando Dam as a result of the San Fernando earthquake of 1971.
- Accelerograph records showing peak accelerations during earthquake shaking greater than 0.3 g.

At about the time of these events, finite element methods and high-speed computers had become available for making improved analyses for seismic response. The ability of dynamic response analyses to provide better insights into probable field performance was studied. The results of these studies were sufficiently convincing for the changeover of methodology. With the improvement in characterization of static and dynamic properties of soils, and advances in cyclic testing, the dynamic method of analysis has been continuously improved and refined.

The pseudostatic method and the dynamic response methods are discussed in the section entitled Seismic Analysis Methods.

EFFECTIVE STRESS VERSUS TOTAL STRESS ANALYSIS

The two different dynamic analysis procedures are:

- Effective stress dynamic analysis.
- Total stress dynamic analysis.

In effective stress dynamic analysis, the dynamic response of pore fluid and its interaction with the soil skeleton are considered essentially simultaneously. The effective stress dynamic analysis is generally performed with the finite element method. The results of this analysis yield stresses and deformations. These results are used to determine answers to various modes of embankment dam failure discussed previously.

EFFECTIVE STRESS VERSUS TOTAL STRESS ANALYSIS (Continued)

In total stress analysis, the dynamic response of the soil skeleton and the dynamic response of pore fluid are studied separately and their results combined. The soil and pore fluid response interaction can be approximated by performing several total stress dynamic analyses using modified dynamic soil properties which are consistent with the corresponding dynamic pore pressure analysis results. The total stress dynamic analysis is generally used for the various modes of embankment dam failure discussed previously. The analyses are generally one-dimensional or two-dimensional and use numerical discertization of the problem in layers or finite elements.

Figure V-3 on the following page shows a schematic description of the two procedures (A. K. Chugh and J. L. Von Thun, 1985). Both of these analysis methods are computation intensive and do not lend themselves to longhand calculations for practical applications in embankment dam engineering. Therefore, computer programs have been developed implementing these analysis methods to facilitate their use in practice.

The choice of effective stress or total stress dynamic analysis depends upon:

- Availability of a particular computer program.
- Experience and training of the user engineer.
- Organizational preference.

If used properly, the two methods should yield similar results and lead to essentially identical conclusions. However, the effective stress dynamic analysis method should be a preferred choice provided that site-specific test data and an effective stress dynamic analysis computer program are available.

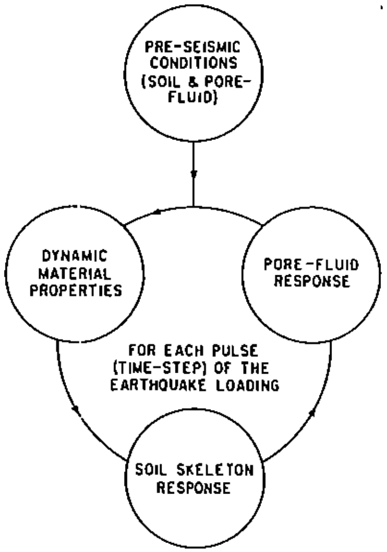
TYPES OF ENGINEERING ANALYSES

The following analyses are considered adequate to assess the dynamic safety of existing embankment dams and their foundations (Safety of Dams: Flood and Earthquake Criteria, National Academy Press, 1985):

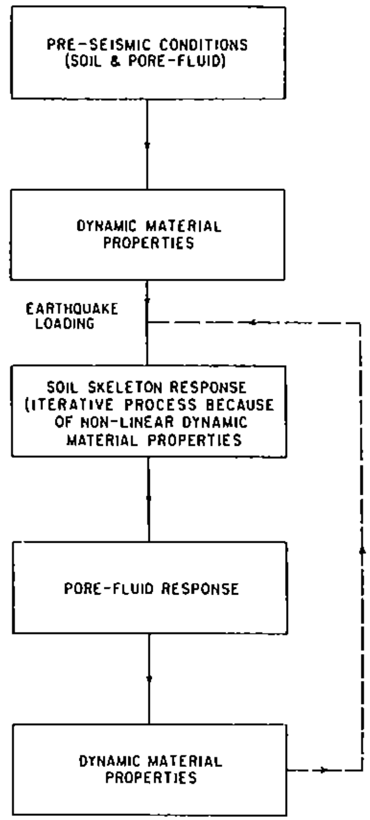
- For reasonably well-built dams on rock or stable soil foundations, the pseudostatic method of stability analysis may be used if estimated peak ground accelerations are less than 0.2 g. The value of the seismic coefficient should be selected as per recommendations given in the article by H. B. Seed and G. R. Martin entitled **The Seismic Coefficient in Earth Dam Design.**

- For reasonably well-built dams on stable soils that do not lose strength as a result of earthquake shaking and for estimated peak ground accelerations exceeding 0.2 g, deformations should be estimated using dynamic deformation analysis.

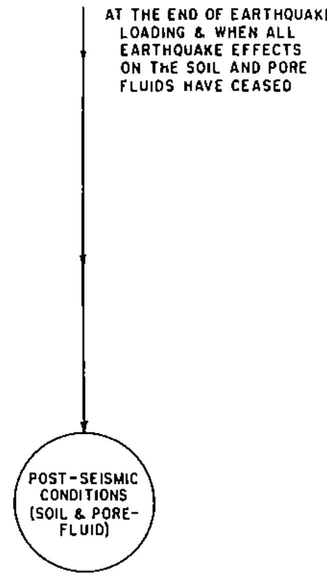
FIGURE V-3. SCHEMATIC DESCRIPTION OF EFFECTIVE AND TOTAL STRESS DYNAMIC ANALYSIS



(a) PHYSICAL PHENOMENA AND SEQUENTIAL ACCOUNTING OF THE EARTHQUAKE EFFECTS IN NUMERICAL ANALYSIS



(b) ITERATIVE ACCOUNTING OF THE EARTHQUAKE EFFECTS IN NUMERICAL ANALYSIS



(c) POST EARTHQUAKE PHYSICAL SITUATION

V. DYNAMIC STABILITY: ANALYTICAL METHODS

TYPES OF ENGINEERING ANALYSIS (Continued)

- For dams involving embankment and/or foundation soils that may lose a significant portion of their strength or develop high excess pore water pressure under the effects of earthquake shaking, dynamic analysis for liquefaction potential, or strength reduction potential, should be performed.

Whereas the first two analysis types offer a simple and reasonable means for evaluating the potential behavior of embankments built of clay soils, or those constructed of very dense cohesionless soils with little likelihood of a major decrease in undrained shear strength due to anticipated levels of shaking, the third analysis type can be used for all soils.

In addition to these three formalized analyses, it is a usual practice to compare the damsite under study with the historical occurrence of liquefaction at damsites during earthquakes. This comparison is in terms of epicentral distance and earthquake magnitude, and is independent of the dam height, valley shape and size, and type of embankment. Similarly, for liquefaction potential of sand and silt at the damsite under study, its steady-state strength and corrected blow count in standard penetration resistance tests are compared with the historical performance data (**Embankment Dam Design Standard for Seismic Design and Analysis**, Bureau of Reclamation, 1989). This comparison is independent of the earthquake magnitude and its distance from the damsite. These are the empirical approaches which provide a historical performance expectation for the dam under study. However, these are not intended as substitutes for the three formal analyses discussed above.

It should be emphasized that dynamic analysis of an embankment dam should be considered as a compliment to sound engineering judgment and previous experience with similar structures. It should be kept in mind that each completed structure and its immediate environment form a unique system that is not duplicated elsewhere.

Problem And Time Dimensions

The dynamic response analyses of embankment dams and their foundations may be one-, two-, or three-dimensional with time as an additional dimension. In general, however, one-dimensional and two-dimensional analyses, with time as an additional dimension, are commonly performed.

The pore pressure response in an earth deposit continues long after the cessation of earthquake activity, due to slow dissipation of developed pore pressures. The postearthquake period considered critical for an embankment dam stability lasts about 1 to 3 days after the earthquake. Therefore, it is essential to consider dynamic pore pressure response analysis for such an extended period of time.

V. DYNAMIC STABILITY: ANALYTICAL METHODS

SEISMIC ANALYSIS METHODS

There is no basic difference in methodology between seismic analysis of new and existing dams. However, the analysis efforts should be commensurate with the quality and quantity of data available. More complex analyses by themselves do not provide any better understanding of the possible responses; they may lead to a false sense of security. Therefore, major efforts should be made to collect and understand site-specific geotechnical data for adequate and useful analytical work. Still, the limitations of scientific understanding of earthquakes and the semiempirical nature of seismic analysis of embankment dams should be kept in mind in interpretation and use of analytical results. Analysis for earthquake loading should begin with simplified procedures and proceed to more rigorous methods of analysis as a particular situation may warrant (Earthquake Analysis Procedures for Dams, International Commission on Large Dams, 1986).

Pseudostatic Method

Prior to the advent of computers and modern earthquake engineering, this was the most commonly used method for evaluating dynamic stability of embankment dams in seismic areas. This is an extension of the static slope stability method in which a seismic force is applied as a static external load on the slide mass. Typically, the seismic force is applied in a horizontal direction, its magnitude is proportional to the mass, and the proportionality constant is the seismic coefficient. There are a number of approaches that have been suggested in evaluating the magnitude of seismic coefficient and its variation with dam height for use in dynamic stability analysis by this method. The pseudostatic method is also known as the seismic coefficient method (S. Okamoto, 1973; H. B. Seed and G. R. Martin, 1966).

When an embankment and/or its foundation are composed of loose sands, silts, or gravels, the pseudostatic method may not be applicable. The direct use of this procedure is limited to dynamic stability analysis of well-built embankment dams on stable foundation soils, with proper evaluation of the seismic coefficient considering the embankment dam height and material characteristics as well as different positions of the potential slide mass within the embankment section, and when peak ground acceleration is less than 0.2 g. The general use of this method is limited to yield acceleration determination which is a measure of external force required to bring a slide mass to the verge of failure. This result is used in calculating displacement of the slide mass for a given earthquake loading.

V. DYNAMIC STABILITY: ANALYTICAL METHODS

Dynamic Displacement Analysis Method

This method is commonly used for analyzing the magnitude of displacements that might occur where the soil could be considered to behave as a rigid plastic material and where sliding occurs on a well-defined slip surface. The method involves two steps (N. M. Newmark, 1965):

- The determination of yield acceleration, that is, the acceleration at which sliding will begin to occur.
- The evaluation of the displacements that would occur in time intervals when the earthquake caused accelerations exceed the yield acceleration.

The details of these two steps are described below.

Yield Acceleration Determination

The yield acceleration for a shear surface is calculated by adding an arbitrary force $F_e = \lambda W$ at an inclination γ to the horizontal at the centroid of each slice in addition to the forces for static slope stability analysis. $F_e = \lambda W$ corresponds to a constant acceleration λ times that of gravity multiplied by the mass of the slice. The value of λ corresponding to the inclination angle γ for which the calculated factor of safety equals 1 is the yield acceleration coefficient, λ_{yield}. Thus, this analysis gives a value for λ_{yield} corresponding to the inclination angle γ. It is convenient to consider the yield acceleration value for a shear surface to represent a measure of its strength against sliding at the time of an earthquake. The sample problem in the paper entitled **Slope Stability Analysis for Earthquakes**, included in Appendix C, illustrates the details of this calculation.

☞ An earthquake causes a buildup of pore water pressure in a soil deposit which decreases the mobilized shear strength along a shear surface. Therefore, it is likely that the yield acceleration for a slide mass changes during the earthquake. It is essential to recognize this effect.

V. DYNAMIC STABILITY: ANALYTICAL METHODS

Evaluation Of Displacements

The displacement of a slide mass during an earthquake is calculated by superimposing on the time-acceleration response of the slide mass, the yield acceleration value as determined above; integrating twice the equation of motion for the time intervals in which the design earthquake accelerations exceed the yield acceleration value; and summing up the incremental displacements over the duration of the earthquake. The sample problem in the paper entitled **Slope Stability Analysis for Earthquakes**, included in Appendix C, illustrates the use of this procedure.

☞ It should be recognized that except for single plane shear surface, all parts of a slide mass do not ride down on a single plane of constant inclination.

Ground Response Analysis Method

The ground response analysis method is associated with the vertical propagation of horizontally polarized plane shear waves through a linear viscoelastic medium consisting of horizontal layers. The layers are considered to extend to infinity in the horizontal direction and have a half space as the bottom layer (P. B. Schnabel, J. Lysmer, and H. B. Seed, 1972). Every layer in a soil deposit is considered homogeneous and isotropic and is characterized by thickness, mass density, shear modulus, and damping factor. Nonlinear soil behavior is treated as an equivalent linear material. The stress-strain properties of the soil are defined by strain dependent shear moduli and equivalent viscous damping factors. The assumptions associated with this method are valid for small shear strain. This is a total stress method. The procedure can be applied to an embankment dam's finite width by enforcing shear force compatibility across layer interface boundaries (A. K. Chugh, 1985; J. L. Vrymoed and E. R. Calzascia, 1978; S. L. Whiteside, J. W. France, and G. Castro, 1979).

Initial values of moduli and damping are selected corresponding to small strain values or to strain levels judged appropriate for the anticipated earthquake loading. An elastic analysis is carried out for the entire duration of the earthquake by solving the wave equation. The average strain, usually 65 percent of the maximum, is computed at each level; moduli and damping ratios, compatible with these average shear strains, are selected and calculations repeated. The iterative procedure is continued until no significant changes in moduli and damping are necessary. The response determined during the last iteration is considered to be a reasonable approximation to the nonlinear response. The sample problem in the paper entitled **Dynamic Response Analysis of Embankment Dams**, included in Appendix C, illustrates the use of this procedure. (A. K. Chugh, 1985).

Ground Response Analysis Method (Continued)

This analysis procedure is used to obtain:

- Time-acceleration response developed in the layers in the soil deposit. This information can be used for dynamic displacement calculations described in the previous section on the dynamic displacement analysis method.

- Shear stress-time response developed in the layers in the soil deposit. This information is converted into an equivalent number of uniform stress cycles for use in pore pressure response calculations described below.

Pore Pressure Response Analysis Method

The procedure for evaluating the generation and dissipation of pore water pressures in a soil deposit due to an earthquake loading involves the following steps (H. B. Seed, P. P. Martin, and J. Lysmer, 1975):

1. By means of a dynamic response analysis of the soil deposit, determine the shear stress histories caused by the earthquake at the various depths of interest in the soil deposit.

2. For each depth in the soil profile, determine the equivalent uniform cyclic shear stress, τ_{eq}, the equivalent number of uniform shear stress cycles, N_{eq}, and the effective period of each stress cycle, T_{eq}, representing the induced stress history.

3. Determine, from the laboratory cyclic load tests, the relationships between the applied uniform cyclic shear stresses and the number of stress cycles required to produce a condition of liquefaction under undrained conditions for different depths in the deposit.

4. From the data developed in the above step, determine the number of stress cycles of magnitude τ_{eq} required to cause liquefaction of the soil at that depth, N_1.

5. From the known ratios of N_{eq}/N_1 at various depths, determine the rate of pore pressure buildup for each elemental layer of the deposit, if it were undrained, using a representative curve from data on rate of pore pressure buildup in cyclic shear tests.

Pore Pressure Response Analysis Method (Continued)

6. From a knowledge of the coefficients of permeability and compressibility of soil layers, determine the corresponding values of the coefficients of consolidation for the different layers.

7. Solve the differential equation, defining the simultaneous generation and dissipation of pore water pressures in a soil deposit due to seismic loading, for the known values of soil characteristics, pore pressure generation expressions, and boundary conditions. The solution for pore water pressure dissipation may be pursued beyond the duration of earthquake activity.

The analysis gives the pore pressure response in a soil deposit subjected to earthquake loading. Soil liquefaction may or may not occur in a soil deposit during an earthquake.

Finite Element Method

The most accurate evaluation of stresses and deformations in an embankment is obtained when analyses are performed in a series of steps or increments to simulate construction, static and dynamic loading events, and using nonlinear soil behavior characteristics. Refer to Unit III for a general description of the finite element method. The use of this method for static analysis is described in Unit IV. It is convenient to use the same finite element mesh for static and dynamic analysis. The maximum height of the element is generally kept small as compared to the wave length of the shear wave to ensure that the major modes of vibration are included in the embankment response. As a general approach, at least two cross sections of the embankment, for example, the maximum section and an abutment section, are analyzed. Irregular abutments, variable foundation materials, or diverse material properties may necessitate additional sections or a three-dimensional analysis.

Material types within the structure, especially the hydraulic fill dams, should be carefully mapped; foundation materials and bedrock clearly delineated; and the dynamic properties of all materials established. The steady-state seepage condition with the reservoir at its normal water surface elevation is the usual case analyzed. The embankment drainage system should be carefully considered, and if questions arise as to the continuing ability of drains to function, they should be assumed inoperable for analysis. The earthquake input motion is applied simultaneously to all points along the base of the model.

V. DYNAMIC STABILITY: ANALYTICAL METHODS

Finite Element Method (Continued)

Once an analysis cross section has been selected, the following steps are undertaken in the dynamic method of analysis (R. B. Jansen, 1988):

1. Calculate the initial static stresses existing in the embankment and its foundation before the earthquake. Typically, plane strain conditions are used in this step.

2. Select time history of earthquake acceleration to which the dam and its foundation might be subjected. Appropriate inputs from geologists and seismologists are needed to complete this step adequately.

3. Assess the dynamic properties of the soils comprising the dam and its foundation, such as shear modulus, damping characteristics, bulk modulus or Poisson's ratio, that influence response to earthquake excitation. Because the material characteristics are nonlinear, it is also necessary to determine how the properties vary with strain.

4. Using an appropriate finite element procedure, calculate the response of the embankment-foundation system to the selected base excitation, including determination of the stresses induced in the embankment and its foundation. Plane strain conditions are also used in this step.

5. Conduct cyclic tests on representative soil samples from the dam and its foundation, to measure the combined effects of the initial static stresses and the superimposed dynamic stresses in order to evaluate the generation of pore water pressures and the development of strains in these soils. A sufficient number of these tests should be performed to permit similar evaluations to be made, by interpolation, for all elements comprising the embankment-foundation system. Alternatively, for the foundation layers and for existing embankments, the cyclic strength characteristics may be estimated, based on SPT blow count and existing correlations of SPT blow count and cyclic strength. These blow counts are also helpful in estimating the residual strength in these soils.

6. Evaluate the factor of safety against failure, either during or following the earthquake, based on pore pressures generated by the earthquake, the soil deformation characteristics, and the strength characteristics.

In addition to the above steps for a dynamic analysis, the following advice is quite appropriate: Be sure to incorporate the requisite amount of judgment in each of the steps, as well as in the final assessment of probable performance, being guided by a thorough knowledge of typical soil characteristics, the essential details of finite element analysis procedures, and a detailed knowledge of the past performance of embankments in other earthquakes (H. B. Seed, 1979).

V. DYNAMIC STABILITY: ANALYTICAL METHODS

Loss Of Stability

The potential for loss of stability can be analyzed using a static stability analysis incorporating minimum strength values corresponding to the degree to which pore water pressures are generated in the soils by the earthquake shaking. Where the pore pressure ratio in the soil builds up to a value close to 100 percent, the soil is considered to have developed a condition of liquefaction.

The determination of those zones where liquefaction or pore pressure buildup will occur must be made using a dynamic analysis to determine the stresses and strains induced in the embankment by the maximum anticipated earthquake motions and a knowledge of the pore pressure generation characteristics of the soils comprising the embankment and its foundation. In general, clayey soils do not appear to develop increases in pore water pressure due to earthquake shaking. However, loose, saturated cohesionless soils are highly vulnerable to pore pressure development due to earthquake shaking.

Once the degree of pore pressure buildup has been evaluated, and zones of potential liquefaction identified, soil may be assigned strength values for use in a static stability analysis as follows:

SOIL TYPE	SATURATED	UNSATURATED
Impervious (clayey)	S_{up}	S_{up}
Pervious (sands) with r_u = 100 percent	lower of S_{us} or S_r, and S_{ds}	$S_{d\text{-}u}$
Pervious (sands) with r_u < 100 percent	$S_{d\text{-}u}$	$S_{d\text{-}u}$

where:
- S_{up} = Undrained peak shear strength
- S_{us} = Undrained steady-state shear strength
- S_{ds} = Drained steady-state shear strength
- S_r = Residual shear strength of liquefied soil
- $S_{d\text{-}u}$ = Shear strength determined by effective stresses corresponding to induced pore pressure

V. DYNAMIC STABILITY: ANALYTICAL METHODS

Finite Element Method (Continued)

It is a good practice to perform a few one-dimensional analyses using the specific dam and foundation material data and design earthquake loading prior to attempting a two-dimensional analysis. Similarly, if a three-dimensional analysis is considered, it should be preceded by a two-dimensional analysis. The lower dimension analyses are easier to make and provide the engineer an opportunity to resolve any questionable material data and get an estimate for the computed response. Additional refinements in the computed response can be obtained by making higher dimension analyses.

POTENTIAL FOR LIQUEFACTION

The phenomenon of liquefaction of loose saturated sands, gravels, or silts having a contractive structure may occur when such materials are subjected to shear deformations with high pore water pressures developing, resulting in a loss of resistance to deformation (Engineering Guidelines for the Evaluation of Hydropower Projects, Federal Energy Regulatory Commission, 1988; T. A. Mansouri, J. D. Nelson, and E. G. Thompson, 1983). Analyses must be performed to determine:

- If liquefaction potential exists, and

- When such a liquefied condition can lead to failure or excessive deformations of an embankment.

The details of these two analyses are described below.

Liquefaction Potential

The potential for liquefaction in an embankment and/or its foundation must be evaluated on the basis of empirical knowledge and engineering judgment supplemented by special laboratory tests when necessary. Simplified methods for evaluating soil liquefaction potential relate blow count values from standard penetration tests to safe, unsafe, and marginal conditions. These methods are discussed in the articles **Liquefaction of Sands**, Harvard Soil Mechanics Series, No. 81 by G. Castro; and **Simplified Procedure for Evaluating Soil Liquefaction Potential**, Journal of the Soil Mechanics and Foundations Division by H. B. Seed and I. M. Idriss (see References, Appendix B). These empirical charts relate to observations of manifestations of increase of pore water pressure under level ground, such as sand boils. The empirical charts should be considered only as a guide for identifying zones within the dam and its foundation that may require further study.

Loss Of Stability (Continued)

For soils which develop a condition of $r_u = 100$ percent, the value of S_{us} or S_r is likely to control the stability of the slope and appropriate values may be determined as follows:

- Based on empirical information from liquefaction slides in similar soils. There is a general correlation between values of S_r and values of $(N_1)_{60}$, the normalized standard penetration resistance of sands and silty sands, presented in the article **Design Problems In Soil Liquefaction** by H. B. Seed, published in the Journal Of Geotechnical Engineering (see Appendix B).

- Based on laboratory tests using the procedures described in the article **Liquefaction Evaluation Procedure** by S. J. Poulos, G. Castro, and J. W. France, published in the Journal Of Geotechnical Engineering (see Appendix B). In interpreting the test data, it should be noted that values of S_{us} are very sensitive to void ratio changes, and thus, it is necessary to apply corrections to laboratory measured strengths to obtain in situ values and for possible void ratio redistribution during the period of earthquake shaking, and to interpret the results conservatively.

If the stability analysis indicates no potential for a flow failure, then a deformation analysis should be performed.

Deformation

Deformation computations are applicable only to dams not subject to a liquefaction-caused stability failure.

Deformations can be assumed not to be a problem if the dam is well-built, that is, densely compacted, and peak accelerations are 0.2 g or less. If this condition is not satisfied, a deformation analysis should be made. This analysis can be made using the method discussed in the previous section on dynamic displacement analysis method (page V-23). The deformation along the failure plane calculated by this method should not generally exceed 2 feet. Larger deformations may be acceptable depending on available freeboard, and ability of the embankment to heal cracks.

The basic steps involved in conducting a deformation analysis are as follows:

1. Determine the magnitude and source of the earthquake or earthquakes that should be considered.

2. Determine the time-history or time-histories of the ground motion associated with the earthquake or earthquakes.

Deformation (Continued)

3. Determine the yield strength of the embankment and foundation materials.

4. Determine the dynamic response of embankment and foundation materials.

5. Predict the extent of deformations resulting from earthquake shaking.

6. If predicted deformations are not tolerable, explore remedial action alternatives that would provide a tolerable response.

ORDER OF ASSESSMENT

An assessment of the dynamic stability of an existing embankment dam and its foundation involves the following steps:

- Review of the design, construction, and performance data.

- Determination of the magnitude and distance of the earthquake(s) from the damsite.

- Estimation of possible performance of embankment and foundation materials by comparisons with case history prototype performances.

- Determination of the time-acceleration data that would be used in the dynamic analysis.

- Determination of the dynamic properties of the embankment and foundation materials.

- Prediction of deformations and stresses resulting from earthquake loading.

- If predicted deformations and/or stresses are not tolerable, take remedial action.

☞ **NOTE:** Sometimes the dynamic analyses of an existing embankment dam and its foundation are made using estimated values for their material properties. These estimates for material property values are based on:

- Historical data covering many damsites.
- Limited site-specific field and/or laboratory tests for the dam under study.
- Past laboratory test reports on the project under study.

ORDER OF ASSESSMENT (Continued)

While this may be an acceptable practice for preliminary work in a dam safety evaluation process, it is essential that final evaluations and recommendations for remedial work be based on material property values obtained from appropriate laboratory and field tests on a site-specific basis.

It is always a good practice to compare the tested values with the historical data on similar materials or empirical relations, and to resolve the differences. The end objective is to get the best representative property values for the materials involved.

V. DYNAMIC STABILITY: REMEDIAL ACTION

INTRODUCTION

The seismic evaluation of an embankment dam and its foundation provides the engineer quantitative information about dynamic stability deficiencies. The means to rectify these deficiencies can be difficult because of existing conditions, including the presence of the dam itself. The adaptability of possible remedial actions will have to be evaluated on an individual project basis. However, corrective actions need to be taken to ensure the safe, continued presence of the dam and reservoir.

REMEDIAL MEASURES

The commonly encountered dynamic deficiencies and corresponding remedial actions considered are:

- The embankment and its foundation do not contain loose sands, silts, and gravels, with the potential to liquefy under earthquake loading. The remedial measures described in Unit IV to improve the static stability of an embankment dam should be considered.

- The embankment does not contain loose sands, silts, and gravels, but the foundation contains these potentially liquefiable materials. The following remedial treatment alternatives should be considered:

    - Dewater the loose saturated soils
    - Densify the loose saturated soils
    - Drain or lower the reservoir
    - Buttress the downstream embankment slope with a berm

- The embankment contains loose sands, silts, and gravels with the potential to liquefy under earthquake loading, but the dam foundation does not contain these materials. The following remedial treatment alternatives should be considered:

    - Rebuild the dam incorporating modern dam design and construction practices
    - Flatten the downstream and upstream embankment slopes by adding select materials
    - Drain or lower the reservoir
    - Densify the embankment

V. DYNAMIC STABILITY: REMEDIAL ACTION

REMEDIAL MEASURES (Continued)

- The embankment and its foundation contain loose sands, silts, and gravels with the potential to liquefy under earthquake loading. The following treatment alternatives should be considered:

    - Rebuild the dam incorporating modern dam design and construction practices
    - Drain the reservoir
    - Densify the embankment and its foundation

- Embankment overtopping deficiencies. The following remedial action alternatives should be considered:

    - Raise the dam
    - Restrict the reservoir level

A downstream warning system can be installed to provide notice of the need to evacuate people downstream from the dam, before the arrival of flood waters. It should be a part of the emergency action plan.

V. DYNAMIC STABILITY: SUMMARY

SUMMARY: DYNAMIC STABILITY

Unit V provided background information on when dynamic analysis is warranted, the modes of failure related to dynamic instability, and the objectives of dynamic stability analysis. In addition, this unit described what project data should be reviewed, determining the dynamic properties of soils, and the analytical methods and approaches to evaluating dynamic stability. Lastly, remedial measures for dealing with potential dynamic instability were presented.

Review And Evaluation Of Project Data

The types of project data that should be reviewed when evaluating dynamic stability include:

- Site geology
- Dam design and construction
- Reservoir operations and dam performance
- Static analysis results
- Instrumentation data
- Landslide activity along the reservoir

During this review, the presence or absence of favorable and unfavorable factors for the embankment dam's ability to perform satisfactorily during an earthquake should be identified.

Dynamic Properties Of Soils

The dynamic properties of embankment and foundation soils of interest in a dynamic stability analysis are:

- Shear stress - shear strain - damping characteristics under cyclic loading
- Shear strength under cyclic loading
- Susceptibility to liquefaction
- Field and laboratory tests for dynamic property determination, including:
  - Geophysical in situ investigations involving:
    - Steady-state vibratory source methods
    - Impulsive transient source methods
  - Laboratory testing for dynamic properties and cyclic strength.
  - Soil liquefaction tests

V. DYNAMIC STABILITY: SUMMARY

Analytical Methods

Several approaches and analytical methods are used to evaluate the dynamic stability of embankment dams:

- Pseudostatic method
- Dynamic Displacement Analysis method
- Dynamic Response methods
  - Ground Response Analysis method
  - Pore Pressure Response Analysis method
  - Finite Element method
  - Liquefaction Potential evaluation

Remedial Action

Temporary and long-term measures to alleviate the effects of dynamic instability were stated.

APPENDIX A

GLOSSARY

GLOSSARY

ABUTMENTS--Those portions of the valley sides which underlie and support the dam structure, and are usually also considered to include the valley sides immediately upstream and downstream from the dam.

ACCIDENT--An incident where failure was prevented by remedial work or operating procedures, such as drawing down the reservoir.

ACTIVE FAULT--A fault, reasonably located, known to have produced historical earthquakes or showing geologic evidence of displacements and which, because of its present tectonic setting, can undergo movement during the lives of manmade structures.

AXIS OF DAM--The vertical plane, chosen by a designer, appearing as a line in a plan or cross-section, to which the horizontal dimensions of the dam are referenced. The axis is usually coincident with the center line of the crest of the dam.

BERM--A step in the sloping profile of an embankment dam. A step in a rock or earth cut. Also, a placement of fill at the toe of a slide to buttress it against further movement.

CORE--A zone of material of low permeability in an embankment dam.

DIAPHRAGM WALL--A wall of impervious material built through the embankment dam and into the foundation to reduce seepage under the dam as well as through the embankment.

DRAINAGE BLANKET--A drainage layer placed directly over the foundation material.

DRAINAGE LAYER--A layer of pervious material in an earthfill dam to relieve pore pressures or to facilitate drainage of the fill.

DRAINAGE WELLS or RELIEF WELLS (DRAINAGE CURTAIN)--A series of wells or boreholes to facilitate drainage of the foundation and abutments and to reduce water pressure. (This terminology generally is used with concrete dams.)

EARTHFILL DAM--A dam containing more than 50 percent, by volume, earthfill materials (fill composed of soil and rock materials that are predominantly gravel-sized or smaller).

EMBANKMENT--Fill material, usually earth or rock, placed with sloping sides.

EMBANKMENT DAM--Any dam constructed of excavated natural materials. Includes both earthfill and rockfill dams.

GLOSSARY

EMERGENCY--A condition of serious nature which develops unexpectedly and endangers the structural integrity of a dam or endangers downstream property and human life, and requires immediate action.

EMERGENCY ACTION PLAN (EAP)--A plan designed to alleviate hazards or reduce damages that may be caused by flooding due to dam failure or unusually high flow through the spillway system. An EAP contains procedures to be followed in the event of structural malfunctions or the occurrence of a natural event that approaches or exceeds the design limits of the dam.

EPICENTER--That point of the earth's surface that is directly above the focus of an earthquake.

FAILURE--An incident resulting in an uncontrolled release of water from a dam.

FAULT--A fracture or fracture zone in the earth crust along which there has been relative displacement of the two sides.

FILTER or FILTER ZONE--A band of granular material which is incorporated in an embankment dam and is graded so as to allow seepage to flow across or down the filter zone without causing the migration of the material from zones adjacent to the filter.

FOCUS (HYPOCENTER)--The point within the earth that is the center of an earthquake and the origin of its elastic waves.

FOUNDATION--The portion of the valley floor that underlies and supports the dam structure.

FREEBOARD--The vertical distance between a stated water level and the top of a dam or spillway crest.

HOMOGENEOUS EARTHFILL DAM--An embankment dam constructed throughout of more or less uniform earth materials, except for possible inclusion of internal drains or blanket drains.

HYDRAULIC FILL DAM--An embankment dam constructed of materials, often dredged, which are conveyed and placed by suspension in flowing water.

HYDRAULIC FRACTURING--The creation and propagation of cracks in an impervious embankment material due to excessive hydraulic pressures.

GLOSSARY

INTENSITY SCALE--An arbitrary scale to describe the degree of shaking at a particular place. The scale is not based on measurement but on a description by an experienced observer. Several scales are used (e.g., the Modified Mercalli scale, the MSK scale), all with grades indicated by Roman numerals from I to XII.

INTERNAL EROSION--See **PIPING**

LOADING CONDITIONS--Conditions to which a dam is exposed, for example gravity, earthquakes, and floods.

MAGNITUDE--A rating of the size of an earthquake by numerical values, such as M5.6, M8.2, etc. The magnitude number is calculated by means of the logarithm of the amplitude of matrices recorded by a standard seismograph at a known distance from the origin of the earthquake. Each higher whole number expresses an amount of energy released that is approximately 60 times larger than that expressed by the preceding whole number. For example, an M6 earthquake will release about 60 times the energy of an M5 earthquake.

PIPING--The progressive internal erosion of embankment, foundation, or abutment material.

PORE PRESSURE--The pressure of water in the voids within a mass of soil, rock, or concrete.

RICHTER SCALE--A scale proposed by C. F. Richter to describe the magnitude of an earthquake by measurements made in well-defined conditions and with a given type of seismograph. The zero of the scale is fixed arbitrarily to fit the smallest recorded earthquake. The largest recorded earthquake magnitudes are near 8.7. This is the result of observations and not an arbitrary upper limit like that of the intensity scale.

RISK--The probability that an adverse event such as a dam failure will occur.

RISK ASSESSMENT--As applied to dam safety, the process of identifying the likelihood and consequences of dam failure to provide the basis for informed decisions on a course of action.

ROCKFILL DAM--A dam containing more than 50 percent rockfill materials (predominantly cobble sized or larger).

SEEPAGE--The passage of water through embankment, foundation, or abutment material.

SHELL--The upstream and downstream parts of the cross section of an embankment dam on each side of the core.

GLOSSARY

SLOPE PROTECTION--The protection of embankment slope against wave action or erosion.

TECTONIC PROVINCE--A geologic area characterized by similarity of geologic structure and earthquake characteristics.

TOE OF DAM--The junction of the downstream slope of a dam with the ground surface; also referred to as the **downstream toe**. For an embankment dam, the junction of the upstream face with the ground surface is called the **upstream toe**.

TOE WEIGHT--Additional material placed at the toe of an embankment dam to increase its stability.

TRANSITION ZONE--A substantial part of the cross section of an embankment dam comprising material whose grading is of intermediate size between that of an impervious zone and that of a permeable zone.

UPLIFT PRESSURE--Upward water pressure in the pores of a material or on the base of a structure.

UPSTREAM BLANKET--An impervious blanket placed on the reservoir floor upstream of a dam.

ZONED EARTHFILL DAM--An earthfill-type dam, the cross section of which is composed of zones of selected materials having different degrees of porosity, permeability, and density.

APPENDIX B

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APPENDIX C

ADDITIONAL READING

ANALYSIS OF FOUNDATION SETTLEMENTS AT RIDGWAY DAM

Ashok K. Chugh and Luther W. Davidson
Canadian Geotechnical Journal,
Vol. 25, No. 4, pp. 716 - 725, 1988.

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Analysis of foundation settlements at Ridgway Dam

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The foundation material at the Ridgway Dam site is broadly classified as mudstone. The observed foundation settlements along the invert of the river outlet-works conduit at Ridgway Dam are on the order of 0.3 m. Numerical analyses were performed to estimate the deformation properties for a foundation material that under the existing embankment loads would deflect in a manner similar to the settlements surveyed along the invert of the outlet-works conduit. The foundation deformation properties determined from these analyses are compared with those obtained through the laboratory testing of the site-specific foundation materials and the published data. The results of the analyses, the field instrumentation data, the site geology, and the laboratory data provided an input to the decision-making process for the rehabilitation of the river outlet-works conduit.

Key words: foundations, settlements, embankment dams, mudstones, analysis.

Le matériau de fondation sur le site du barrage Ridgway est généralement classifié comme un mudstone. Les tassements observés de la fondation du radier de la conduite de fuite dans la rivière sont de l'ordre de 0,3 m. Les analyses numériques ont été réalisées dans le but d'estimer les propriétés de déformation pour le matériau de fondation qui, sous les charges du remblai existant, va subir une déflexion similaire aux tassements qui ont été relevés le long du radier de la conduite de fuite. Les propriétés de déformation de la fondation déterminées au moyen de ces analyses ont été comparées à celles qui ont été obtenues par des essais de laboratoire sur le matériau de fondation spécifique à ce site, et les données sont publiées. Les résultats des analyses, les données de l'instrumentation sur le chantier, la géologie du site, et les données de laboratoire ont fourni un ensemble d'éléments utilisés dans le processus de décision quand à la méthode de réhabilitation de la conduite de décharge dans la rivière.

Mots clés : fondations, tassements, barrages en terre, mudstones, analyse.

[Traduit par la revue]

Can. Geotech. J. 25, 716–725 (1988)

Introduction

Ridgway Dam is a zoned earthfill embankment across the Uncompahgre River in Ouray County near Montrose, Colorado, U.S.A. The embankment dam has a maximum height of 102 m[1] above the stream bed and a crest length of approximately 750 m. Figure 1 shows the location map and general layout of the Ridgway Dam and its appurtenant structures. The dam was completed in 1987.

The river outlet-works conduit is located on a relatively flat foundation and has about 65.5 m of embankment fill above it under the crest of the dam (see Fig. 1). Figure 2 shows the profile and some cross-sectional details along the outlet-works conduit.

In January 1986, cracking of the river outlet-works conduit was observed and a survey of the conduit invert was made. This survey indicated that settlement had occurred. The maximum settlement was 0.23 m, near station 11 + 11. At the time of this survey, embankment construction near the river outlet-works conduit had been completed to elevation 2078.7 m. Embankment construction was completed in September 1986 when the crest elevation of 2098.9 m was reached. A second survey was completed in October 1986. It indicated that the maximum settlement was 0.28 m, near station 11 + 24. Another survey in early December 1986 showed that the total settlement near station 11 + 24 had increased to 0.29 m. Two additional surveys later in December 1986 indicated no additional settlement. Figure 2 shows the surveyed settlements along the invert of the conduit. From March to September 1987 there had not occurred additional settlements along the conduit length due to reservoir loads.

There are several methods and practices available for use in predicting settlements of structures (Hamdy 1986). Their use in engineering practice is a matter of individual or organizational preference and past experience.

The objectives of this paper are:
(1) to present the rationale for selecting the particular analysis procedures for estimating the deformation properties of a foundation material that under the existing embankment loads would deflect in a manner similar to the settlements surveyed along the invert of the river outlet-works conduit;
(2) to present the results of numerical analyses;
(3) to present a comparison of numerical analysis results with the laboratory data on site-specific foundation materials and the published data from the literature.

It should be kept in mind that the cumulative settlement data and the embankment loading causing the settlements were the only reliable site-specific data available for analysis purposes at the time of this study. The results of laboratory investigative studies, performed in conjunction with the foundation settlements, became available toward the end of the analytical studies. The preconstruction laboratory data could not be completely relied upon because the observed settlements were considerably greater than anticipated. The preconstruction geologic investigations and foundation exploration data were available and used only for the benefit of the problem definition. The problem as posed for analysis is incomplete. The

[1]Imperial units were used on this project. The data and analyses reported in this paper were converted, wherever practicable, to metric units and conveniently rounded. The numeric information contained in this paper should, therefore, be interpreted keeping in mind this change of units.

TABLE 1. Preconstruction rock mechanics laboratory unconfined compression strength test results on mudstone samples (Babcock 1983)

Sample depth (m)	Sample length/diameter	Unconfined compressive strength, q_u (MPa)	Static secant modulus of elasticity, E_s, at 40–60% of ultimate strength (MPa)	Calculated undrained shear strength, $S_u = \frac{1}{2} q_u$ (MPa)	E_s/S_u (approx.)
17.5	1.60	0.5	34	0.25	141
22.3	2.06	3.7	207	1.8	113
22.8*	2.02	4.3	414	2.2	192
22.9	2.01	2.1	207	1.1	193
25.5	1.81	18.5	1380	9.2	149
140.3	1.97	32.8	6895	16.4	421
142.6	2.15	44.6	96525	22.3	4328

*Specimen dried during preparation.

back-calculated values of the operating deformation properties for the foundation material shall depend on the assumptions made in defining the problem. Therefore, the reasonableness of back-calculated values of deformation properties of the foundation material must be evaluated in view of the site-specific laboratory data, and other data available in the literature. Even though this comparison is after-the-event, it may serve as a useful learning exercise for future use in geotechnical engineering practice.

Though it may appear to be an unusual set of conditions for an engineering problem, it did happen in practice and requires a solution. Thus, the approach to the problem at hand and the methods of analysis adopted may be of equal significance.

A brief description of the site geology and representative site-specific laboratory data is presented first, then the main objectives of the paper. Additional information on these items can be obtained from the authors on request.

Site geology

The dam and the river outlet-works conduit are founded on the Morrison Formation of Jurassic age. The Morrison Formation is about 213 m thick near the damsite and is divided into the upper Brushy Basin member and the lower Salt Wash member. The Brushy Basin member is exposed in the damsite area and is the foundation for the river outlet-works conduit. This formation consists mainly of shale and mudstone units with random, generally thin- to medium-bedded sandstone and siltstone layers. The Salt Wash member was not encountered during the dam construction and is thought to occur at more than 30 m below the conduit. The Salt Wash member contains massive sandstone beds interstratified with layers of mudstone.

Five shallow drill holes with depths 2.4–15 m below the conduit were completed in conjunction with this investigation. The geologic logs and visual inspection of the drilled core show high variability in the thickness and integrity of the mudstone layers. Based on these logs, it is estimated that approximately 26–33% of the foundation material is very soft to medium mudstone (q_u = 0.2–0.7 MPa).

Applying the estimate of 30% of the foundation material to be of soft to medium mudstone to a depth of 30 m below the conduit, one would infer a thickness of compressible foundation material of ~9 m.

Laboratory data

The preconstruction rock mechanics laboratory tests on mudstones from the Ridgway Dam site were performed on core samples from the dam's drainage and grouting tunnel. These test results are shown in Table 1 (Babcock 1983.)

To study the problems associated with the conduit settlement, additional soil mechanics laboratory testing was performed on the very soft to medium mudstone samples taken from under the river outlet-works conduit. Eleven NX size and 15.25 cm diameter waxed core samples and three 15.25 cm diameter samples protected in split polyvinyl chloride pipe were obtained for laboratory investigations. All tests were performed in accordance with procedures described in the Earth Manual (1980). Some of these representative test results are shown in Tables 2 and 3 (Redlinger and Casias 1987).

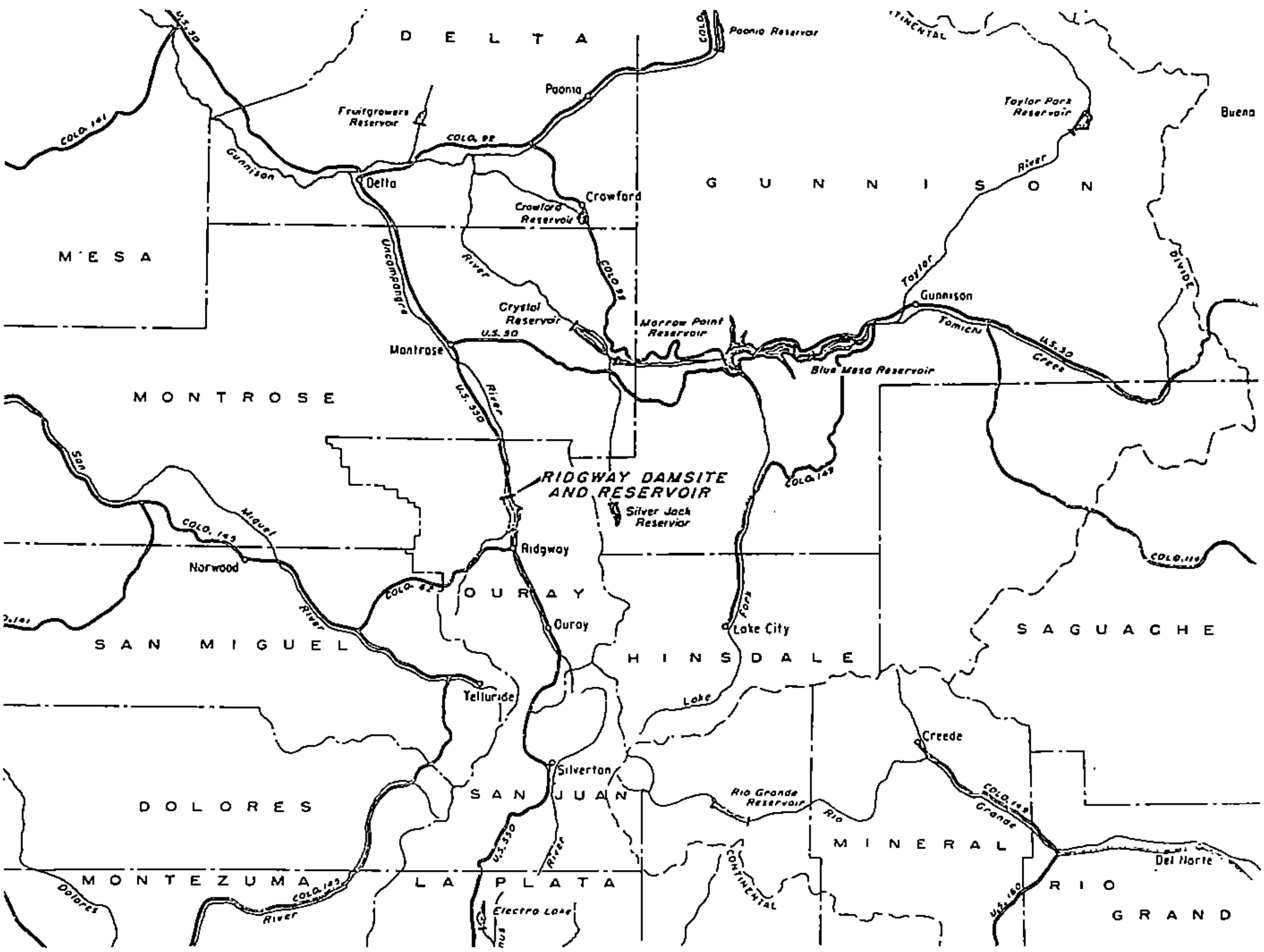
Rationale for analyses

The compressibility of a foundation material may be characterized in terms of
(1) coefficient of subgrade reaction, K;
(2) Young's modulus of elasticity, E, and Poisson's ratio, ν;
(3) recompression index, C_r, and (or) compression index, C_c, and initial void ratio, e_0.

Associated with each of the above characterizations of material is a method of settlement calculation. Obviously, one needs to make additional assumptions with regard to material behaviour, i.e., linear or nonlinear for characterizations (1) and (2), normally consolidated or overconsolidated for (3); thickness of foundation undergoing compression for (2) and (3); boundary conditions for (1), (2), and (3), etc. For purposes of this paper, only linear, homogeneous, and isotropic properties for K, E, ν, and a uniform value for the slope of the $e - \log p$ curve for C_c are considered.

The motivation for the choice of analysis methods came, in general, from the following considerations:
(1) The embankment load and the foundation settlement data have provided a pseudo-plate bearing test of the prototype foundation and one should be able to calculate the coefficient of subgrade reaction, K, which is an average representation of the load–deformation behaviour of the entire foundation under the dam. The magnitude of K shall indicate whether the foundation behaviour is one of a soil-like material or a rock-like material.
(2) If the foundation deformations occurred over a short time, the foundation response to embankment load should be essentially elastic, and one needs to know E and ν.
(3) If the foundation deformations occurred over some time, the foundation settlement under embankment load should be due to consolidation of the foundation materials, and one needs to know C_r, C_c, e_0, etc.

The number and significance of assumptions required for



LOCATION MAP

FIG. 1. Location map and general layout of the Ridgway Dam and appurtenant structures.

making the analyses depended on the analysis procedure adopted. These are described in the following section (Chugh 1987).

Analyses and results

1. Coefficient of subgrade reaction

The analytical model for this calculation is shown in Fig. 3. In this approach the surveyed settlement data are used to calculate the total vertical reaction assuming a uniform coefficient of subgrade reaction, K, and seeking static equilibrium of forces in the vertical direction (see Fig. 3). The main assumptions of this procedure are

—no interelement shear;
—a uniform and linear load–displacement response of the foundation material;
—only vertical displacements;
—an incompressible foundation underlies the compressible zone.

The calculated value of K is about 6.11 MPa/m of settlement. This is indicative of a soil-like behaviour of the foundation material. Obviously, this calculation procedure does not require a prior knowledge of the thickness of the foundation material within which the settlement occurs. The results of this calculation provided a convenient measure of the deformation characteristic of the foundation material based only on the surveyed settlement data and the weight of the dam.

2. One-dimensional elastic analysis

This simple calculation procedure was used to estimate magnitude (high or low) of modulus of elasticity of the foundation material using the observed settlement data. The analytical model for this calculation is shown in Fig. 4. In the use of this approach, the thickness of compressible foundation zone at any point was assumed to be a constant fraction of the embankment height above it. A uniform modulus of elasticity value for the foundation material is calculated by seeking an equilibrium of forces in the vertical direction (see Fig. 4). The main assumptions of this procedure are the same as those for analysis 1 above.

The results of this analysis show that the modulus of elasticity, E, of the compressible foundation zone should be quite low for a reasonable depth of influence in the dam foundation.

3. Two-dimensional elastic analysis

The analytical model for this calculation is shown in Fig. 5. This analysis is similar to analysis 2 described above except that a uniform depth of compressible foundation is assumed and interelement shear is allowed. The table in Fig. 5 shows the assumed elastic properties for the embankment materials.

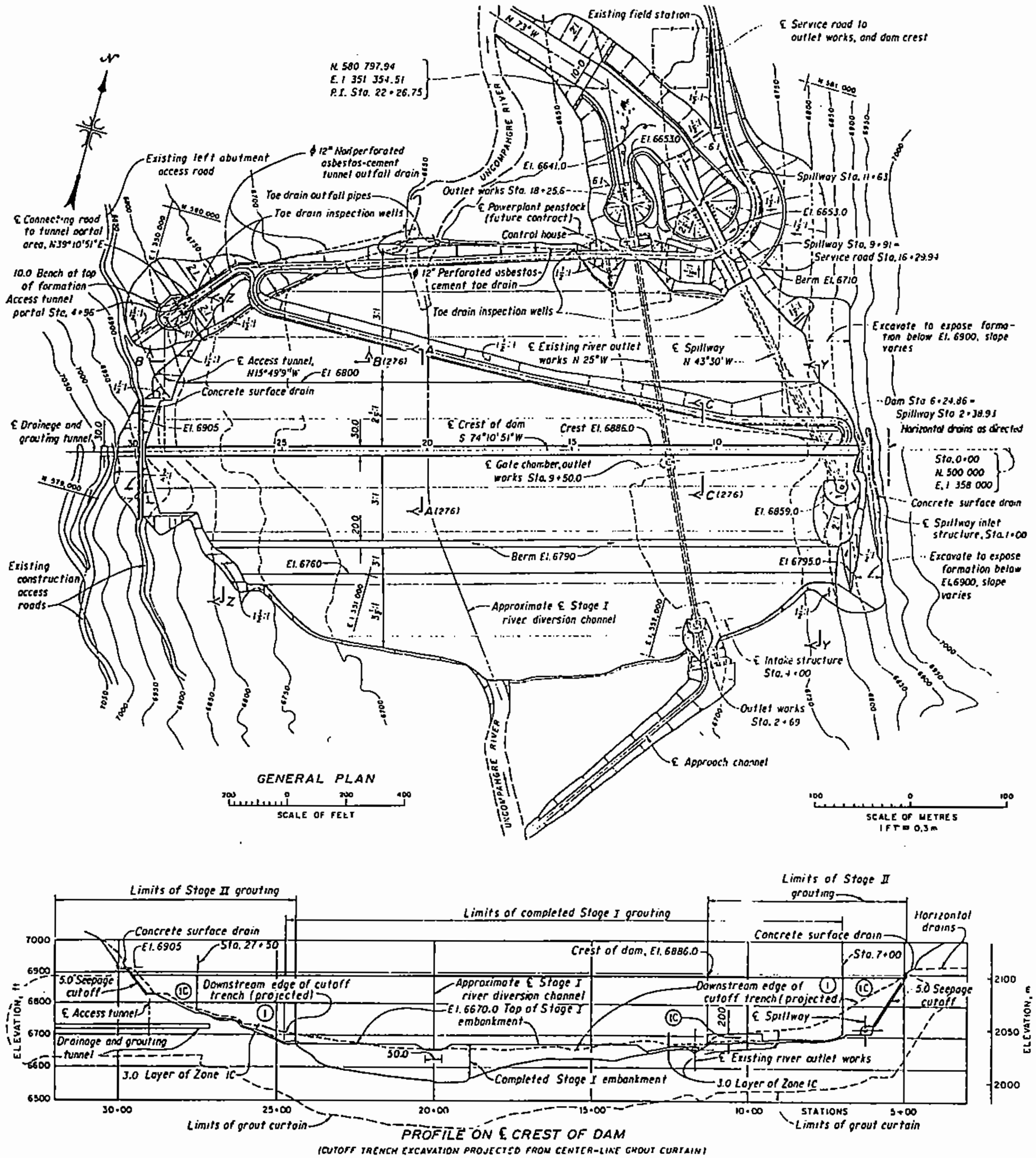


FIG. 1 (*concluded*)

By making three finite element analyses, using zero density and assumed elastic properties for the compressible foundation layer, a uniform modulus of elasticity of about ~4.86 MPa/m thickness of compressible layer was estimated to yield the deflection curve that matches well the measured settlement data (see Fig. 5). However, the thickness of compressible foundation layer is needed to fix a value for E.

The results of analyses shown in Figs. 4 and 5 should be interpreted for a reasonable thickness of the compressible layer, as the deformations are allowed to occur only in this layer.

4. One-dimensional consolidation settlement analysis

The analytical model for this calculation is shown in Fig. 6. This analysis is for the possibility that all deformations observed are a result of consolidation in soft materials. The

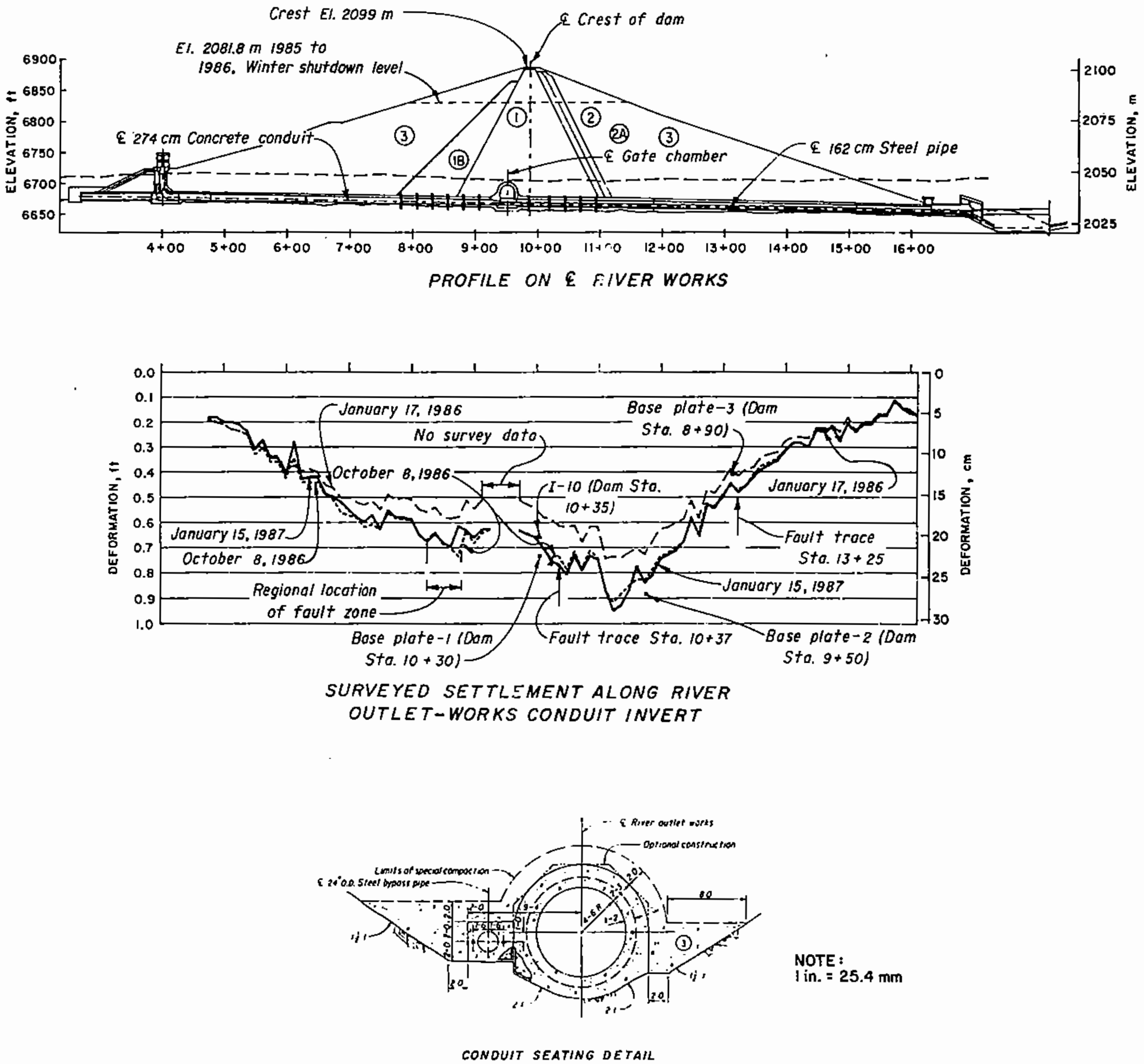


FIG. 2. General layout of the river outlet-works conduit and observed settlements along the conduit invert. Embankment explanation for circled numbers: 1. Selected clay, silt, sand, gravel, and cobbles to 5 in. maximum size compacted by tamping roller to 6 in. layers. 1A. Processed coarse-grained materials compacted by vibratory rollers to 12 in. layers (stage I only). 1B. Selected clay, silt, sand, gravel, and cobbles to 12 in. maximum size compacted by pneumatic tired rollers to 12 in. layers. 1C. Selected clay, silt, sand, and gravel of higher plasticity to 3 in. maximum size compacted by tamping rollers to 6 in. layers. 2. Processed sand filter, compacted by vibratory rollers to 12 in. layers. 2A. Processed gravel drain, compacted by vibratory rollers to 12 in. layers. 3. Selected silt, sand, gravel, and cobbles to 12 in. maximum size compacted by vibratory rollers to 12 in. layers. 3A. Processed gravel and cobbles to 12 in. maximum size compacted by vibratory rollers to 12 in. layers.

main assumptions made in this calculation were
—initial void ratio $e_0 = 1.0$ for the compressible foundation material, which allowed a convenient scaling of calculation results for other values of e_0;
—compression index, C_c, is the same for all compressible foundation material;
—change in vertical stress due to embankment load is given by the relation $\Delta\sigma = \gamma_{emb} \times h_{emb}$;
—an incompressible foundation underlies the compressible layer.

The settlement calculations were made at three different locations along the conduit using the standard formula shown in Fig. 6. The thickness of compressible layer was varied in increments of 3 m. Results of these calculations are shown in Fig. 6 for the three locations. The observed settlements at the corresponding locations are drawn in Fig. 6.

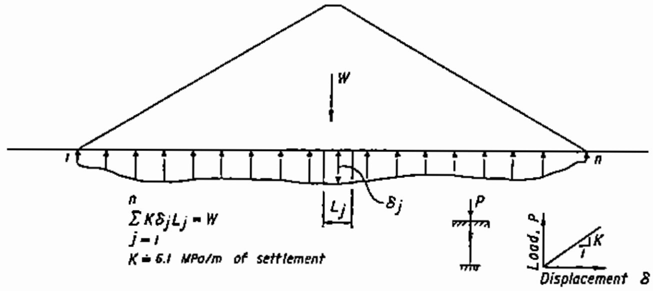


FIG. 3. Coefficient of subgrade reaction model and results.

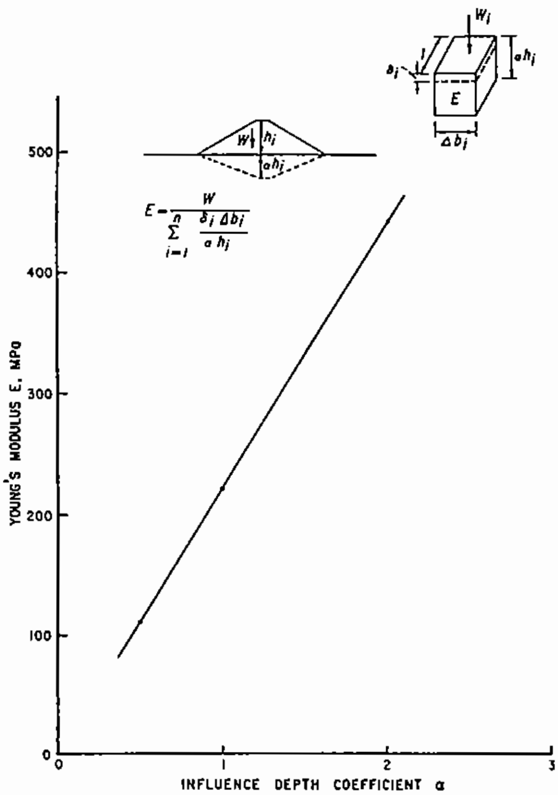


FIG. 4. One-dimensional elastic model and results.

Comparing the cumulative settlement data of November 1, 1986, with the calculated values, one can infer that

—C_c should be between 0.05 and 0.15;

—compressible foundation thickness should be between 4.6 and 9.1 m for C_c = 0.05; 1.2 and 2.4 m for C_c = 0.15.

Interrelation between elastic properties and compression index

The elastic material properties, E and ν, and the compression index, C_c, are related through constrained modulus, D, by the following equations:

$$[1] \quad D = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$$

$$[2] \quad C_c = \frac{(1+e_0)}{0.435D}\sigma_{va}$$

TABLE 2. Postconstruction soil mechanics laboratory one-dimensional consolidation test results on mudstone samples (Redlinger and Casias 1987)

Drill hole location	Sample depth (m)	Liquid limit (%)	Plasticity index (%)	Initial void ratio e_0	Initial moisture content (%)	Compression index		Recompression index		Preconsolidation pressure (MPa)
						Natural moisture content	Wet	Natural moisture content	Wet	
Under the dam	2.1–2.4	46	20	0.51	19.3	0.09		0.02		
		42	19	0.32	11.5	0.03	0.05		0.02	
Under the dam	4.3–4.7	34	14	0.53	17.9		0.13		0.04	0.62
Downstream of the dam	1.7–2.0	27	8	0.28	10.5		0.05		0.02	0.24
		35	14	0.39	13.8	0.06		0.01		0.52
Downstream of the dam	7.5–7.8	26	10	0.39	14.0	0.09		0.02		0.28
				0.35	10.5		0.06			0.31
Downstream of the dam	9.4–9.6	43	21	0.47	16.9	0.15				0.69
				0.35	12.8		0.10			0.59

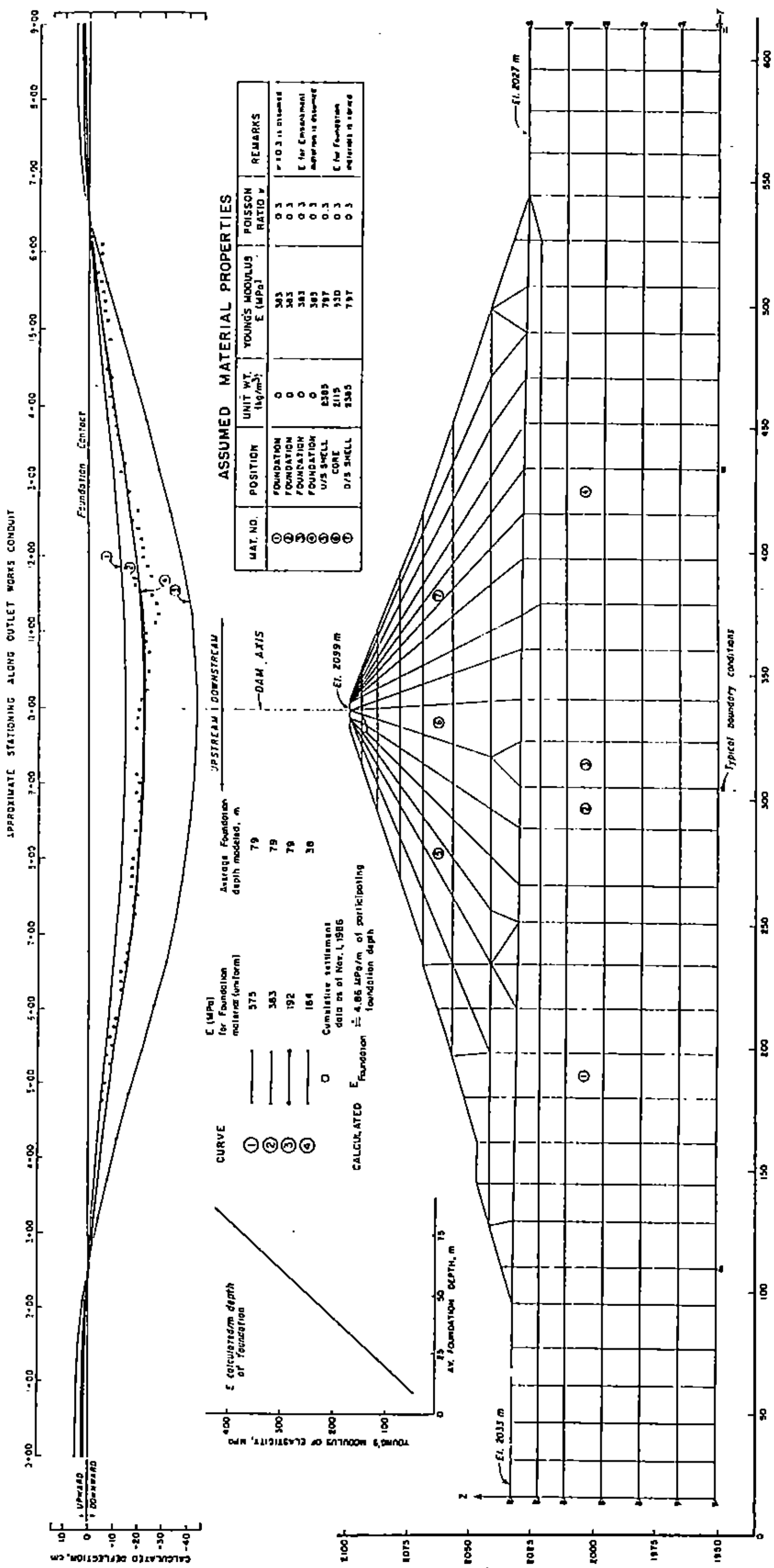


FIG. 5. Two-dimensional elastic model and results.

TABLE 3. Postconstruction soil mechanics laboratory unconfined compressive strength test results on very soft to medium mudstone samples (Redlinger and Casias 1987)

Sample depth (m)	Sample length/diameter	Unconfined compressive strength, q_u (MPa)	Tangent modulus of elasticity, E_s, at 40–60% of ultimate strength (MPa)	Calculated undrained shear strength, $S_u = \frac{1}{2} q_u$ (MPa)	E_s/S_u (approx.)
2.1–2.4	1.95	0.65	21	0.33	64
4.3–4.7	1.89	0.20	4	0.10	35
9.4–9.6	2.22	0.33	25	0.16	153

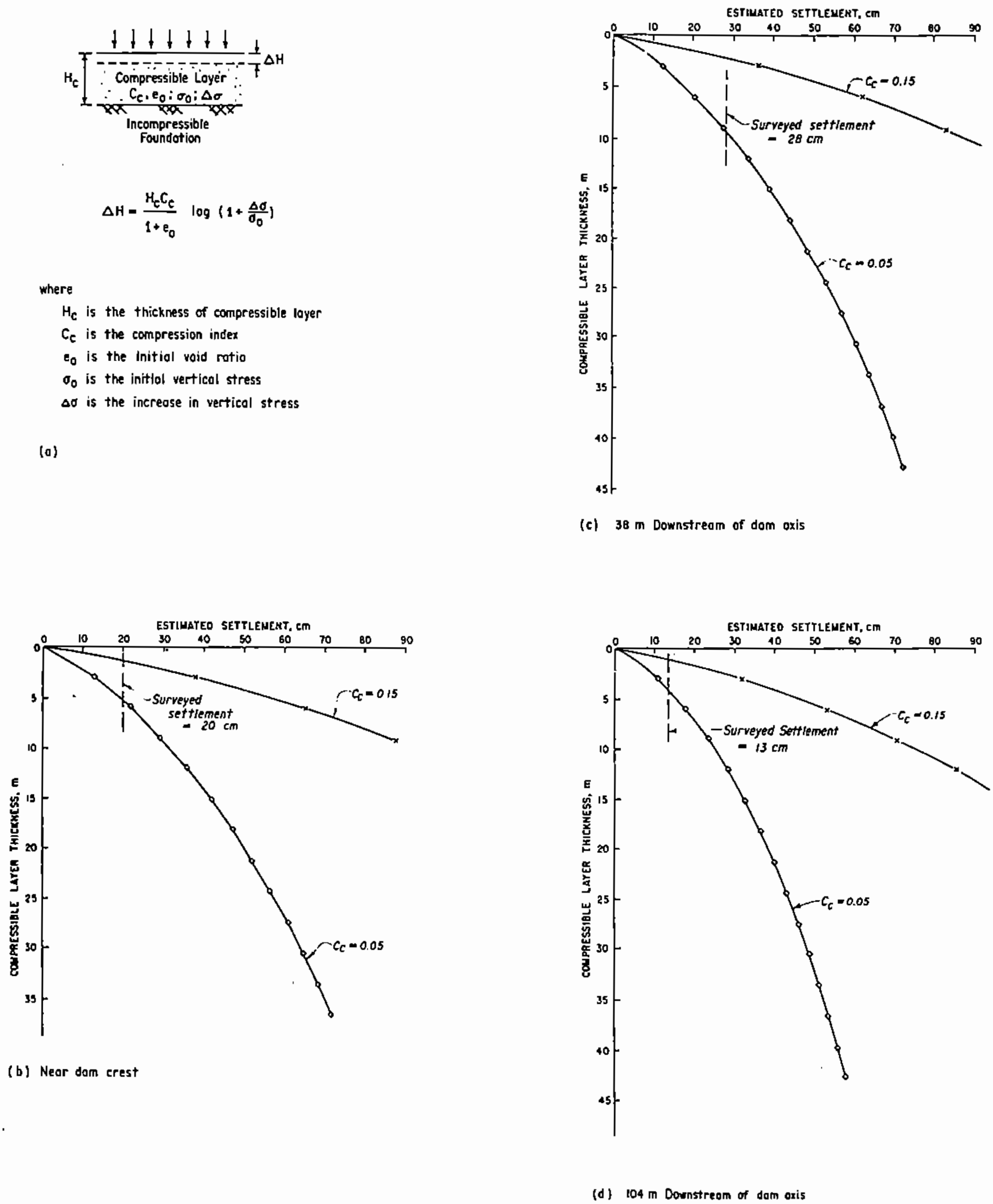


FIG. 6. One-dimensional consolidation settlement model and results.

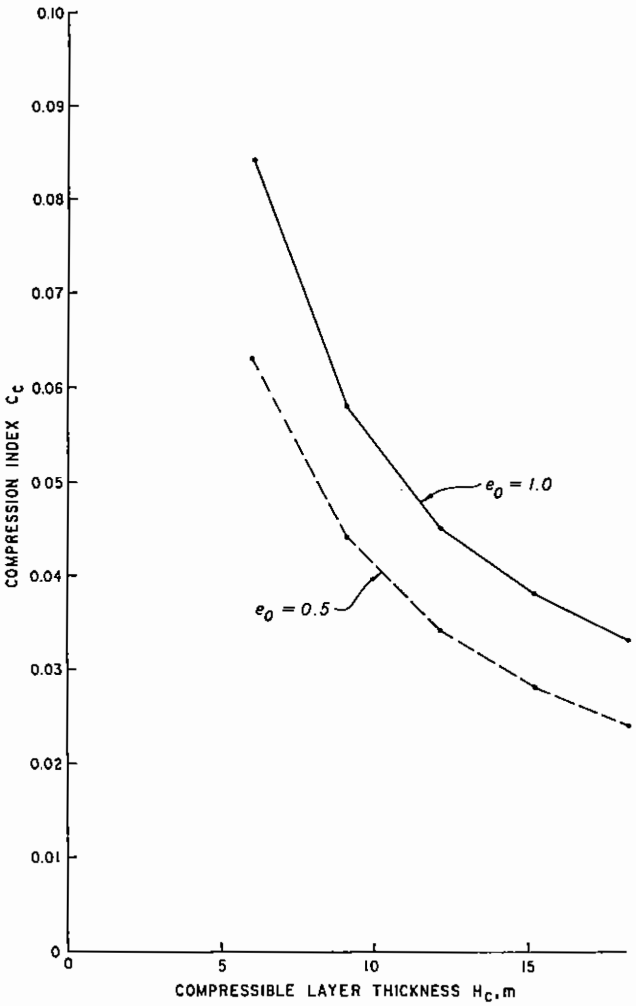


FIG. 7. Calculated interrelation between one-dimensional consolidation parameters, eq. [3].

where σ_{va} denotes the average of the initial and final vertical normal stress.

Using the results of two-dimensional elastic analysis, that is, $E = 4.86H_c$ MPa and $\nu = 0.3$, in [1], one gets $D = 6.54H_c$ MPa. H_c denotes the thickness of the compressible layer in metres. Expressing $\sigma_{va} = 0.5(\gamma_{emb} \times h_{emb} + \gamma_{found} \times H_c)$ and substituting the expression for D in [2], one gets

$$[3] \quad C_c = \frac{(1 + e_0)(0.673 + 0.0098H_c)}{2.845H_c}$$

for the near dam crest location.

Figure 7 is a plot of [3] for C_c versus H_c for $e_0 = 0.5$ and 1.0. This provides a calculated relationship, using the results of two-dimensional elastic analysis, for the one-dimensional consolidation parameters C_c, e_0, and H_c.

Discussion and summary of results

The two-dimensional elastic analysis results indicate that $E = 4.86$ MPa/m thickness of compressible layer gives a reasonable match between the computed deflection curve and the surveyed settlement along the conduit invert.

The interrelation between the elastic properties and compression index, using the results of the two-dimensional elastic model, gives possible combinations of C_c, e_0, and H_c for equally reasonable results from the consolidation settlement analysis. The results shown in Fig. 6 agree quite well with the interrelationship results shown in Fig. 7. Supplementing this information with the geologic logs, visual inspection of the drilled cores, and the local geology, one can infer that there is about 9 m of compressible material in the foundation under the river outlet-works conduit. For calculation purposes, however, this 9 m of compressible material was lumped together and placed at the embankment–foundation contact.

Using $H_c = 9$ m, one obtains $E = 44$ MPa, $\nu = 0.3$ from the two-dimensional elastic analysis; and $e_0 = 0.5$, $C_c = 0.044$ or $e_0 = 1.0$, $C_c = 0.058$ from the one-dimensional consolidation analysis as estimates for the deformation properties for a foundation material that, under the Ridgway embankment loading, would deform in a manner similar to that observed. The coefficient of subgrade reaction, K, is about 6.11 MPa/m of settlement.

Comparison of results

The C_c, e_0 and E, ν values estimated by the back analyses of the observed settlements at the Ridgway Dam are consistent with the values obtained by mathematically interrelating the two characteristic properties of soils, that is, the constrained modulus, D, and the compression index, C_c.

The preconstruction rock mechanics laboratory data on mudstone samples and postconstruction soil mechanics laboratory data on soft to medium mudstone samples from the Morrison Formation at the damsite are shown in Tables 1–3. There were variations in the rock samples, even though they were generally classified as mudstones. The secant modulus values for mudstones, Table 1, at 40–60% of the ultimate strength, range between 34 and 96 500 MPa.

The tangent modulus, at 40–60% of the ultimate strength, from the soil mechanics laboratory data for soft to medium mudstones, Table 3, range between 4 and 25 MPa. Since the softer units in the foundation must be responsible for the observed settlements, the back-analysed value for $E = 44$ MPa is in fair agreement with the laboratory data.

The computed value of compression index is in fair agreement with the laboratory one-dimensional consolidation test results shown in Table 2.

There is no published data on engineering properties of Morrison shales (Underwood 1967). Figure 8 is a plot of the uniaxial compression strength versus Young's modulus for typical rocks and clays (Legget and Karrow 1983). If one considers mudstones as a subcategory of shale, the laboratory data of E, q_u fit the published data quite well, as shown in Fig. 8. However, the laboratory data on soft to medium mudstones do not fit the statistical relations for clays, such as $C_c = 0.009$ (LL-10), $S_u = (0.11 + 0.0037I_P)\bar{\sigma}_v$, and $E = 600S_u$ (Peck 1974).

Actual conduit performance during reservoir filling

The river outlet-works conduit is instrumented with remote-reading strain gauges along its upstream length, and with settlement points and telltale gauges along the downstream length. The upstream and downstream lengths are referenced from the gate chamber (see Fig. 2). The reservoir filling commenced in March 1987 and rose from elevation 2060 m to about elevation 2083 m by July 1987. The reservoir was drawn down to elevation 2073 m in August and September 1987 to facilitate construction of upstream recreation facilities. From March to September 1987, there did not occur any discernible

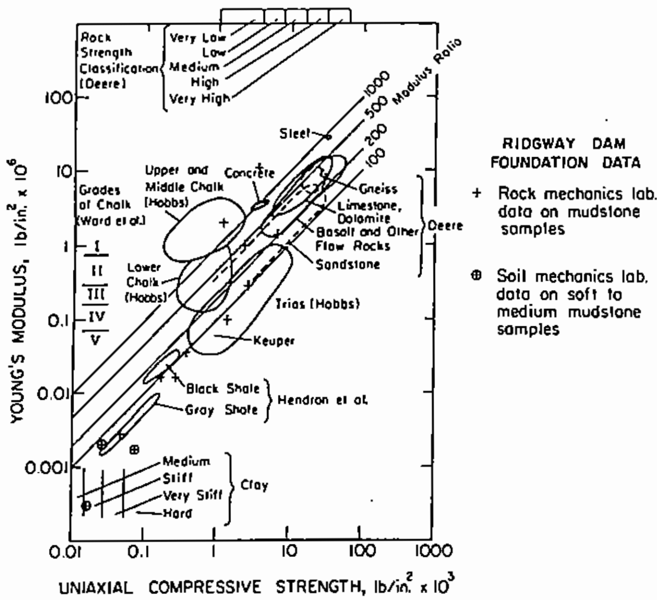


FIG. 8. Relation between the compressive strengths and Young's moduli for typical rocks and clays (Legget and Karrow 1983). (1 $lb/in^2 \doteq 6.89$ kPa.)

deformation along the conduit length due to reservoir loads. During the 1988 filling season, the reservoir rose to elevation 2086 m and again there has been no further settlement of the conduit.

Conclusions

The deformation properties for the compressible foundation material under the river outlet-works conduit at Ridgway Dam as calculated by the back analyses fit the site-specific laboratory test data and the published data quite well. Even though these comparisons are after-the-event, they provide a useful learning exercise for possible future use in geotechnical engineering practice. The analysis procedures selected for the problem were intended to be simple. While the analysis results by themselves provided a reasonable indication of possible values for the deformation properties of the compressible foundation material, a knowledge of the site geology and visual inspection of the drilled core were required to assign the numerical values to the various parameters. The values thus determined yield deflections that match well the measured settlement pattern.

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SUGGESTIONS FOR SLOPE STABILITY CALCULATIONS

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SUGGESTIONS FOR SLOPE STABILITY CALCULATIONS

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Abstract—Considerations for the selection of potential slide surface geometry in slope stability calculations by the limit equilibrium method are presented. Relations between the solution variables and the Mohr–Coulomb strength parameters for no failure on the interslice boundaries are derived. Also included are the relations between the solution variables for effective and total stress considerations. The materials are assumed to be "no-tension" type. Significance of evaluating the calculated response for the individual slices making up the potential slide mass is indicated.

NOMENCLATURE

- b width of slice
- c' cohesion with respect to effective stress
- e eccentricity of the external force P
- E horizontal component of interslice force
- F factor of safety
- H force exerted by the pore water on the interslice boundary
- h height of force above slip surface
- i ground slope
- K_A coefficient of active earth pressure
- N force normal to base of slice
- P external force acting on the slice
- S total shear force
- T interslice force for the back wedge
- W weight of slice or back wedge
- Z interslice force
- U force exerted by the pore water on the base of the slice
- α slope of base of slice
- β slope of the top of slice
- δ slope of interslice force
- θ backrest angle with horizontal

INTRODUCTION

The problem of slope stability is an important part of geotechnical engineering. As a result much has been said and written in the technical literature about various methods of analyses—their merits, complexities and simplifying assumptions, justifications for their use in actual practice, and about comparison of results obtained from their use[1, 4, 6–8, 12–14].§ Most of the slope stability analysis procedures have been converted into computer programs for their fast and accurate implementation[3, 15]. Frequent occurrence of slope design problems combined with availability of the computers, makes it reasonable to assert that at present almost every geotechnical engineer has an access to one or more of the slope stability analysis computer codes and that these codes are frequently used in engineering practice.

Amongst the different methods of analyses, limit equilibrium methods satisfying the three equations of statics are now being used extensively for estimating the stability of both natural slopes and man-made embankments[5–8, 12]. The method of slices, considering the interslice forces, as presented by Spencer[10, 11, 13], is representative of the modern versions of limit equilibrium methods. It seeks the solution of the slope stability equations, starting with an assumed value of factor of safety F and thrust inclination δ, satisfying the boundary conditions at the toe and head of the slide mass. The other parameters, such as interslice forces, their magnitude, location and direction, do not play an active role in the solution scheme but are calculated as a part of the iteration procedure. In this presentation, the boundary conditions are considered as being distinct from the interslice forces.

The objectives of the present paper are to present suggestions for:

1. Geometrical configuration of a critical segmented failure surface,
2. Interrelationship of the mathematical solution for total and effective stress considerations; and
3. Limits imposed by the material strength on the validity of the mathematical solution.

The materials are assumed to be homogeneous and isotropic and obey the Mohr–Coulomb strength hypothesis. An example of stability analysis of a natural slope is included. The following terms used in this paper are defined as follows:

Backrest: Geometric configuration of the heel of a segmented failure surface.

Thrust: Resultant force on interslice boundary.

The words "no-tension" and "cohesionless" are implied to have identical meaning.

Figure 1 is a general description of the problem. For any vertical slice, abcd, the forces acting are shown in Fig. 1(b). H_L and H_R are the hydrostatic forces exerted by the subsurface water on the vertical boundaries of the slice (assumed to be known). Other forces acting on the free body diagram of a slice are defined in the Nomen-

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§References included in this paper are representative, but not a complete list, of the works on the subject.

is not important. While these elements influence the choice of large segments of potential slide surfaces for analysis, they do not, per se, assist in estimating the toe and heel of the slide surface[2].

CRITICAL BACKREST INCLINATION

The critical backrest of a segmented slide surface should be such as to give the least contribution to the overall factor of safety against sliding. For a planar backrest of the failure surface, the forces acting on the wedge of material are shown in Fig. 2. Pore water pressure is not considered in the following derivation.

Summing forces along the backrest plane:

$$S = W\sin\theta - T\cos(\theta-\delta) \tag{3}$$

Summing forces normal to the backrest plane:

$$N = W\cos\theta + T\sin(\theta-\delta) \tag{4}$$

For the Mohr–Coulomb material, shear strength along the backrest plane is:

$$\begin{aligned}\text{shear strength} &= c.AC + N\tan\phi\\ &= c\frac{h_0\cos i}{\sin(\theta-i)}\\ &\quad+\frac{1}{2}\gamma h_0^2\frac{\cos\theta\cos i}{\sin(\theta-i)}\cos\theta\tan\phi\\ &\quad+T\sin(\theta-\delta)\tan\phi\end{aligned} \tag{5}$$

The expression for the factor of safety from equations (3) and (5) is:

$$F=\frac{\dfrac{ch_0\cos i}{\sin(\theta-i)}+\dfrac{1}{2}\dfrac{\gamma h_0^2\cos\theta\cos i}{\sin(\theta-i)}\cos\theta\tan\phi+T\sin(\theta-\delta)\tan\phi}{W\sin\theta-T\cos(\theta-\delta)} \tag{6}$$

Thus for the backrest to yield the least F, its orientation should be such as to give the minimum value for the thrust, T.

For a given material and ground slope, the thrust T depends upon W and δ, Fig. 2. For the planar backrest of the failure surface, T is a function of (θ, δ).

At the instant of shear failure, from eqn (6)

$$\begin{aligned}w\sin\theta - T\cos(\theta-\delta)\\ &= c\frac{h_0\cos i}{\sin(\theta-i)}+\frac{1}{2}\gamma h_0^2\frac{\cos\theta\cos i}{\sin(\theta-i)}\cos\theta\tan\phi\\ &\quad+T\sin(\theta-\delta)\tan\phi\end{aligned}$$

$$T=\frac{\frac{1}{2}\gamma h_0^2\cos\theta\cos i\sin(\theta-\phi)-ch_0\cos i\cos\phi}{\cos(\theta-\delta-\phi)\sin(\theta-i)} \tag{7}$$

For a cohesionless material, eqn (7) becomes

$$T=\frac{1}{2}\gamma h_0^2K_A \tag{8}$$

where

$$K_A=\frac{\cos\theta\cos i\sin(\theta-\phi)}{\cos(\theta-\delta-\phi)\sin(\theta-i)} \tag{9}$$

Differentiating $K(\theta, \delta)$ with respect to δ and equating it to zero, one gets

$$\delta=\theta-\phi \tag{10}$$

Similarly differentiating $K(\theta, \delta)$ with respect to θ, equating it to zero, and substituting eqn (10), leads to the transcendental equation:

$$\sin(\theta-\phi)\sin(\theta-i)\tan\theta-\sin(\phi-i)=0 \tag{11}$$

The solutions of the eqn (11) are graphed in Fig. 3 for several values of i. The corresponding values of K_A, the coefficient of active earth pressure, are evaluated from eqn (9) and are given in Table 1.

It may be mentioned that the zeros of the eqn (11) were approximated by the one-step linear interpolation formula[9], $x_3=(x_1y_2-x_2y_1)/(y_2-y_1)$ where x_1 to x_2 is the range in which the solution lies. This range was obtained numerically by incrementing θ through 1°.

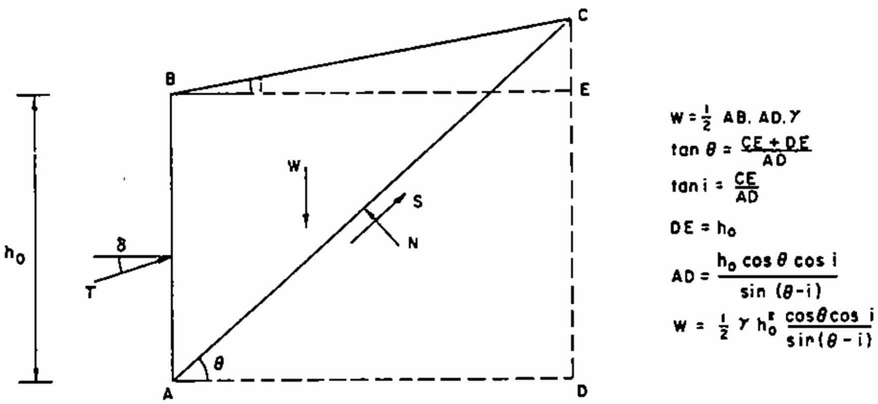

Fig. 2. Forces acting on the backrest portion of a slide mass.

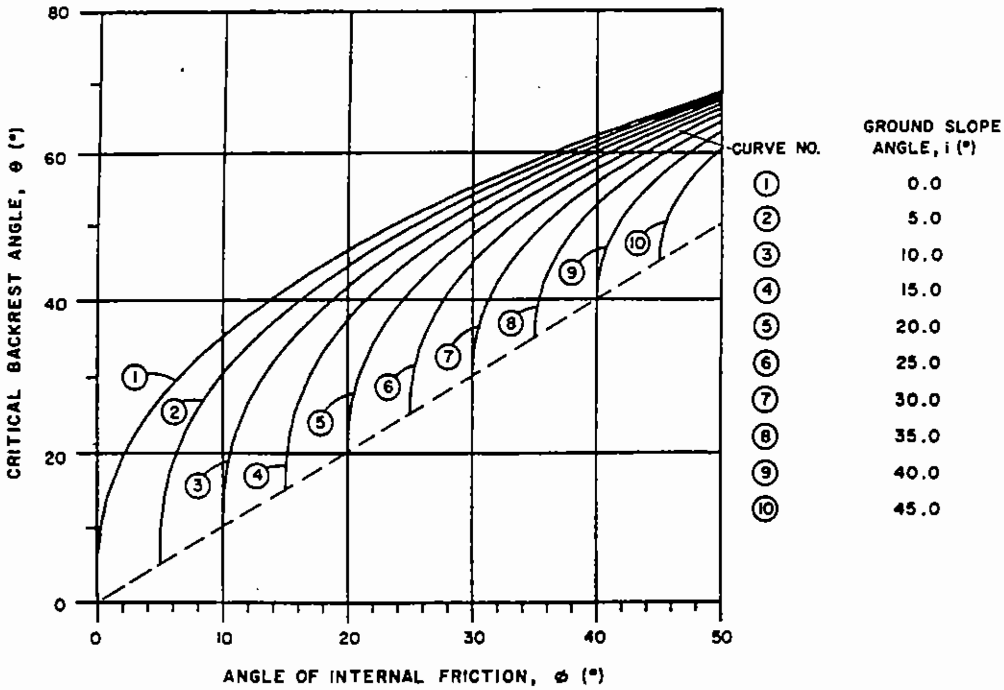

Fig. 3. Solution of eqn (11) to first approximation.

Table 1. Values of K_A

i		φ IN DEGREES								
DEG.	SLOPE	5	10	15	20	25	30	35	40	45
0	0	.735	.604	.504	.423	.355	.296	.244	.199	.160
5	1:11.5	.992	.698	.563	.463	.383	.316	.258	.209	.166
10	1:5.7		.970	.654	.517	.418	.340	.275	.220	.174
15	1:3.7			.933	.602	.467	.371	.295	.234	.183
20	1:2.7				.883	.545	.414	.322	.250	.193
25	1:2.1					.821	.485	.359	.273	.207
30	1:1.7						.750	.422	.305	.225
35	1:1.4							.671	.359	.251
40	1:1.2								.587	.297
45	1:1.0									.500

RELATION OF THRUST AND ITS INCLINATION FOR EFFECTIVE AND TOTAL STRESS

For the interslice forces to be statically equivalent for the effective and total stress, Fig. 4(a, b):

$$T\cos\delta = T'\cos\delta' + H \tag{12}$$

$$T\sin\delta = T'\sin\delta' \tag{13}$$

$$T(h_0-h)\cos\delta = T'(h_0-h')\cos\delta' + H\left(h_0-\frac{1}{3}h_1\right) \tag{14}$$

Equations (12)–(14) ensure the horizontal, vertical, and moment equivalence respectively of the interslice forces for the total and the effective stress.

From eqn (13)

$$T' = T\frac{\sin\delta}{\sin\delta'} \tag{15}$$

From eqns (12) and (15)

$$\tan\delta' = \frac{T\sin\delta}{T\cos\delta - H} \tag{16}$$

From eqns (14)–(16), one gets

$$h' = \frac{\left[T\cos\delta - \frac{1}{3}H\frac{h_1}{h}\right]h}{T\cos\delta - H} \tag{17}$$

Equations (15)–(17) relate the interslice force, its location, and orientation for total and effective stress. However eqns (12)–(17) do not account for the effect of the hydrostatic forces on the interslice boundaries on the calculated factor of safety, F. It is essential, therefore, to include these hydrostatic forces in the devivation of slope stability equations as shown in eqns (1) and (2).

LIMITS ON THRUST LINE INCLINATION AND ITS MAGNITUDE

Considering the vertical equilibrium of shear force acting on the interslice boundary and the mobilized shear

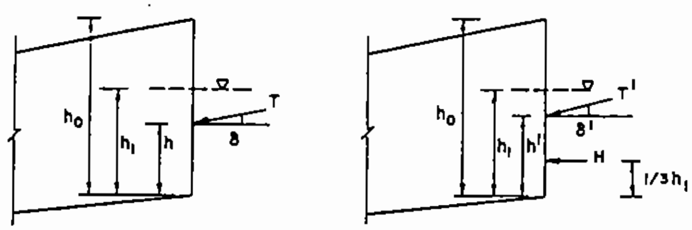

Fig. 4. Total and effective interslice forces.

strength, Fig. 4(b):

$$T' \sin \delta' = \frac{1}{F}[c'h_0 + T' \cos \delta' \tan \phi']$$

$$\tan \phi' = F \tan \delta' - \frac{c'h_0}{T'} \sec \delta'. \qquad (18)$$

Therefore, in a cohesionless material, ($c' = 0$), the necessary condition for no shear failure on the interslice boundaries is:

$$\delta' \leq \phi'. \qquad (19)$$

The corresponding inclination of the thrust for total stress is obtained from eqns (16) and (18):

$$\tan \phi' = F \frac{1}{1 - \frac{H}{T} \sec \delta} \tan \delta. \qquad (20)$$

Since $\left(\frac{1}{1-(H/T)\sec \delta}\right)$ must be ≥ 1, it follows from eqn (20) that

$$\delta \leq \phi'. \qquad (21)$$

Similarly from eqn (16), it follows that:

$$\delta \leq \delta' \qquad (22)$$

and from eqns (15) and (22) that:

$$T' \leq T. \qquad (23)$$

It is perhaps clear that equality in eqns (21)–(23) holds for $H = 0$.

LOCATION OF THRUST LINE

The slope stability eqns (1) and (2) deal primarily with the equilibrium of forces acting on the free body diagram of a typical intermediate slice. Therefore, the location of the thrust line on the interslice boundaries in terms of h_1 and h_2 is of direct significance. If the calculated location of the thrust line for a particular slice is outside the sliding mass, a tension of some magnitude and extent is implied. For no tension on the interslice boundaries, it is imperative that the thrust line be located within the sliding mass for every interslice boundary. A further assumption for normal stress distribution (such as linear) on the interslice boundary shall further define the bounds (such as middle third) within which the thrust line must be located for no tension. Since the limit equilibrium solution procedure does not consider the tensile character of the material, it is important that a designer consider the results of this calculation along with the calculated value of the factor of safety.

COMMENTS

In the derivation for critical backrest inclination presented, it is presumed that the total factor of safety of a segmented failure surface is composed of the factor of safety of its various constituents, i.e.

$$F = \sum_{i=1}^{n} f_i$$

where f_i is the contribution of the ith segment and n is the total number of segments making up the geometry of the potential slide surface. Therefore, for a slide surface to have the least factor of safety the contribution of each unit should be minimized.

In a wedge type of slope stability analysis, where the back and toe wedges of material are replaced by horizontal forces exerted by them on the middle wedge and the factor of safety of the slope is calculated by considering the equilibrium of the forces acting on the middle wedge, the backrest angle for maximum horizontal force is greater than the backrest angle that gives the least interslice force, T, used in eqn (6). Since this type of wedge analysis implies occurrence of shear failure condition on the vertical interfaces between the wedges, it gives a lower value for the computed factor of safety of the slope. For stability of natural slopes and embankments, this assumption of shear failure on the vertical interfaces of slices is unrealistic and gives unduly lower estimates of the factor of safety, and hence results in more extensive remedial treatment(s) than may be necessary to meet a design criterion of factor of safety. Alternatively, it could lead a designer into remedying a smaller zone, the middle wedge, of a potentially large slide mass and thus underestimate the extent of essential treamtnet. In any case, a wedge type of analysis for slope stability problems is unrealistic and is not recommended for general use.

Since slope stability problems generally occur in geologic formations and embankments composed of different materials and complicated by complex pore water pressure distribution, it is essential that a designer make a parametric study on the backrest and exit slopes for a segmented failure geometry. Figure 3 may be used in making an initial estimate for the backrest angle.

Since there is no way in general to predict what a solution to a nonlinear system, while satisfying the prescribed boundary conditions, may calculate for the various slices making up a slope, it is important to keep in mind the physics of the actual problem in interpreting the calculated response. Assuming com-

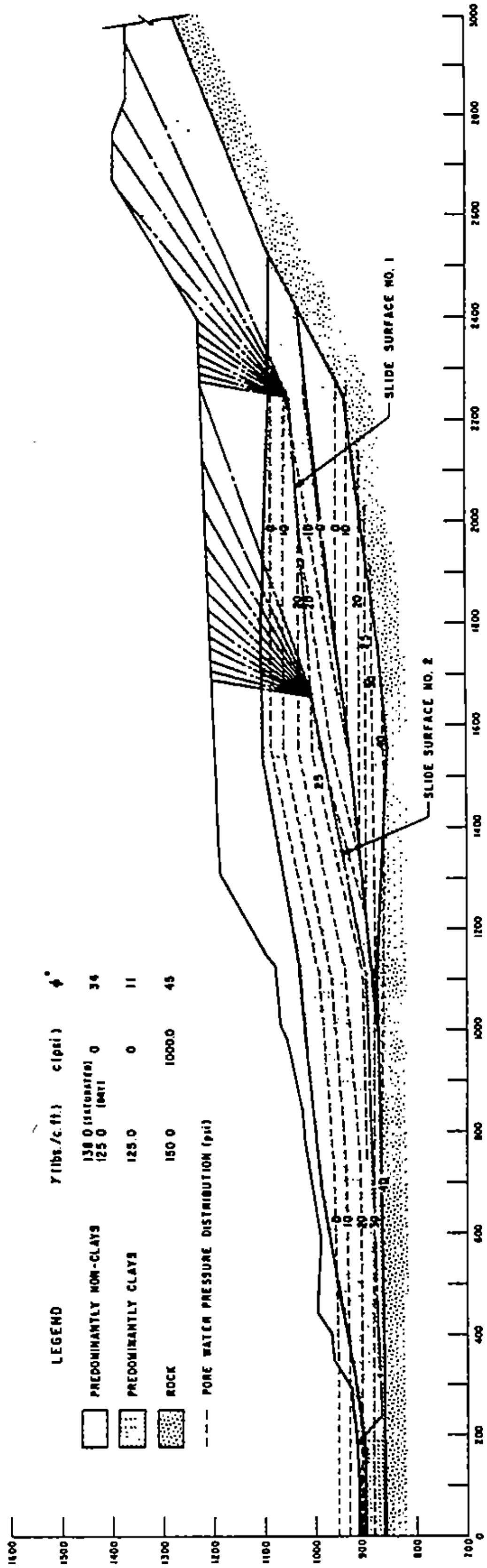

Fig. 5. Example problem.

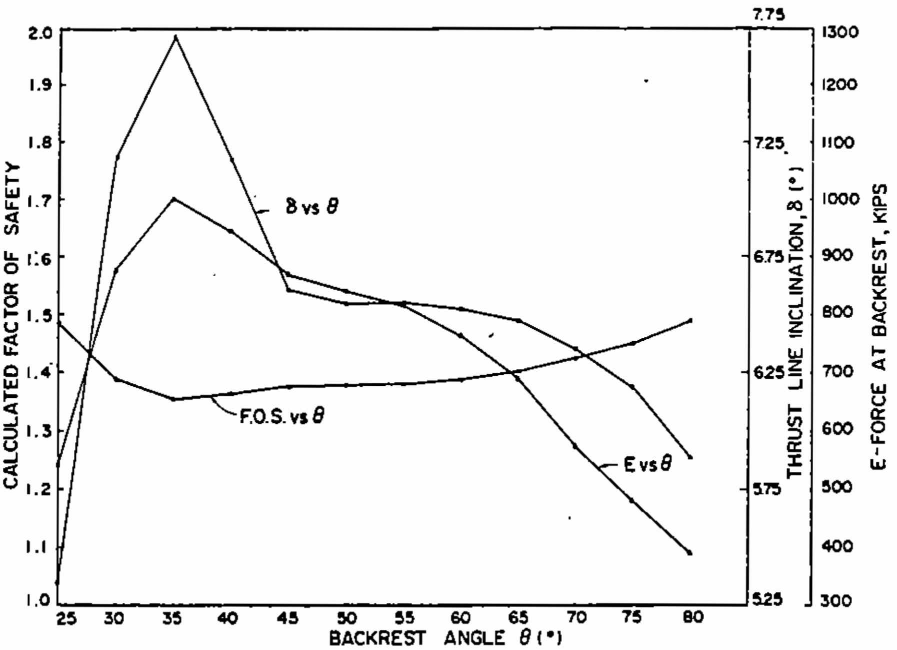

Fig. 6. Parametric study results, slide surface No. 1.

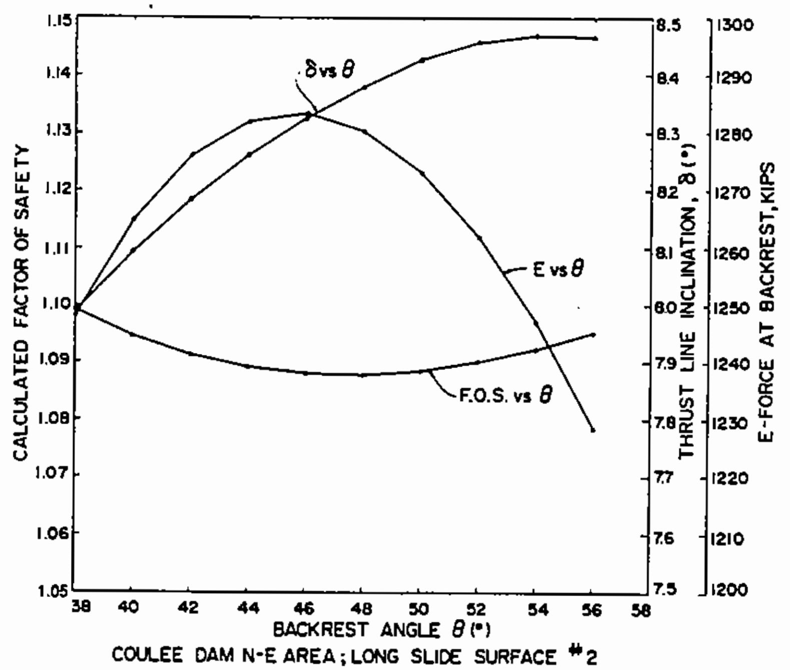

Fig. 7. Parametric study results, slide surface No. 2.

pression to be positive, the calculated negative interslice forces imply the presence of tensile normal stresses in the soil mass. Unless the solution scheme is formulated to account for the tensile character of the material, the calculated results for the (F, δ) pair can be quite meaningless. Similar comments apply for the solutions that give thrust line inclination δ in violation of the limits imposed by eqns (19) or (22) for the ideal material assumed. Even in actual geologic formations, the limits imposed by these equations cannot be grossly violated by a nonlinear solution and still be acceptable.

A poor mathematical solution does not necessarily imply a poor nonlinear solution procedure; it can also indicate a poor physical model. An evaluation of the intermediate response of a poor mathematical solution to a nonlinear system generally reveals the bad character of

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SLOPE STABILITY ANALYSIS FOR EARTHQUAKES

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SUMMARY

The method of slices commonly used for estimating static stability of natural slopes and embankments is extended to include the dynamic effects due to earthquake loading. The equation of motion of a typical slice along a path different from its base is derived. The calculations of displacement of a slide mass using the Newmark procedure are discussed and illustrated with an example problem.

INTRODUCTION

Slope stability is a subject of great practical significance in geotechnical engineering. Presently, the method of limit equilibrium satisfying all equations of static equilibrium is generally used in estimating the stability status of natural slopes and embankment structures along potential slide surfaces. The use of numerical solution procedures which account for the deformation properties of the materials is quite limited for slope stability problems because of uncertainties in the material properties determination under all stress and boundary conditions encountered in real structures made of soils and also for the lack of a generalized soil model which can realistically consider load deformation properties of soils.

For static slope stability analysis, the method of slices as proposed by Spencer (see Spencer[1] and Wright[2,3]) is commonly used. This procedure assumes the location of normal forces on the base of each slice to be at its midpoint and the resultant interslice forces to be parallel. These $(2n-2)$ assumptions reduce the total number of unknowns from $(5n-2)$ to $3n$, where n is the number of slices used to discretize the slide mass, thereby achieving statical determinancy. The solution to the slope stability equations is obtained in an iterative manner, starting with an assumed value for the factor of safety, F, and thrust line inclination, δ, satisfying the boundary conditions at the toe and head of a slide mass.[4]

For dynamic slope stability analysis, the Newmark method[5] is usually used in estimating the magnitudes of displacement along potential slide surfaces. While this procedure has been available and used for some time, there appear to be variations in its use in practice.[6–9] This difference in interpretation and use of the method leads to some difference in calculated displacement response, though the order of magnitude of results does not necessarily change.

The essential link between static and dynamic slope stability analysis is the determination of yield acceleration. The yield acceleration is defined as the threshold average acceleration for a slide mass above which permanent deformations occur.

The objectives of this paper are:

1. to extend the equations of slope stability analysis by the method of slices to calculate the yield acceleration;

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2. to develop the equation of motion necessary to estimate the displacement along a potential shear surface of any geometric shape; and
3. to illustrate by a sample problem the use of the numerical integration for calculating the displacement.

The developments reported in this paper are for two-dimensional slope stability problems.

SLOPE STABILITY EQUATIONS

Figure 1 is the general description of a slope stability problem. For any vertical slice, abcd, the forces acting are shown in Figure 1(b). H_L and H_R are the hydrostatic forces exerted by the subsurface water on the vertical boundaries of the slice (assumed to be known) and $F_e = \lambda W$ is a force, which corresponds to a constant acceleration λ times that of gravity, acting at an inclination γ to the horizontal and through the centre of mass of the slice. Other forces acting on the free body diagram of a slice are defined in Appendix I. Considering static equilibrium

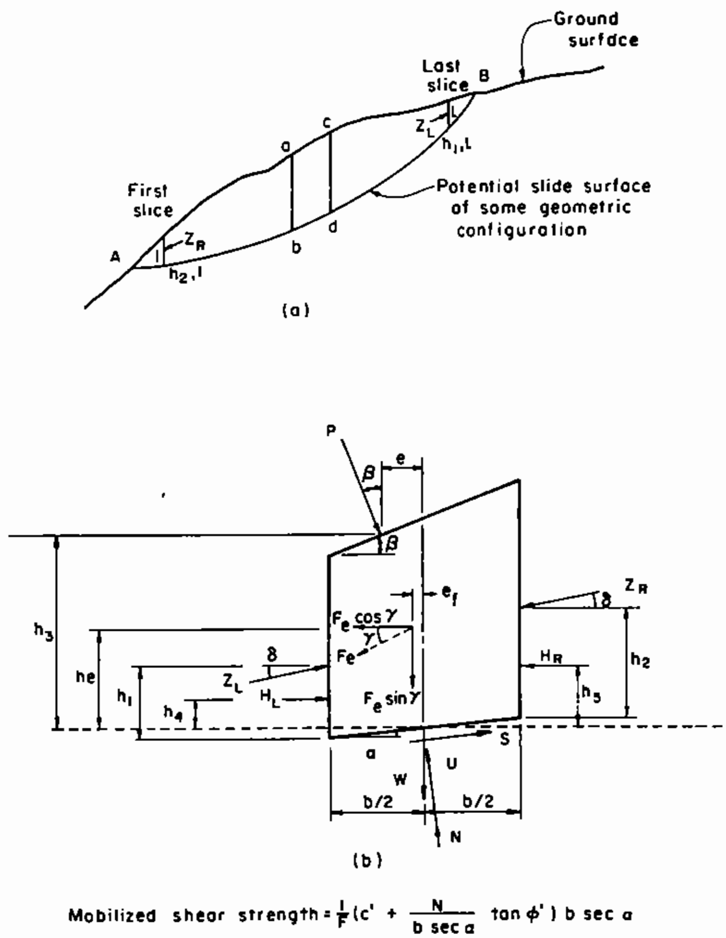

Figure 1. (a) General slope stability problem description. (b) Forces acting on a typical slice

of the forces:

$$Z_R = Z_L + \frac{\frac{1}{F}c'b \sec\alpha - W\sin\alpha + \frac{1}{F}(w\cos\alpha - U)\tan\phi'}{\cos(\delta-\alpha)\left[1-\frac{1}{F}\tan(\delta-\alpha)\tan\phi'\right]}$$

$$+\frac{P\cos(\beta-\alpha)\left[\tan(\beta-\alpha)+\frac{1}{F}\tan\phi'\right]}{\cos(\delta-\alpha)\left[1-\frac{1}{F}\tan(\delta-\alpha)\tan\phi'\right]}$$

$$+\frac{H_L\cos\alpha\left[1+\frac{1}{F}\tan\alpha\tan\phi'\right]}{\cos(\delta-\alpha)\left[1-\frac{1}{F}\tan(\delta-\alpha)\tan\phi'\right]}$$

$$-\frac{H_R\cos\alpha\left[1+\frac{1}{F}\tan\alpha\tan\phi'\right]}{\cos(\delta-\alpha)\left[1-\frac{1}{F}\tan(\delta-\alpha)\tan\phi'\right]}$$

$$-\frac{F_e\cos(\alpha-\gamma)\left[1+\frac{1}{F}\tan(\alpha-\gamma)\tan\phi'\right]}{\cos(\delta-\alpha)\left[1-\frac{1}{F}\tan(\delta-\alpha)\tan\phi'\right]} \tag{1}$$

$$h_2 = \frac{Z_L}{Z_R}h_1 + \frac{b}{2}[\tan\delta - \tan\alpha]\left[1+\frac{Z_L}{Z_R}\right] + \frac{P}{Z_R}\sec\delta\cos\beta[h_3\tan\beta - e]$$

$$+\frac{1}{Z_R}\sec\delta[H_L h_4 H_R h_5] - \frac{F_e}{Z_R}\sec\delta\cos\gamma[h_e + e_f\tan\gamma] \tag{2}$$

See Appendix I for meaning of symbols.

Equation (1) combines the summation of forces along and normal to the base of the slice for force equilibrium, and equation (2) is obtained by taking moments of all the forces acting on the slice about the centre of the base of the slice for moment equilibrium. In the derivation of equations (1) and (2) and elsewhere in this paper, the factor of safety, F, is defined as the ratio of the total shear resistance available on the slip surface to the total shear force required to reach a condition of limiting equilibrium.

The formulation of equations (1) and (2) is in terms of recursive relationships. The boundary conditions for the slope stability problem are defined in terms of Z_L, h_1, for the first slice (toe) and Z_R, h_2, for the last slice (head). The unknowns in equations (1) and (2) are Z_L, Z_R, h_1, h_2, λ, γ, δ and F. For a preselected value of λ, γ, assuming a value for the solution parameters (F, δ) and using the boundary conditions for slice 1, i.e. Z_L, h_1; equations (1) and (2) can be used in a recursive manner to evaluate Z_R and h_2 for the last slice. By suitably adjusting the (F, δ) pair values until the calculated values of Z_R and h_2 for the last slice agree with the known boundary conditions, one can determine the solution to the slope stability problem.

The formulation of slope stability problem in terms of equations (1) and (2) offers the advantage of dealing directly with the boundary parameters. The criterion for numerical convergence of a solution scheme is made in easily recognizable terms. The numerical value of Z_R and h_2 for the last slice, if in the directions shown in Figure 1(b), could be interpreted as a force exerted by water and its location, respectively, if there were to exist a vertical crack at the head of the slide mass.

The solution to slope stability equations (1) and (2) proceeds along the following steps.

1. Assign a value to γ.
2. Assign a value to λ.
3. Assume some non-zero value for the factor of safety, F, and the thrust line inclination, δ. (F, δ) are the solution parameters for the problem.
4. Knowing the boundary conditions on the left face of slice 1, i.e. $Z_{L,1}$ and $h_{1,1}$, calculate the $Z_R|_1$ and $h_2|_1$ for the right face of slice 1. Owing to material continuity between slices along vertical boundaries, $Z_R|_1 = Z_L|_2$ and $h_2|_1 = h_1|_2$. Thus, one can calculate $Z_R|_i$ and $h_2|_i$ for $i = 1$ to n where n is the number of slices.
5. Compare $Z_R|_n$ and $h_2|_n$ against the known boundary conditions $Z_{R,n}$, $h_{2,n}$. The difference between the calculated values $Z_R|_n$, $h_2|_n$ and the known values $Z_{R,n}$, $h_{2,n}$ is due to error in values F, δ assumed in Step 3.
6. By carefully adjusting the values of F, δ, Steps 4 and 5 are repeated until the calculated value of Z_R and h_2 for the last slice agree with the known boundary condition values to the desired accuracy. The iterative refinements of the guessed solution are made using non-linear equation solution schemes.[3,10]* Alternatively, equations (1) and (2) could be solved separately numerically and the final solution obtained graphically.[4,11]
7. The refined value of (F, δ), for which the boundary conditions are satisfied, constitutes a potential solution to the slope stability problem corresponding to the assigned value λ in Step 2 and γ in Step 1. The potential solution becomes an acceptable solution if for the (F, δ) the calculated response for each slice in terms of normal and shear stress on the base of the slice, interslice force, and its location are reasonable.
8. Assign a different value to λ and repeat Steps 2 through 7. The value of λ for which the calculated value of factor of safety equals 1 is by definition the coefficient of yield acceleration $\lambda_{yield}|_i$ corresponding to inclination $\gamma|_i$.

By varying γ, and repeating Steps 1 through 8, one can calculate λ_{yield} as a function of γ. Calculations for solution pair (F, δ) with $\lambda = 0$ correspond to the static case and those with $\lambda > 0$ correspond to the dynamic case with λW representing resistance as a generalized force. The material strength properties and pore water pressure relatable to the dynamic situation need to be used in calculating $\lambda_{yield}|_i$ corresponding to inclination $\gamma|_i$.

EQUATION OF MOTION

The equation of plane motion for a typical slice, Figure 2, along a path different from the base of the slice, can be developed conveniently using D'Alembert's principle of dynamic equilibrium.[12] According to this method, an additional imaginary force, the inertia force, equal to the product of the mass of the slice, M, and acceleration of the mass, $\ddot{X}$, is applied in the

* It is quite possible that the solution to the non-linear system of equations (1) and (2) may not yield reasonable response for the slices used to discretize the slide mass. For such cases, the use of numerical–graphical procedures[4] may be necessary to seek other possible solutions. The criteria of acceptability of a numerical solution to a slope stability problem are discussed in Reference 4.

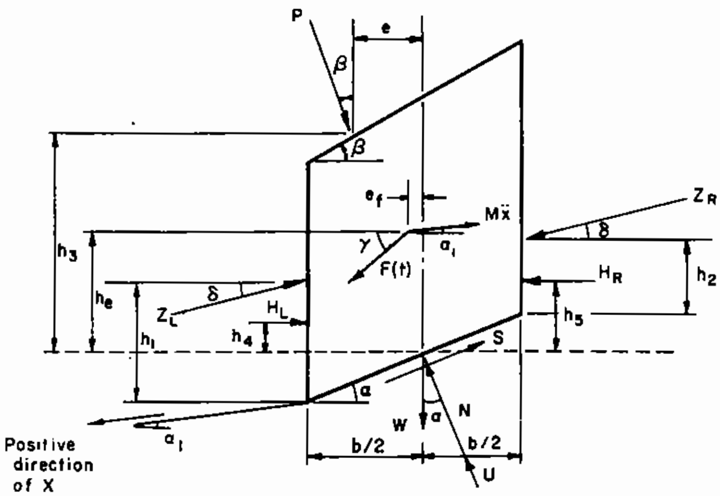

Figure 2. Dynamic equilibrium of a typical slice

direction opposite to that of positive displacement, see Figure 2. Having added this force, the situation in Figure 2 is treated exactly as a problem in static equilibrium and leads to the following development.

Summing forces along the base of the slice gives

$$c'b \sec\alpha + N \tan\phi' - W \sin\alpha + H_L \cos\alpha + Z_L \cos(\delta-\alpha) + P \sin(\beta-\alpha) - F(t)\cos(\gamma-\alpha) + M\ddot{X}\cos(\alpha-\alpha_1) - Z_R\cos(\delta-\alpha) - H_R\cos\alpha = 0 \quad (3)$$

Summing forces normal to the base of the slice gives

$$N + U - W\cos\alpha - H_L\sin\alpha + Z_L\sin(\delta-\alpha) - P\cos(\beta-\alpha) - F(t)\sin(\gamma-\alpha) - Z_R\sin(\delta-\alpha) + H_R\sin\alpha - M\ddot{X}\sin(\alpha-\alpha_1) = 0 \quad (4)$$

Eliminating N from equations (3) and (4) and rearranging terms gives

$$M\ddot{X}\cos(\alpha-\alpha_1-\phi') = [U\sin\phi' + (H_R-H_L)\cos(\alpha-\phi') + (Z_R-Z_L)\cos(\delta+\phi'-\alpha) - P\sin(\beta+\phi'-\alpha) + W\sin(\alpha-\phi') - c'b\sec\alpha\cos\phi'] + F(t)\cos(\gamma-\alpha+\phi') \quad (5)$$

At the instant of yield, $F=1$ and the corresponding $F_e = F_e|_{\text{yield}}$, and equation (1) after rearrangement of terms becomes

$$F_e|_{\text{yield}} = \frac{-1}{\cos(\alpha-\gamma-\phi')}[U\sin\phi' + (H_R-H_L)\cos(\alpha-\phi') + (Z_R-Z_L)\cos(\delta+\phi'-\alpha) - P\sin(\beta+\phi'-\alpha) + W\sin(\alpha-\phi') - c'b\sec\alpha\cos\phi'] \quad (6)$$

Substituting equation (6) into equation (5) gives

$$M\ddot{X} = \frac{\cos(\gamma-\alpha+\phi')}{\cos(\alpha_1-\alpha+\phi')}[F(t) - F_e|_{\text{yield}}] \quad (7)$$

the physical model. A designer should take into consideration both of the above possibilities in interpreting the computed response of a slope stability problem.

It is assumed, in these comments, that there is only one real value of F and of δ that will satisfy both the force and moment conditions of equilibrium.

SAMPLE PROBLEM

Figure 5 illustrates a section located in the Coulee Dam northeast area downstream of the Grand Coulee Dam in the State of Washington. The identification of potential slide surfaces in the hillside is of interest. For slope stability analysis, the geologic makeup of the site is assumed to be composed of four materials. Their broad identification and estimated properties are given in Fig. 5. The pore water pressure distribution in the hillside is shown in Fig. 5. The geometry of the segmented slide surfaces analyzed are also marked in this figure. The analyses were performed using the computer code STABLTY[15] available at the U.S. Bureau of Reclamation, Engineering and Research Center. This computer program implements the method of slices satisfying the three equations of static equilibrium and calculates a constant value for the factor of safety and a constant inclination for the interslice forces. The effect of backrest angle on the calculated factor of safety, inclination of the interslice force, and the horizontal component of the interslice force are shown in Figs. 6 and 7. It should perhaps be mentioned that each potential slide surface was analyzed individually. For the slide surface No. 2, the calculated least factor of safety corresponds to backrest angle of 48°, Fig. 7. The critical backrest angles for the surface topography and the two material strength values, from Fig. 3, are: $\theta = 35°$ for $\phi = 11°$ and $\theta = 58°$ for $\phi = 34°$. The average of these two critical backrest angles is 46.5°. The corresponding values for the critical backrest angle assuming validity of $45 + \phi/2$ would be 50.5° and 62° with the mean value of 56.25°. Thus, the backrest angle corresponding to the least factor of safety tends to agree with the values indicated by Fig. 3 rather than $45 + \phi/2$ [2].

It should be mentioned that these analyses were performed using the existing conditions of surface topography, interpreted material horizons, tested material properties for predominantly clay materials (residual strength) and estimated material properties for predominantly non-clay materials, and interpreted pore water pressure distribution for the steady-state river operation (tailbay elevation 955.0 ft). The residual strength value for clay materials used in these calculations tends to align with the lower end of the range of strength values obtained to date by both the back calculations of known past slides in the area and laboratory tests. The results of calculated factor of safety as far as they apply to the specific site are preliminary and do not reflect the remedial treatment alternatives under study to imporve stability.

SUMMARY

The selection of potential slide surface geometry in slope stability analysis by the limit equilibrium method deserves a careful consideration. A close scrutiny of the calculated results of slope stability analysis in terms of interslice forces—their magnitude, direction, and location is of significant importance and should be considered along with the factor of safety.

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Expressing $F(t)$ and $F_e|_{yield}$ in terms of a coefficient multiplied by the acceleration due to gravity and multiplied by mass of slice, equation (7) can be rewritten as

$$\ddot{X} = \frac{\cos(\gamma - \alpha + \phi')}{\cos(\alpha - \alpha_1 - \phi')}[\lambda_{\text{earthquake}} - \lambda_{\text{yield}}]g \tag{8}$$

This is the equation of motion of the slice along a plane inclined at an angle α_1 to the horizontal, Figure 2. For $\alpha = \alpha_1$, Figure 2 corresponds to the condition of a block sliding on an inclined plane with angle of inclination of α, and equation (8) becomes[7]

$$\ddot{X}_{\text{along the base of slice}} = \frac{\cos(\gamma - \alpha + \phi')}{\cos\phi'}[\lambda_{\text{earthquake}} - \lambda_{\text{yield}}]g\ . \tag{9}$$

DISPLACEMENT CALCULATIONS

In a segmented shear surface geometry, movement of the slide mass is not along a single inclined plane but instead the movement takes place along planes of different inclination, see Figure 3. For the slide mass to stay together (no shear failure along interslice boundaries), it shall also experience a rotational displacement. Thus, the slope of the slide mass begins to change from the instant relative motion begins. These features of displacement of necessity require that there exists a zone rather than a plane along which movement occurs, if there were to exist a material contact across shearing boundaries. For this type of a displacement response, the rear portion of the slide mass shall slide on the shear surface as the slide mass rotates and translates outwards in any two consecutive configurations, Figure 3.

The lateral displacement of a point in the slide mass near the shear surface shall be greater than that observed on the exposed surface. The vertical displacement of a point in the slide

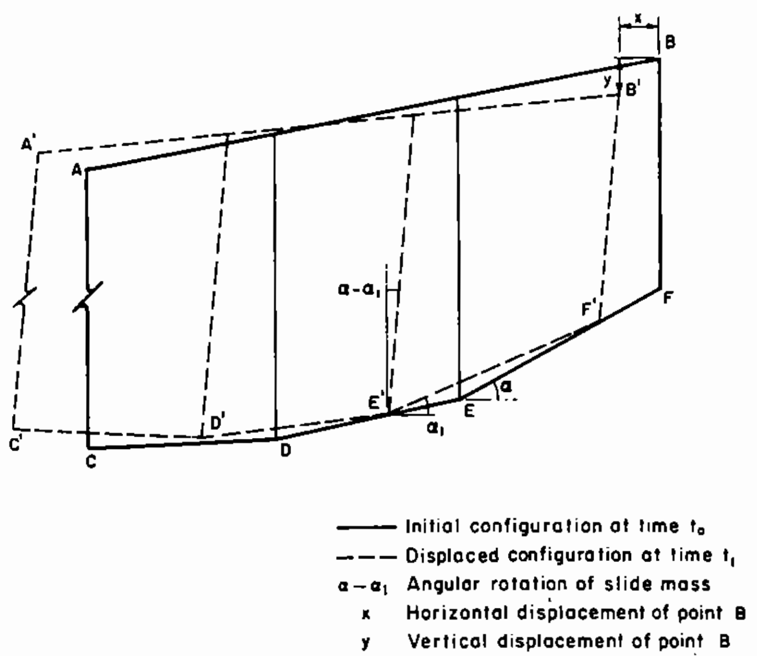


Figure 3. Rigid body displacement of a slide mass (hypothesis)

mass near the shear surface shall be less than that observed on the exposed surface. The displaced configuration of any two points in the slide mass is related through the rotational displacement of the slide mass. Thus, for calculating displacements of a slide mass during an earthquake, it is possible to work with any one wedge. Since one of the items of interest in predicting the behaviour of an embankment for a design earthquake is to calculate the vertical displacement of the crest of the dam, it may be desirable to select the wedge, in a shear surface geometry, next to the crest of the dam.

Equation (8) can be used to calculate the type of displacement discussed above using the Newmark method.[5] In this equation α and α_1 can be interpreted as angles of inclination of two planes occupied by a slice in two consecutive time steps—α corresponding to time t_0 and α_1 corresponding to time t_1. By making $(t_1 - t_0)$ small, one can essentially trace the geometric configuration of a slide mass during a design earthquake.

The calculations for estimating displacements of a potential slide mass may proceed along the following steps.

1. Calculate the yield acceleration by solving the slope stability equations (1) and (2) for a particular value of γ, as discussed previously in this paper. By varying γ in small increments, one can construct a table of values of λ_{yield} corresponding to each inclination γ. In these calculations, the equivalent force $F_e = \lambda W$ is applied at a desired location for each slice, i.e. centre of gravity of each slice or middle of base of each slice, etc. This location of application of equivalent force is the same for all the slices making up the slide mass.
2. For the design earthquake, calculate the time history of resultant acceleration coefficient, $\lambda_e(t)$ and its inclination $\gamma_e(t)$.
3. Corresponding to each $\gamma_e(t_i)$, select by interpolation λ_y from the table of values constructed in Step 1. Thus, one has corresponding to every time step of the earthquake record, values of γ_e, λ_e, and λ_y. Calculate $\lambda_e - \lambda_y$ for each time step and let it be designated λ_{e-y}.
4. Starting from time t_0 (beginning of an earthquake), check the sign of λ_{e-y}. For the rigid plastic system assumed in the Newmark method, relative displacement of the slide mass shall occur only when λ_{e-y} is positive. Let λ_{e-y} change sign in the time interval $t_{j-1} < t < t_j$. The time at which $\lambda_{e-y} = 0$, by linear interpolation, is

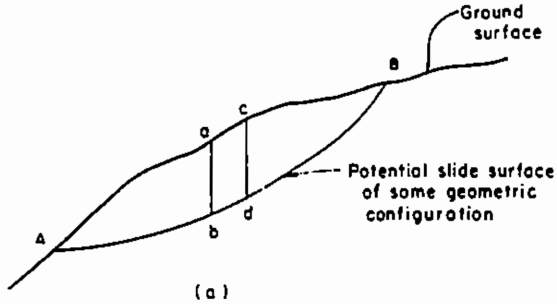
$$t_{\lambda_{e-y}=0} = t_{j-1} + \frac{\lambda_{e-y}|_{j-1}(t_j - t_{j-1})}{\lambda_{e-y}|_{j-1} - \lambda_{e-y}|_j} \tag{10}$$

At the instant of time when the earthquake acceleration exceeds the yield acceleration, the relative velocity of the slide mass begins to increase and the relative displacement of the slide mass starts.

5. Knowing the relative velocity $\dot{X}_s$ at time t_s, the relative velocity $\dot{X}_{s+1}$ at time t_{s+1} can be calculated, by the linear acceleration method,[12] as

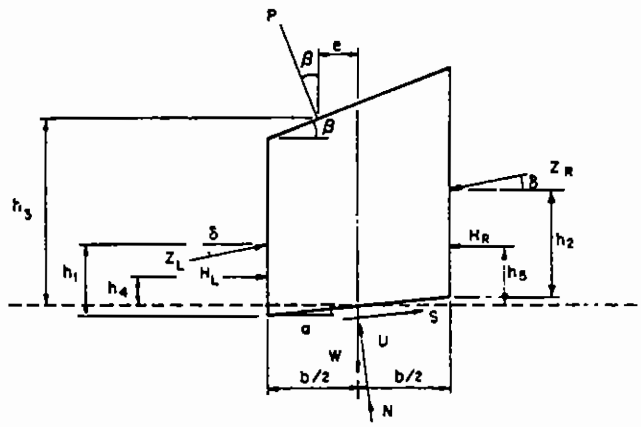
$$\dot{X}_{s+1} = \dot{X}_s + \frac{\ddot{X}_s + \ddot{X}_{s+1}}{2}(t_{s+1} - t_s) \tag{11}$$

The $\dot{X}_s$ corresponding to equation (10) shall be zero. Starting from the time when λ_{e-y} equals zero, calculate the relative velocity of the slide mass using equation (11) until the time step when $\dot{X} \leq 0$. In these calculations $\ddot{X}$ is obtained from equation (8) or (9), depending upon the geometry of the shear surface. This implies that deceleration, past the maximum value of relative velocity of the sliding mass, varies according to the time history of the earthquake acceleration. Displacement continues until the relative velocity becomes zero. The time at which $\dot{X}$ becomes zero can be calculated by quadratic interpolation. For linear variation of



(a)

Fig. 1(a). General slope stability problem description.



(b)

Available shear strength $= \frac{1}{F}\left(c' + \frac{N}{b \sec \alpha} \tan \phi'\right) b \sec \alpha$

Fig. 1(b). Forces acting on a typical slice.

clature. Considering static equilibrium of the forces:

$$Z_R = Z_L + \frac{\frac{1}{F} c'b \sec \alpha - W \sin \alpha + \frac{1}{F}(W \cos \alpha - U) \tan \phi'}{\cos(\delta - \alpha)\left[1 - \frac{1}{F} \tan(\delta - \alpha) \tan \phi'\right]}$$

$$+ \frac{P \cos(\beta - \alpha)\left[\tan(\beta - \alpha) + \frac{1}{F} \tan \phi'\right]}{\cos(\delta - \alpha)\left[1 - \frac{1}{F} \tan(\delta - \alpha) \tan \phi'\right]}$$

$$+ \frac{H_L \cos \alpha \left[1 + \frac{1}{F} \tan \alpha \tan \phi'\right]}{\cos(\delta - \alpha)\left[1 - \frac{1}{F} \tan(\delta - \alpha) \tan \phi'\right]}$$

$$- \frac{H_R \cos \alpha \left[1 + \frac{1}{F} \tan \alpha \tan \phi'\right]}{\cos(\delta - \alpha)\left[1 - \frac{1}{F} \tan(\delta - \alpha) \tan \phi'\right]} \tag{1}$$

$$h_2 = \frac{Z_L}{Z_R} h_1 + \frac{b}{2}\{\tan \delta - \tan \alpha\}\left[1 + \frac{Z_L}{Z_R}\right]$$

$$+ \frac{P}{Z_R} \cos \beta \sec \delta [h_3 \tan \beta - e]$$

$$+ \frac{1}{Z_R} \sec \delta [H_L h_4 - H_R h_5] \tag{2}$$

In the derivation of eqns (1) and (2), and elsewhere in this paper, the factor of safety (F) is defined as the ratio of the total shear strength available on the slip surface to the total shear force required to reach a condition of limiting equilibrium.

GEOMETRIC CONFIGURATION

For circular configuration of potential slide surfaces, most computer programs have a routine that optimizes on a circle that gives a minimum factor of safety[3, 15]. This optimization is generally in the neighborhood of the initial estimate for the center of rotation of critical circle provided by a designer.

For a segmented geometry of potential slide surfaces, generally a discrete calculation is performed for the stability determination along the specified configuration. The critical elements of a slope that should be considered in the selection of a segmented failure surface geometry are:

1. The profile of the weak material responsible for the occurrence of slope stability problem.
2. The profiles of the underlying and overlying relatively stronger material.
3. The pore water pressure distribution.
4. The length of the potential slide surface.
5. The indication of localized weak material zones.

The order in which the above elements are mentioned acceleration, the velocity expression is: $\dot{X} = pt^2 + qt + r$. The time at which $\dot{X}$ becomes zero can be calculated from the formula

$$t_{\dot{X}=0} = \frac{-q \pm \sqrt{(q^2 - 4pr)}}{2p} \tag{12}$$

The coefficients p, q, r are evaluated using the three consecutive computed values of the velocity and the corresponding time, i.e. $\dot{X}_{k-1}$, $\dot{X}_k$, $\dot{X}_{k+1}$; t_{k-1}, t_k, t_{k+1} when $\dot{X}_k > 0 > \dot{X}_{k+1}$ corresponding to time $t_k < t_{\dot{X}=0} < t_{k+1}$.

It is the interval from the time when the earthquake acceleration exceeds the yield acceleration to the time when the relative velocity drops to zero during which relative displacement of the slide mass takes place. There may exist quiet intervals of time during the earthquake when the contributions to the relative displacement along the shear surface are zero.

6. The displacement of the slide mass, between the time when λ_e exceeds λ_y to the time when $\dot{X}$ drops to zero, can be calculated from the formula, based on the linear acceleration method[12]

$$X_{s+1} = X_s + \dot{X}_s(t_{s+1} - t_s) + \frac{2\ddot{X}_s + \ddot{X}_{s+1}}{6}(t_{s+1} - t_s)^2 \tag{13}$$

Comments

The displacement calculation procedure can be applied to any point in any wedge used in the discretization of the slide mass. The calculated displacement X shall be translation of the point, along the planar surface having an inclination α_1 with the horizontal, and the angular rotation shall be $(\alpha - \alpha_1)$. The displacement of any other point on the slide mass can then be computed by geometry.

The displacement calculation procedure discussed in the previous section is essentially an integration of the differential equation of motion and uses the input data for yield acceleration of the slide mass and the earthquake acceleration record. It does not consider any of the earthquake-related effects on material strengths and pore fluids. It was mentioned that during an earthquake relative displacement along a shear surface may occur. These earthquake-related effects alter the yield acceleration of the slide mass and the initial geometry of the slide mass. The calculated values of displacement shall be reasonably acceptable to the extent that the values of the yield acceleration based on the geometry, strength and pore fluids pressure used remain valid. If the yield acceleration values could be updated for every geometric configuration of the slide mass and if the corresponding earthquake acceleration be obtained, the procedure does not impose any restriction on its validity based on magnitude of displacement.

One possible procedure for accounting for the effects of changing geometry, material strength and pore fluid pressure during relative displacement of the slide mass is to calculate the yield acceleration *vs.* inclination of the unstabilizing force for a range of values of material strength, pore fluid pressure and geometry, and then during numerical integration of the differential equation of motion use the appropriate yield acceleration *vs.* inclination table depending upon the magnitude of displacement calculated. Similarly the earthquake acceleration records, if determinable, for the conditions anticipated (or conditions determined by a simple analysis in which the earthquake induced effects are ignored) can be specified and used in the calculations of displacement.

For a segmented shear surface geometry, equation (8) offers a means of refining the calculation procedure. The calculation procedure may be initiated using equation (9) and the displacement increment obtained during an interval when the earthquake acceleration exceeds the yield acceleration to the time when relative velocity drops to zero. This estimate of displacement increment can be refined using equation (8) and controlling the rigid body rotation $(\alpha - \alpha_1)$ permitted in one step of calculation of displacement. α and α_1 are the angles of inclination of the base of a wedge in two consecutive geometric positions.

Unlike the method given in Reference 13, the procedure for the calculation of yield acceleration of a slide mass presented in this paper is an extension of the method of slices used for the static slope stability analysis; it satisfies all the requirements of statics and acceptability criteria[4] for the validity of the calculated solution. The displacement calculation procedure presented uses a linear acceleration method[12] and time history of earthquake acceleration in numerical double integration of the differential equation of motion. Considering the small magnitude of the time step for digitized design earthquake record, the linear variation of acceleration between time stations is an acceptable engineering assumption.

Sample problem

Figure 4 represents a typical cross section of an embankment dam. The material properties and the potential failure surfaces, selected for illustration, are shown in this figure.

The yield acceleration for the shear surfaces is calculated by applying the equivalent force λW_i at the midpoint of the base of each slice. There are 18 slices used to discretize the slide mass for each shear surface geometry. The results of these calculations are shown in Figure 5.

The earthquake acceleration record, available in digital form, used to calculate the displacement is shown in Figure 6. In order to illustrate the displacement calculation procedure, the earthquake time interval $5{\cdot}0 \leq t \leq 5{\cdot}15$ was selected arbitrarily. The longhand calculations for the numerical integration of the differential equation of motion, equation (9), are shown in a tabular form in Figure 7 for the shear surface 1. Both components of the earthquake acceleration are used in these calculations. Common multiplication factors for every item of calculations are listed in the column labelled as 'FACTORS', Figure 7. Thus the increment of displacement during this time interval is $1{\cdot}3420g(\Delta t^2/\cos\phi')$. Using $g = 32{\cdot}2$ ft/s per s, $\Delta t = 0{\cdot}01$ s and $\phi' = 45°$, the increment of displacement comes to $6{\cdot}11 \times 10^{-3}$ ft.

The calculation scheme discussed in this paper was implemented in a computer program 'DISP'. A listing of the program in FORTRAN IV and the user's instructions can be obtained from the author on request.

The total displacement along the base of the slide mass, for the shear surface 1, was calculated considering (1) both components of earthquake acceleration and (2) only the horizontal component of earthquake acceleration. The calculations were performed for four possible combinations of positive horizontal and vertical earthquake acceleration record for case (1) and for two possible positive horizontal earthquake acceleration records for case (2). Linear interpolation was used in calculating the time at which $\dot{X}$ becomes zero. The maximum calculated displacements of the slide mass along the base, for the two cases, were 0·06 ft and 0·05 ft, respectively for the shear surface 1. The maximum calculated displacements of the crest of the dam along the shear surface 2, for arbitrary magnifications of the design earthquake record are shown in Figure 8.

CONCLUSIONS

The equations of static slope stability using the method of slices and satisfying all equations of statics have been extended to account for the dynamic effects. The earthquake induced

CONNECTIVITY DESCRIPTION

LINE No	NODES ○	STRENGTH PROPERTIES FOR THE MATERIALS: C Lbs/sq. ft	STRENGTH PROPERTIES FOR THE MATERIALS: φ°	MATERIAL DENSITY Lbs/c ft as used	MATERIAL No △
1	1-2-3-4-5-6-7-7'				
		0.0	35.0	125.0	1
2	1-2-3-9-10-6-7-7'				
		0.0	45.0	125.0	2
3	1-2-8-9-10-6-7-7'				
		0.0	40.0	130.0	3
4	1-2-8-12-9-10-6-7-7'				
		1008.0	35.0	120.0	4
5	1-2-8-12-9'-10-6-7-7'				
		288.0	35.0	130.0	5
6	1-2-8-12-13-10-6-7-7'				
		0.0	45.0	135.0	6
7	1-2-8-12-13-10-11-7-7'				
		0.0	40.0	140.0	7
8	1-2-8-12-13-10-14-11-7-7'				
		288.0	35.0	135.0	8
9	1-2-8-12-13-14-11-7-7'				
		10000.0	45.0	150.0	9

PROFILE GEOMETRY

NODE ○	X(Ft)	Y(Ft)
1	0.0	0.0
2	34.0	0.0
3	1254.0	658.0
4	1267.0	665.0
5	1333.0	665.0
6	1348.7	658.0
7	2874.0	0.0
7'	3000.0	0.0
8	602.0	0.0
9	1280.0	658.0
9'	1252.0	605.0
10	1320.0	658.0
11	1772.0	0.0
12	941.0	0.0
13	1252.0	0.0
14	1388.0	0.0

PIEZOMETRIC LINE DESCRIPTION

X (Ft)	PIEZOMETRIC LINE ELEVATION (Ft)
0.0	0.0
34.0	0.0
602.0	0.0
941.0	0.0
1207.73	518.88
1252.0	605.0
1320.0	658.0
1348.7	658.0
3000.0	658.0

SURFACE LOADING

X (Ft)	Y(Ft)	LOAD (Lbs/sq ft)
1348.7	658.0	0.0
1410.0	630.8	-692.28
2874.0	0.0	41059.20
3000.0	0.0	41059.20

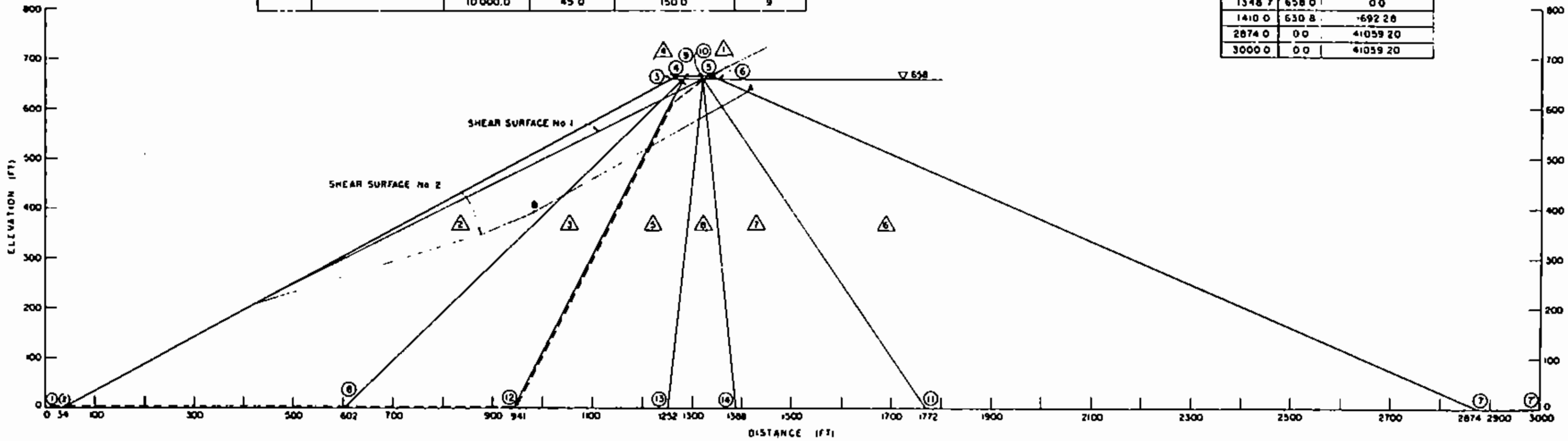


Figure 4. Sample problem

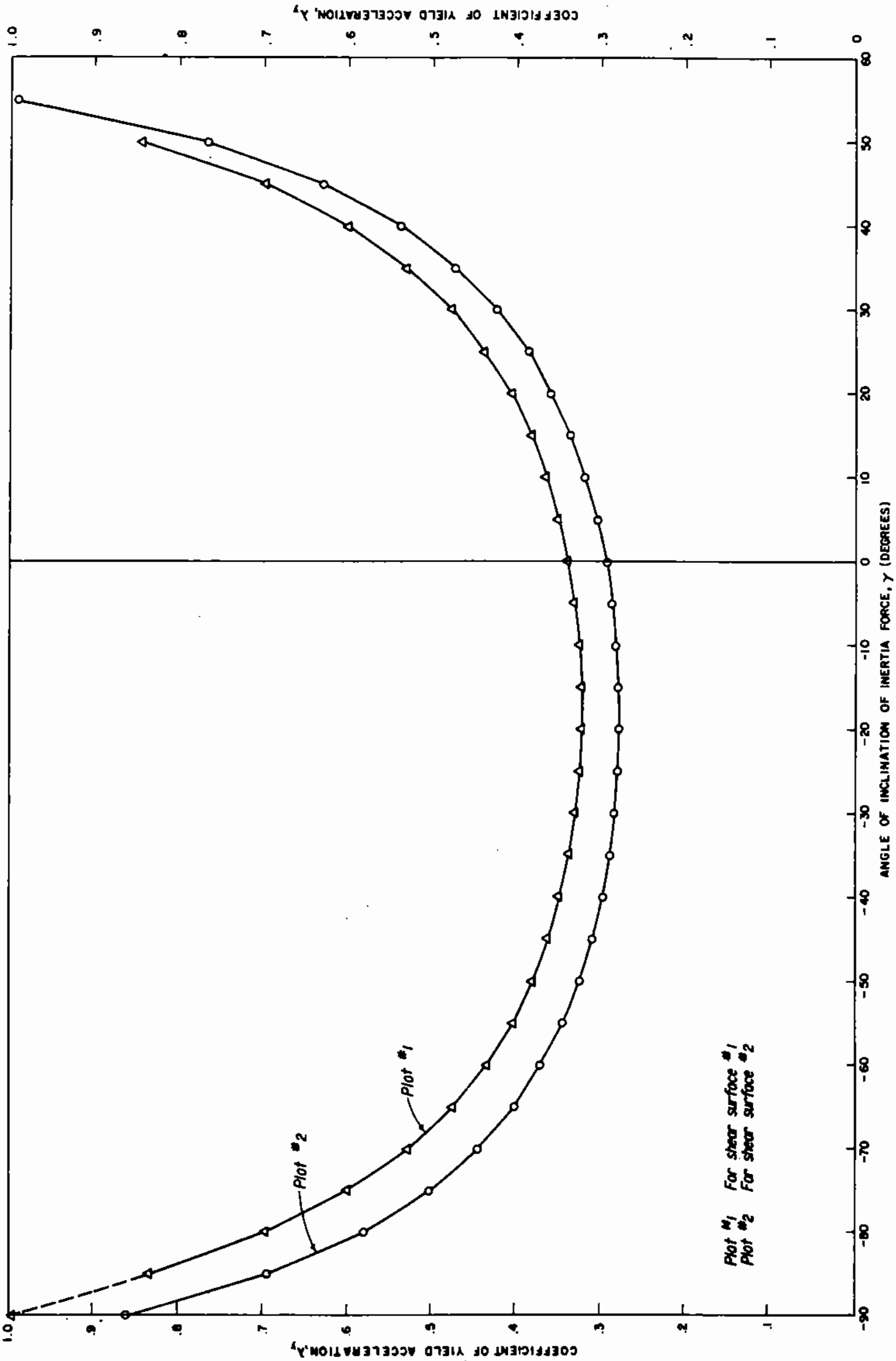


Figure 5. Yield acceleration as a function of direction of acceleration—sample problem

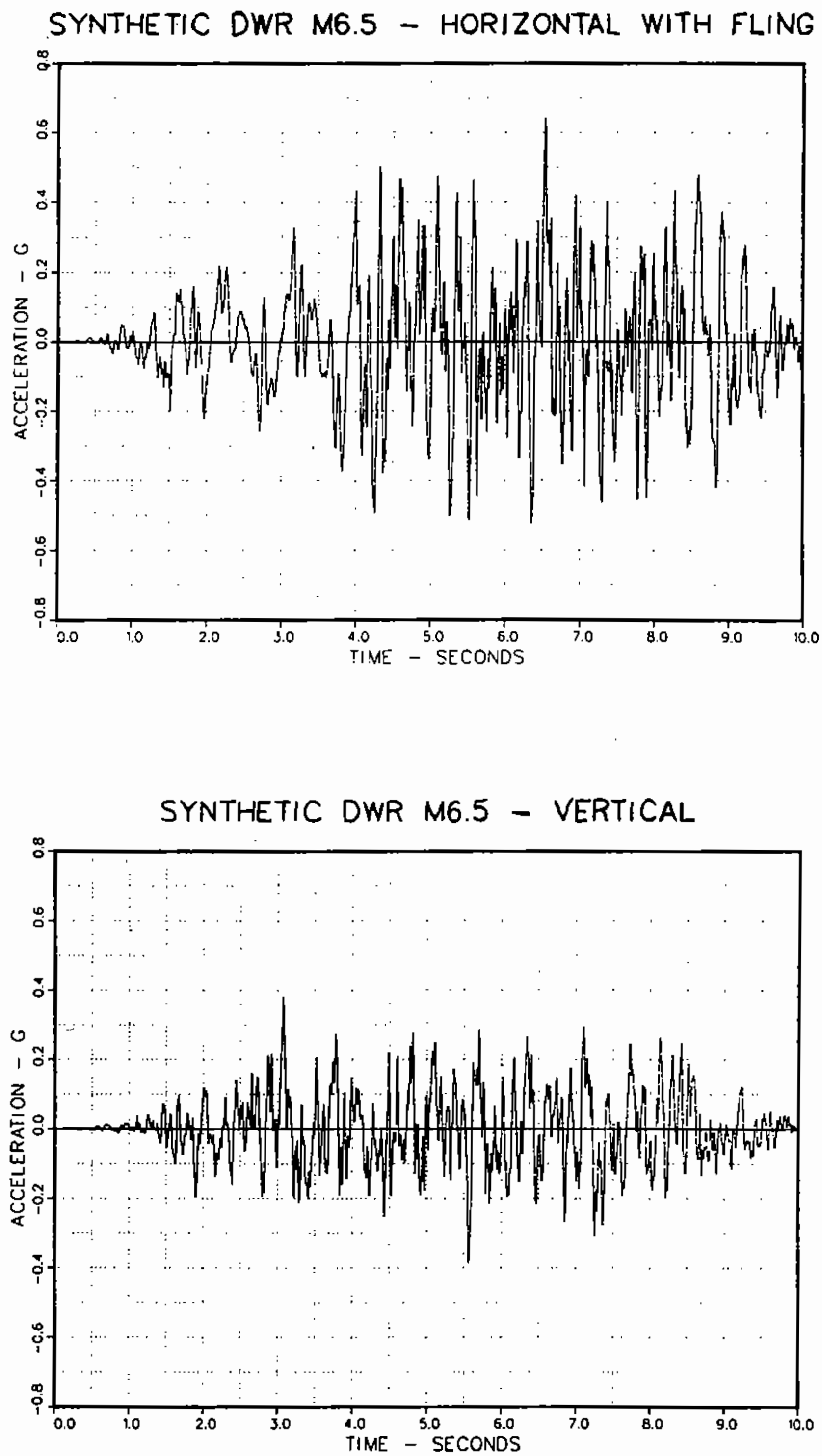


Figure 6. Design earthquake—sample problem

ITEM	DESCRIPTIONS	5.00	5.01	5.02	5.03	5.04	5.05	5.06	5.07	5.08	5.09	5.10	5.11	5.12	5.13	5.14	5.15	FACTORS	COMMENTS
A	TIME (t)	5.00	5.01	5.02	5.03	5.04	5.05	5.06	5.07	5.08	5.09	5.10	5.11	5.12	5.13	5.14	5.15		
B	INERTIA FORCE DUE TO DESIGN EARTHQUAKE	−.281	−.178	.071	.111	.115	.082	.174	.305	.461	.508	.426	.337	.221	.112	−.052	−.051	W	
C	ANGLE OF INCLINATION, γ	−159.74	177.70	59.12	−29.28	−53.32	−70.68	−75.06	−44.77	−29.25	−20.82	−27.78	−48.20	−38.67	−38.37	−91.30	141.81		
D	INERTIA FORCE FOR YIELD CORRESPONDING TO INCLINATION, γ	∞	∞	1.0	.332	.400	.540	.603	.365	.332	.326	.331	.377	.349	.348	1.0	∞	W	
E	ACCELERATION THAT CAN CAUSE DOWNWARD MOTION (B-D)								−.060	.129	.182	.095	−.040	−.128	−.236	−1.052		W	
F	ACCELERATION ALONG DIRECTION X i.e. $\ddot{X}$								−.054	.127	.182	.094	−.035	−.120	−.222	−.309	∞	$g/\cos\phi'$	ALONG THE BASE OF SLICE
									$*'$ 5.07298										$*'$ – TIME FOR $\ddot{X}=0$
G	ACCELERATION ALONG DIRECTION x FOR DOWNWARD MOTION								0.0	.127	.182	.094	−.035	−.120	−.222	$*'' =$ −.2607 −.309	∞	$g/\cos\phi'$	$*''$ – VALUE FOR $\ddot{X}$ AT t WHEN $\dot{x}=0$
H	AVERAGE ACCELERATION								.04458	.1545	.138	.0295	−.0775	−.1710	−.2655			$g.\Delta t/\cos\phi'$	
I	VELOCITY $\dot{x}$								0.	.04458	.19908	.33708	.36658	.28908	.11808	−.14742		$g.\Delta t/\cos\phi'$	
															$*''' =$ 5.13445				$*'''$ – TIME FOR $\dot{x}=0$
J	VELOCITY ALONG DIRECTION x FOR DOWNWARD DISPLACEMENT								0.	.04458	.19908	.33708	.36658	.28908	.11808	0		$g.\Delta t/\cos\phi'$	
K	DISPLACEMENT INCREMENT								.01043	.11725	.27541	.36258	.33491	.21208	.02929			$g.\Delta t^2/\cos\phi'$	
L	DISPLACEMENT								0.	.01043	.12768	.40309	.76567	1.10058	1.31266	1.3420		$g.\Delta t^2/\cos\phi'$	CONTRIBUTION TO DISPLACEMENT FOR $5.0 \le t \le 5.15$

Figure 7. Details of numerical integration—sample problem

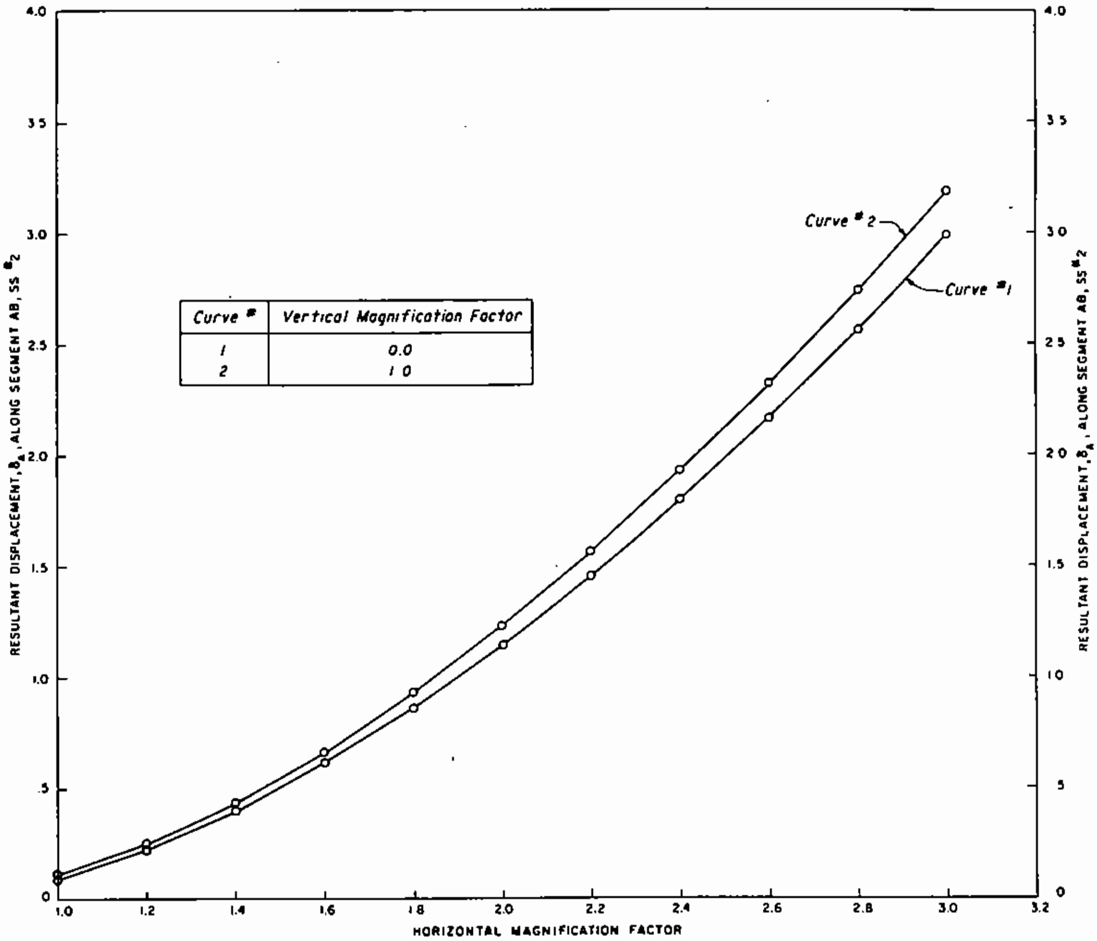


Figure 8. Displacement response—sample problem

displacements of a potential slide mass along a segmented failure surface can be estimated using the Newmark method applied to any point in any wedge used to discretize the slide mass. As asserted by Newmark,[5] the calculation procedure permits a quick but only approximate estimate of the displacements caused by earthquake loadings. While this may offer an upperbound solution on displacement, more refined solution procedures, which consider the deformation characteristics of the material, if reasonable material and loading data could be available for use, should be employed for final design of projects. Possibilities of different mechanisms of failure of embankment during earthquake type of loadings should be investigated.

APPENDIX I: LIST OF SYMBOLS

b	Width of slice
c'	Cohesion with respect to effective stress
e	Eccentricity of external force P

e_t	Eccentricity of inertia force
F	Factor of safety
F_e	Inertia force
$F(t)$	Force as a function of time
H	Force exerted by the pore water on the interslice boundary
h	Height of force above slip surface
$h_{1,1}$	Height of boundary force at toe of slide mass
$h_{2,1}$	Height of boundary force at head of slide mass
$h_1\|_j$	Location of interslice force on the left face of slice j
$h_2\|_i$	Calculated location of interslice force on the right face of slice i
n	Number of slices
P	External force acting on the slice
t_j	Time step j
U	Force exerted by the pore water on the base of a slice
W	Weight of slice
X	Displacement
$\dot{X}$	Velocity $= dX/dt$
$\ddot{X}$	Acceleration $= d^2X/dt^2$
Z	Interslice force
$Z_{L,1}$	Force boundary condition at toe of slide mass
$Z_{R,1}$	Force boundary condition at head of slide mass
$Z_L\|_j$	Interslice force on the left face of slice j
$Z_R\|_i$	Calculated interslice force on the right face of slice i
α	Angle of inclination of base of slice
α_1	Angle of inclination of the plane along which displacement of base of slice occurs
β	Angle of inclination of top of slice
δ	Angle of inclination of interslice force
γ	Angle of inclination of inertia force
λ	Coefficient which when multiplied with acceleration due to gravity gives the desired acceleration
λ_e	Coefficient which when multiplied with acceleration due to gravity gives the resultant design earthquake acceleration
λ_y	Interpolated value of coefficient of yield acceleration for a particular inclination of the resultant design earthquake acceleration
λ_{e-y}	$\lambda_e - \lambda_y$
ϕ'	Angle of internal friction with respect to effective stress

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DYNAMIC RESPONSE ANALYSIS OF EMBANKMENT DAMS

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DYNAMIC RESPONSE ANALYSIS OF EMBANKMENT DAMS

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SUMMARY

A one-dimensional wave propagation method for earthquake response analysis of horizontally-layered sites of infinite lateral extent is adapted to account for the finite cross-sectional dimensions of an embankment dam overlying a foundation deposit which may be considered infinite in its lateral extent. The procedure is used to study the response of an existing embankment dam for an actual earthquake record. A two-dimensional dynamic finite element analysis is also performed for this case. The records of ground acceleration at an outcropping base rock and at the crest of the dam are available for the site. The comparisons of computed and observed responses support the modified use of the simple numerical procedure.

INTRODUCTION

There are several methods and related computer procedures available for studying the dynamic response of embankment dams and level ground deposits to seismic loading.[1–25*] There is a fair amount of information available in the literature which gives comparisons of results obtained by use of these procedures to their respective class of problems, i.e. References 2, 8, 10, 11, 12, and 14–16 for embankment dams, and References 22, 24 and 25 for level ground deposits. Reference 8 gives comparisons of computed results for earth dams obtained using a dynamic finite element analysis procedure with the results obtained using a simplified analysis procedure for level ground deposits. The computer program QUAD-4[3] was used to perform the two-dimensional dynamic finite element analyses of the embankment dams; the computer program SHAKE[18] was used to perform the one-dimensional analyses of the embankment dams. The work reported in this reference leads to the conclusion that for dynamic stresses a zoned embankment dam can be effectively analysed as a one-dimensional problem. The advantages of being able to do so are significant, not only with respect to cost and effort, but also with respect to the ability to analyse a particular area of interest in an embankment without having to consider the entire embankment.

The variability of materials in an embankment dam and its foundations requires rather elaborate field and laboratory testing programmes to estimate dynamic properties of the materials involved. For dynamic studies of some structures, it may be adequate to use 'typical' values of material properties available in the literature if 'sufficient' knowledge of materials involved exists. The uncertainties in the complete knowledge of in-place material behaviour, simplifying assumptions of mathematical formulations and their numerical implementations, and generally incomplete knowledge of seismic loads are, in general, the reasons for preference in the use of simplified computational procedures over the more complicated (though theoretically more correct) analysis procedures.

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*References included in this paper are representative but not a complete list of the works on the subject.

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The results of any analysis procedure need to be interpreted with 'judgement' in making design related decisions. The judgement base can be expanded by comparisons of results obtained by different analysis procedures and also by comparing them with actual observed responses of structures whenever such performance records are available. However, there is relatively limited information available on the actual instrumentally measured records of the response of real earth dams during strong or moderate size earthquake shaking.[12] These records, whenever available, are incomplete in that they do not provide adequate definition of input ground motion and dam structural response, soil and pore water pressure response, etc. Thus, there is relatively little information available on comparisons of computed responses with the observed responses of earth dams during actual earthquakes.[2,10,12,14,15]

The use of judgement becomes increasingly important when the uncertainties in the complete problem definition are relatively large, such as those encountered in existing embankment dams which were designed and built prior to the realization by the engineering profession of the importance for the safety of embankment dams of understanding earthquake effects. There are, however, a large number of such dams in service at present. For the reasons stated above, and also for objectives in studying the dynamic response of existing embankment dams, simple computational procedures are generally used. Sometimes, these simple computational procedures are also used in the analysis and design of new earth dams.[5,7,26,27] One such simple computational procedure for ground response analysis is 'SHAKE'.[18]

QUAD-4[3] is a dynamic finite element method (FEM) program which solves the equation of motion within the time domain for an assembly of elements. Through iterations, the program adjusts the damping and shear modulus with respect to the induced shear strain for each element by the equivalent linear method. This computer program has been superseded by a more efficient and effective computer program FLUSH.[4] The details of the solution technique and the formulation of the system matrices are readily available in References 4 and 9.

The theory implemented in the computer program SHAKE considers the responses associated with the vertical propagation of horizontally polarized plane shear waves through a linear viscoelastic system consisting of homogeneous and isotropic horizontal layers which extend to infinity in the horizontal directions and have a half space as the bottom layer. The effect of the finite lengths of the layers in an embankment dam overlying the essentially infinite lengths of the layers in the foundation zone on the embankment response to a seismic event can be quite significant. The influence of confining pressure on the damping ratio may be important for relatively thick deposits.[26] The conventional use of the computer program 'SHAKE' for ground response calculations does not account for either of the two characteristics of the physical problem. The objectives of this paper are to

(1) present the use of the computer program SHAKE in accounting for the finite cross-sectional dimensions of an embankment dam
(2) document the use of an empirical relation for damping ratio as a function of confining pressure
(3) compare the results of analysis of the Bradbury Dam using the computer programs FLUSH and SHAKE with and without the above two modifications for the observed time–acceleration records at the crest of the dam, and at an outcropping base rock near the outlet works during the 13 August 1978 Santa Barbara earthquake. Crest and outcropping rock time–acceleration records in two orthogonal directions (the 340° component is along the axis of the dam and the 250° component is normal to the axis of the dam) at this site are available.[28] Only transverse motion records (250° component) are applicable for this study.

A short description of the computer program SHAKE is included here for completeness sake. Similar descriptions of other computational procedures are given in Reference 22.

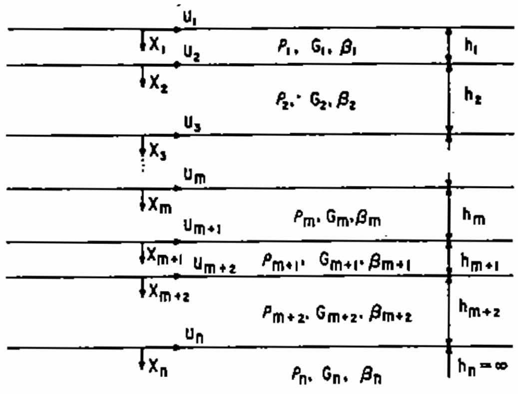


Figure 1. General horizontally layered deposit

SHAKE[18]

SHAKE uses a total stress method which treats the non-linear soil by an equivalent linear procedure. The non-linear response is approximated by a damped linear elastic model. The stress–strain properties of the soil are defined by strain dependent shear moduli and equivalent viscous damping factors. An equivalent modulus and damping ratio at any strain level are determined from the slope of the major axis of the hysteresis loop corresponding to that strain and the area of the loop, respectively. Since the vertical distribution of shear strain is unknown, initial values of moduli and damping are selected corresponding to small strain values or to strain levels judged appropriate for the anticipated earthquake loading and an elastic analysis is carried out for the entire duration of the earthquake.

The average strain (some percentage of the maximum value, usually 65 percent) is computed at each level; moduli and damping ratios, compatible with these average strains, are selected and calculations repeated. The iterative procedure is continued until no significant changes in moduli and damping are necessary. The response determined during the last iteration is considered to be a reasonable approximation to the non-linear response. The integration of the equations of motion is based on classical wave propagation theory using transfer functions. The computer program can be used to compute the responses for a design motion given anywhere in a soil deposit. Every layer in a soil deposit is homogeneous and isotropic and is characterized by the thickness, h, mass density, ρ, shear modulus, G and damping factor, β; see Figure 1.

FINITE WIDTH OF LAYERS

In the computer program SHAKE, the wave equation is solved for each layer in a soil deposit and the compatibility at the interface between any two consecutive layers is achieved by equating displacements and shear stresses at the nearest faces of the two layers, i.e.: $\tau_m(X_m = h_m) = \tau_{m+1}(X_{m+1} = 0)$; $u_m(X_m = h_m) = u_{m+1}(X_{m+1} = 0)$; see Figure 1.[18] The finite width of the layers can be involved in the formulation if shear force compatibility is enforced instead of shear stress compatibility at the interfaces between the layers. Thus, at the interface between layers i and $i + 1$, shear force compatibility requires (see Figure 2[27]):

$$\tau_i L_i = \tau_{i+1} L_{i+1} \tag{1}$$

where

τ_i is the shear stress at the interface in layer with length L_i,
τ_{i+1} is the shear stress at the interface in layer with length L_{i+1}.

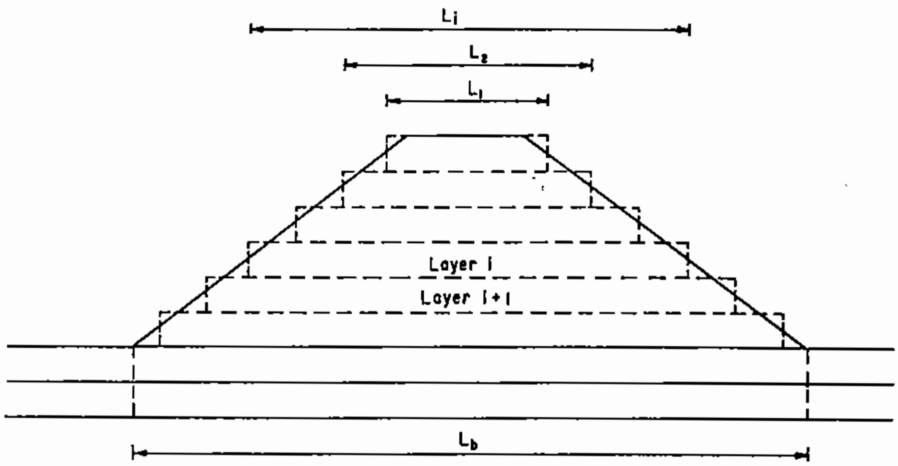


Figure 2. General horizontally layered deposit in an embankment dam—hypothesis

The wave equation solved in the computer program SHAKE is

$$\rho\frac{\partial^2 u}{\partial t^2}=G\frac{\partial^2 u}{\partial x^2}+\eta\frac{\partial^2 u}{\partial x^2 \partial t} \tag{2}$$

where

ρ is the mass density
u is the horizontal displacement
G is the shear modulus
η is the viscosity.

Multiplying the wave equation (2) for layer i, by L_i/L_b where L_i is the length of layer i and L_b is the length of the layer representing the base of the dam, we get:

$$\left(\rho_i\frac{L_i}{L_b}\right)\frac{\partial^2 u}{\partial t^2}=\left(G_i\frac{L_i}{L_b}\right)\frac{\partial^2 u}{\partial x^2}+\left(\eta_i\frac{L_i}{L_b}\right)\frac{\partial^3 u}{\partial x^2 \partial t} \tag{3}$$

Specializing equation (3) for harmonic displacements with frequency ω and working with the general solution of the resulting ordinary differential equation, it is found that the expressions for the horizontal displacements do not change, but the shear stress expressions now have a multiplier of L_i/L_b for layer i, and L_{i+1}/L_b for layer $i+1$. This was, in fact, required for shear force compatibility as per equation (1). In order to get actual shear stress values, the calculated shear stresses will have to be multiplied by L_b/L_i for each layer i.

The above derivation shows that if the values of ρ_i, G_i and η_i are multiplied by L_i/L_b for each layer i having a finite width, the use of the computer program SHAKE in its standard form will enforce a shear force compatibility at the layer interfaces. The shear stresses as printed by the computer program should be multiplied by L_b/L_i to obtain actual shear stresses.

If the shear modulus of layer j is calculated from the shear wave velocity $V_s|_j$ by the formula $G_j=\rho_j(V_s|_j)^2$, then only the density of layer j needs to be multiplied by L_j/L_b. The viscosity η is related to the damping ratio β by the relation:

$$\omega\eta=2G\beta \tag{4}$$

Since η and G are scaled by the same constant, the damping ratio β does not need to be changed.

EFFECT OF CONFINING PRESSURE ON THE DAMPING RATIO

Reference 26 shows the effect of confining pressure on the damping ratio. There is an empirical relation implemented in the standard version of the computer program SHAKE which calculates the factor to modify the input values of damping ratio. This relationship is:

$$BF(I) = 2{\cdot}53 - 0{\cdot}45 \log_{10} \text{(effective overburden pressure)} \tag{5}$$

where BF (I) is the factor modifying the input values of damping ratio for layer I.

There is no mention of this option in the user's manual of SHAKE. It can, however, be implemented in the calculations by specifying BFAC in the second input card of option 2 equal to 0. The availability of this option and its implementation through the above procedure was confirmed with Professor John Lysmer of the University of California, Berkely, during a telephone call by the writer.

SAMPLE PROBLEM

Figure 3 shows the location map and general layout of the Bradbury Dam and its appurtenant structures. Figure 4 gives the cross-sectional details of the embankment dam and its foundation at the maximum section. Figure 5 shows the locations and orientation of the strong-motion accelerometers. These instruments were installed after the completion of the dam in 1950. There is a measured record of time versus ground acceleration at the crest of the Bradbury Dam and at the outcropping base rock layer (near the outlet works) during the Santa Barbara earthquake of 13 August 1978. Two horizontal components of ground acceleration in orthogonal directions (the 340° component is along the axis of the dam and the 250° component is normal to the axis of the dam) were recorded at both instrument sites. Figure 6 shows the plots of these recorded time–acceleration data.

The Bradbury Dam is a zoned earthfill structure with a crest length of 3350 feet and a maximum height of 206 feet above streambed. The cutoff trench beneath the dam has a maximum depth of about 65 feet. The embankment consists of three main zones; the impervious core (zone 1), sand and graval shells (zone 2) and a special zone 3 within the downstream shell. The upstream face of the dam is protected by riprap and downstream face by a cobble zone. Figure 4 shows the relative positions of these material zones and also includes a general description of these materials. References 29 and 30 give the details of field and laboratory investigations carried out to provide basic data for an evaluation of the seismic stability of the existing dam and its foundation for a synthetic earthquake of magnitude $7\frac{1}{4}$. During the course of the dynamic studies of the dam, analyses were performed for the Santa Barbara earthquake record of 13 August, 1978, to provide a check on the accuracy of the computational procedures. It is these later calculations which are used for this sample problem. Figure 7 shows the shear wave velocity values for the layers selected as being representative of the measured data. The shear wave velocity measurements were made at two locations along the crest of the dam using the crosshole technique. The holes were located approximately $4\frac{1}{2}$ feet upstream of the guardrail on the downstream edge of the crest. For other details of the data for dynamic analysis of the dam, the reader is referred to References 29, 30 and 31.

TWO-DIMENSIONAL DYNAMIC FINITE ELEMENT ANALYSIS

Figure 8(a) shows the finite element discretization used to perform the two-dimensional dynamic finite element analysis of the Bradbury Dam cross-section using the computer program FLUSH. This cross-section is located in the vicinity of the crest instrument location (deep section). The

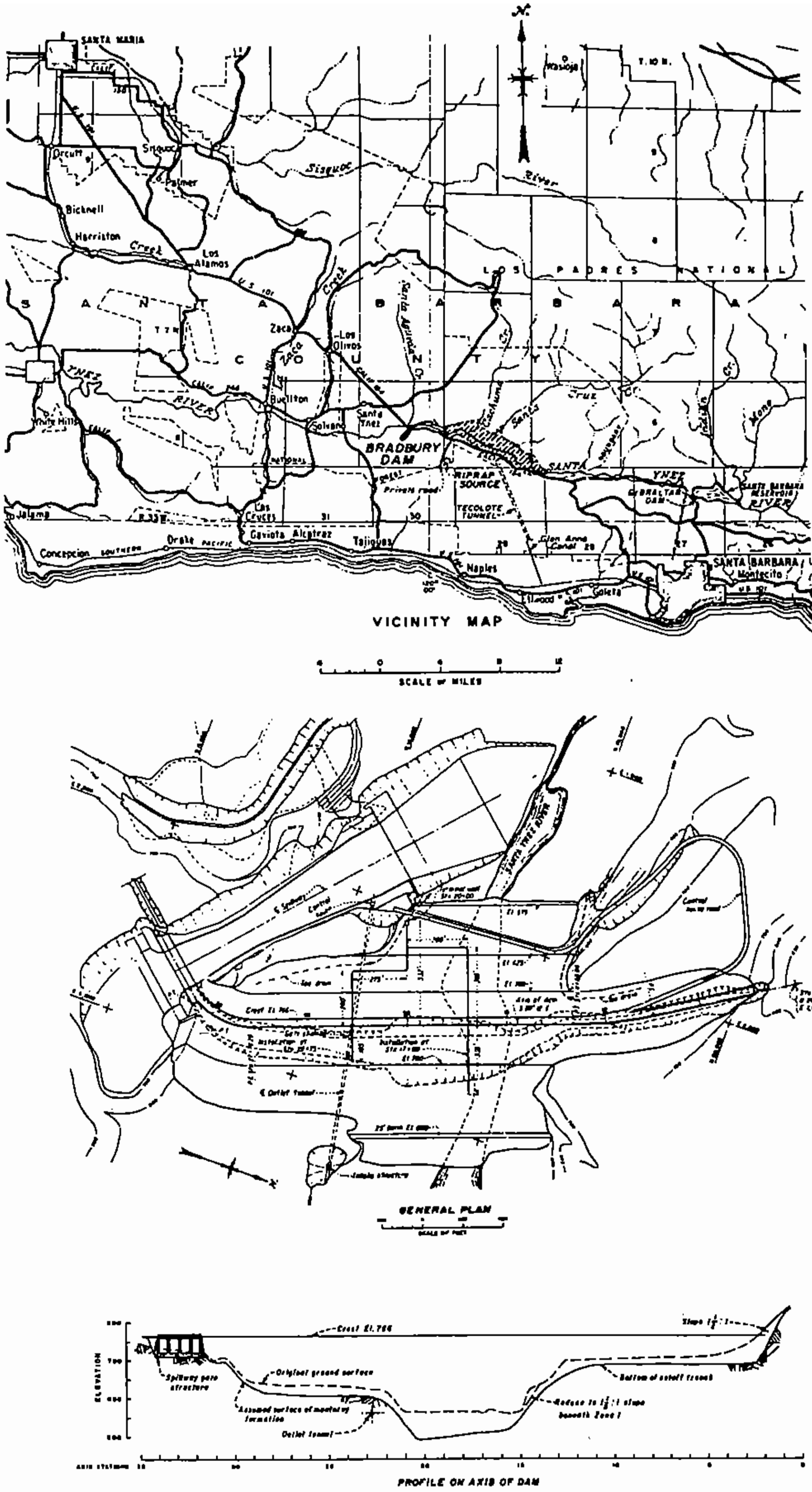
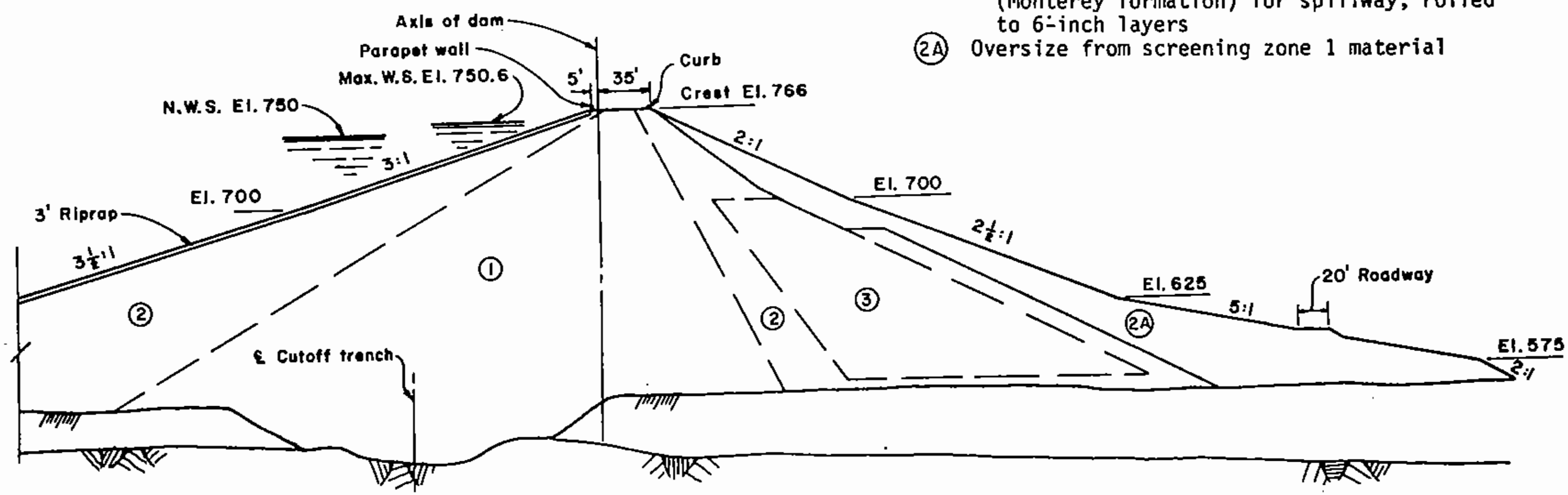


Figure 3. Location map and general layout of the Bradbury Dam and appurtenant structures



EMBANKMENT EXPLANATION

1. Selected clay, sand, and gravel from borrow area No. 1, reservoir borrow area, and excavations for dam and appurtenant works, rolled to 6-inch layers
2. Selected sand, gravel, and cobbles from borrow area No. 2, excavations for dam and appurtenant works, and oversize from screening zone 1 material, compacted to 12-inch layers by crawler-type tractor
3. Selected material from class 2 excavation (Monterey formation) for spillway, rolled to 6-inch layers

2A Oversize from screening zone 1 material

Figure 4. Cross-section of the Bradbury Dam

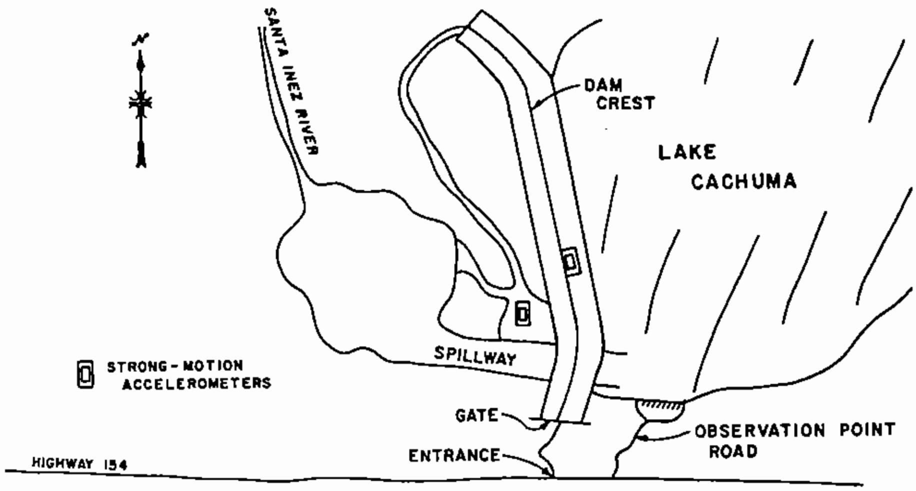


Figure 5. Locations of strong-motion accelerometers—general view

critical inputs required for these calculations are the bedrock acceleration–time data, and the shear modulus and damping parameters.

Input base rock acceleration

The 250° component of the bedrock acceleration–time data was calculated from the corresponding measured bedrock outcrop record using the computer program SHAKE. The SHAKE column consisted of 3 layers, including the halfspace, in the free field. The layer thickness, in-place material density and initial shear wave velocity data used for these calculations are shown in Figure 7. The initial critical damping ratio for each of the two layers was assumed to be 0·05. The variations of shear modulus and damping ratio with effective shear strain were assumed to be as given in Reference 32. The computed bedrock acceleration–time response in the free field is shown in Figure 8(b).

Shear modulus

The average shear modulus of the embankment materials was estimated from the fundamental period of the dam. Fourier amplitude spectra were calculated for the 250° component of the measured time–acceleration records at the baserock outcrop and crest of the dam. The number of points used for the spectral analysis was 2048; the time interval was 0·01 in each case. The amplification spectrum was calculated by dividing the Fourier amplitude of the crest record by the corresponding Fourier amplitude of the outcrop baserock record. These results are shown in Figure 9. The amplification spectrum for the recorded 250° component shows two dominant peaks at frequencies of 1·563 and 1·758 Hz. This corresponds to period of 0·64 and 0·57 s, respectively. The fundamental period of the dam was taken to be 0·64 s. The shear modulus of the dam material was estimated from

$$V_s = 2{\cdot}6\frac{H}{T} \tag{6}$$

$$G = \rho\, V_s^2 \tag{7}$$

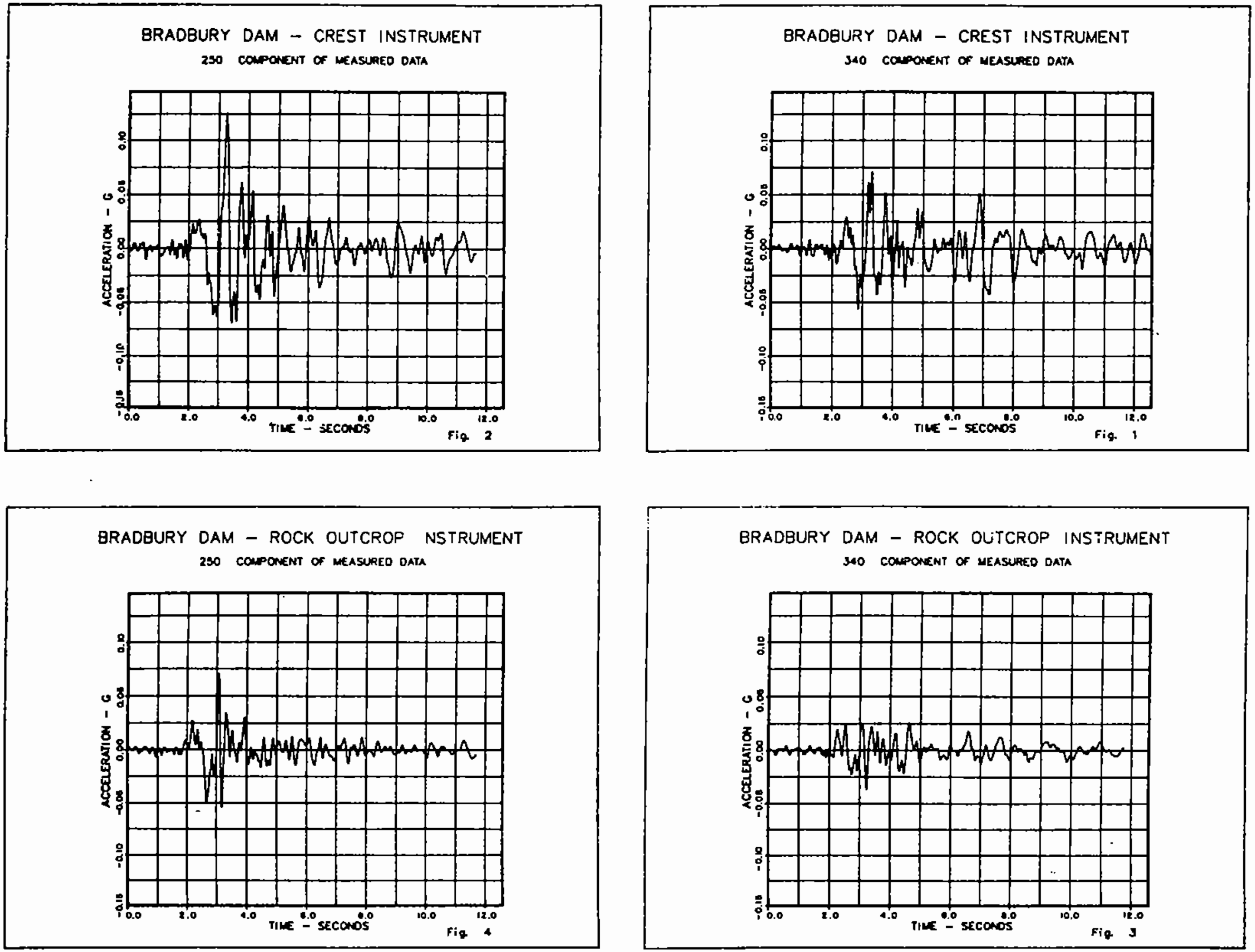


Figure 6. Acceleration-time records of measured data

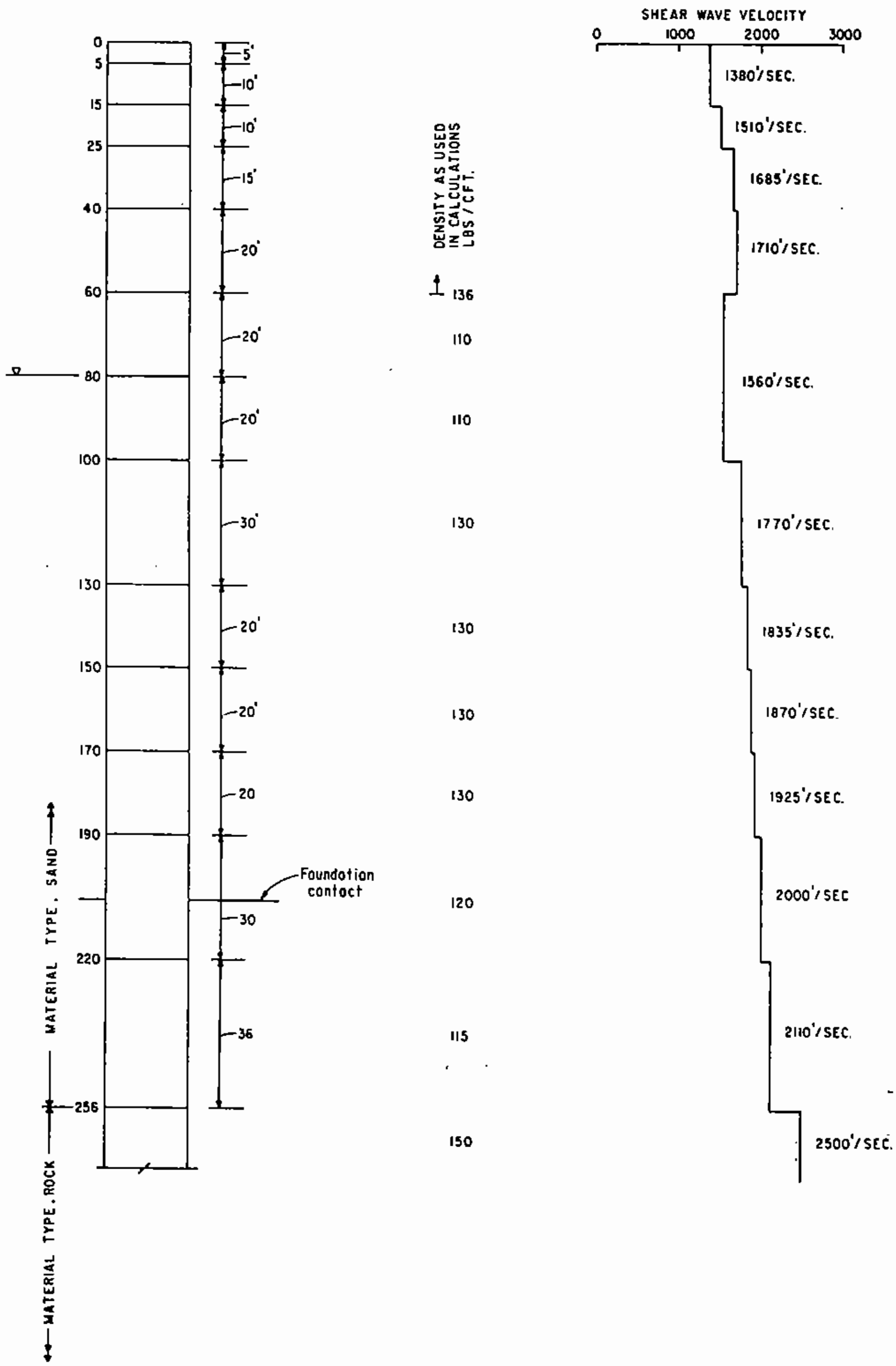


Figure 7. Basic data for the Bradbury Dam site

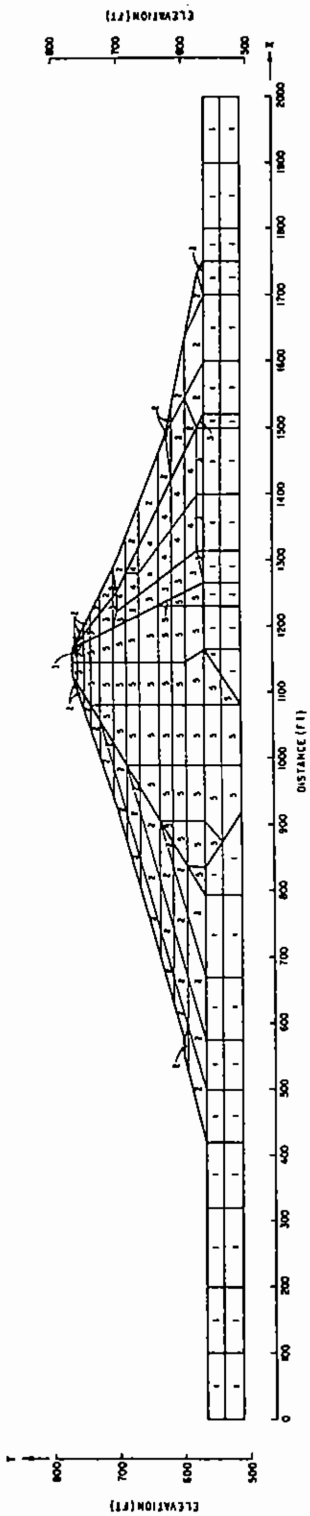


Figure 8(a). Finite element discretization

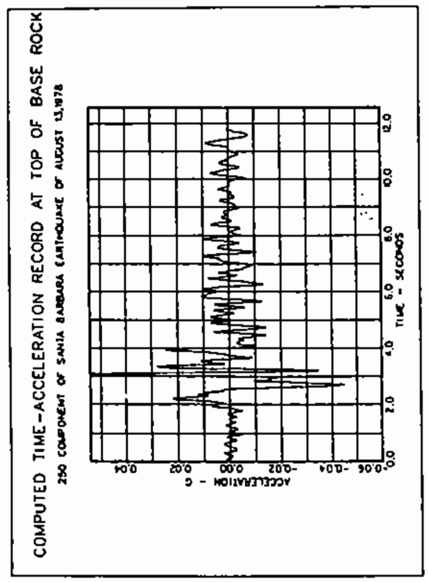
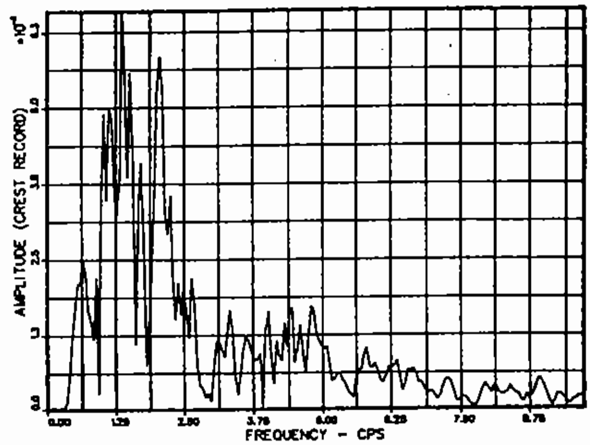


Figure 8(b). Computed baserock time-acceleration data

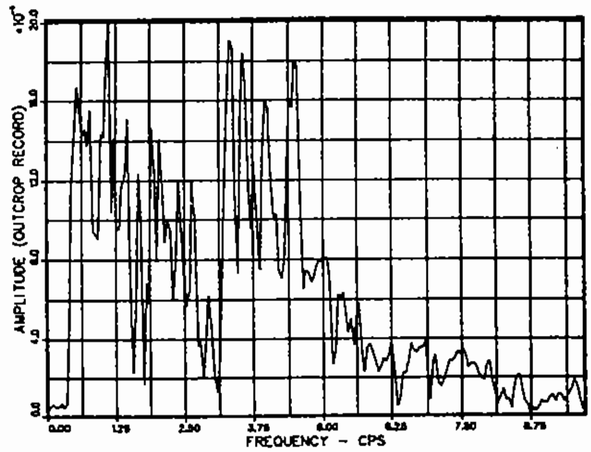
FOURIER SPECTRUM OF 250 COMPONENT OF CREST RECORD

SANTA BARBARA EARTHQUAKE OF AUGUST 13,1978



FOURIER SPECTRUM OF 250 COMPONENT OF OUTCROP RECORD

SANTA BARBARA EARTHQUAKE OF AUGUST 13,1978



RATIO OF FOURIER AMPLITUDES OF 250 COMPONENTS
(MEASURED RECORDS)

SANTA BARBARA EARTHQUAKE OF AUGUST 13,1978

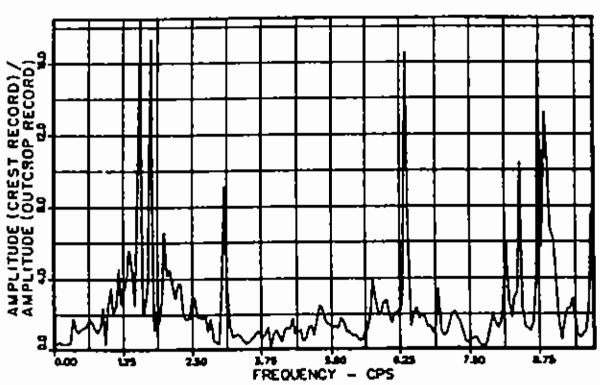
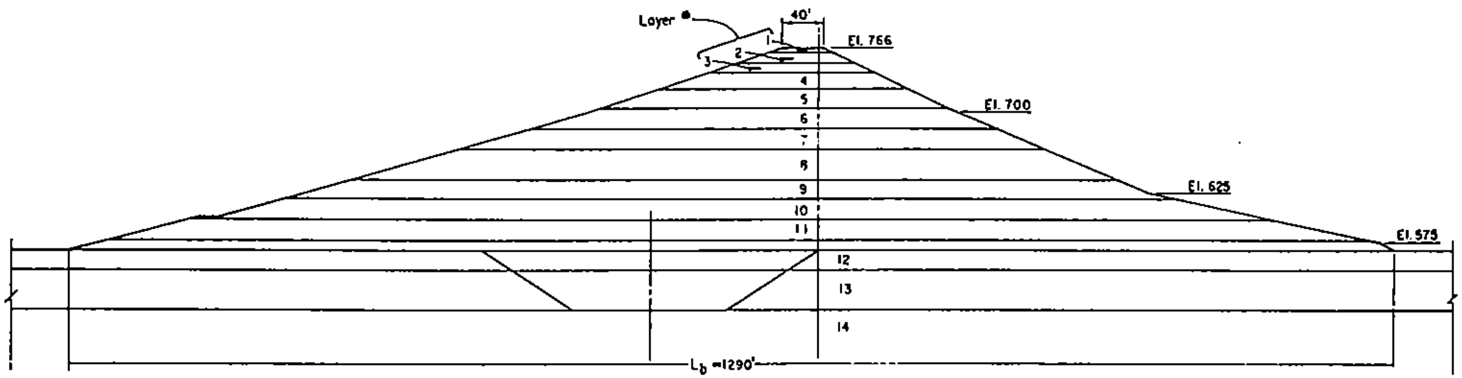


Figure 9. Fourier spectrum analysis of measured time-acceleration records



Layer #	L_i (FT.)	γ_i (p.c.f.)	$\frac{\gamma_i L_i}{L_b}$ (p.c.f.)
1	(40+60)/2 = 50.	136	5.27
2	(60+116)/2 = 88.	136	9.28
3	= 138.	136	14.55
4	= 200.	136	21.09
5	= 290.	136	30.57
6	= 396.	110	33.77
7	= 509.	110	43.40
8	= 657.	130	66.21
9	= 812.	130	81.83
10	= 970.	130	97.75
11	= 1145.	130	115.39
12	(1290+1290)/2 = 1290.	120	120.00
13	1290 = 1290.	115	115.00
14	1290 = 1290.	150	150.00

Figure 10. Vertical discretization through the Bradbury Dam and its foundation

where

H is the height of the dam
G is the shear modulus of the dam material
ρ is the mass density of the dam material
V_s is the shear wave velocity
T is the fundamental period of the dam.

For $H = 256$ ft, $T = 0{\cdot}64$ s, equation (6) gives $V_s = 1040$ ft/s. For an average material weight density of 125 lb/ft^3, it gives an average value for shear modulus of 4198·75 k/ft^2.

Analysis

For the dynamic analysis of the embankment section, the dam was modelled as a homogeneous material body with an average shear wave velocity of 1040 ft/s. The factors for scaling the shear modulus and damping ratio with effective shear strain are those given in Reference 32. The foundation zone was modelled as a layered medium with transmitting lateral boundaries. The 250° component of the baserock motion was applied at the rigid base of the model. The highest frequency considered in the analysis was 8 Hz. The height of the elements in the finite element discretization was kept identical to the layer thicknesses selected as representative of the measured shear wave and material density data (Figure 7).

The calculations were made to obtain maximum dynamic shear stress in all elements caused by the earthquake loading; maximum acceleration values at the nodes lying along the line passing through the centre of the dam crest; time–acceleration response at the node lying at the centre of the dam crest; and the acceleration, velocity and displacement response spectra for the time–acceleration response at the centre of the dam crest. The response spectra were calculated with 5 per cent damping ratio. Results of these calculations are shown in Figures 11–15 where they are properly identified as being results of FLUSH calculations. The comparison of these results with 'SHAKE' results with and without modifications is discussed in the next section.

ONE-DIMENSIONAL DYNAMIC ANALYSES

The response of the Bradbury Dam along with its foundation in the vicinity of the crest instrument location (deep section) was carried out using the SHAKE computer program with the two features discussed above being active. This modified procedure is named SHAKEM. Calculations were also made using the computer program SHAKE (without modifications). In each case the analysis was performed using the 250° component of the measured rock outcrop record as an input motion. The layer thickness, the shear wave velocity and material density data used for calculating the shear modulus for each layer are shown in Figure 7. Figure 10 shows the arrangement of layering, based on the shear wave velocity data, selected for the one-dimensional dynamic analyses. The variations of shear modulus and damping ratio with effective shear strain were assumed to be as given in Reference 32. For each analysis, maximum dynamic shear stress, and maximum calculated acceleration for all layers were printed out. The acceleration, velocity and displacement spectra were computed for the crest and outcrop recorded data and also for the computed crest time–acceleration values using 5 per cent damping ratio.

Results of these calculations are shown in Figures 11–15 where they are properly identified as being results of measured data, computed response using SHAKEM and computed response using SHAKE.

Comparison of results

Table I gives the comparison of measured peak acceleration and the corresponding values of the computed accelerations using the three analysis procedures, i.e. two-dimensional FEM procedure

Table I. Comparison of computed results with the observed data at Bradbury Dam site; 13 August 1978 Santa Barbara earthquake; FLUSH, modified SHAKE and SHAKE computer programs

No	Input motion location	Response motion location	Response: Peak acceleration: Measured *g	Peak acceleration: Computed *g	Acceleration spectrum: Max. of measured record *g	Acceleration spectrum: Max. of computed record *g	Acceleration spectrum: Area under measured record *g	Acceleration spectrum: Area under computed record *g	Velocity spectrum: Max. of measured record ft/s	Velocity spectrum: Max. of computed record ft/s	Velocity spectrum: Area under measured record ft	Velocity spectrum: Area under computed record ft	Method
1	Outcrop record 250° component	Crest 250° orientation	0·12604		0·301		0·312		1·084		1·700		Measured
2	Baserock	Crest 250° orientation		0·1418		0·360		0·458		1·858		2·853	FLUSH †
3	Outcrop record 250° component	Crest 250° orientation		0·14676		0·475		0·267		0·815		1·290	SHAKEM
4	Outcrop record 250° component	Crest 250° orientation		0·09081		0·250		0·189		0·615		0·969	SHAKE

† Baserock motion is computed from the measured baserock outcrop record using SHAKE computer program in the free field. The embankment material is assumed homogeneous with an average shear wave velocity of 1040 ft/s.

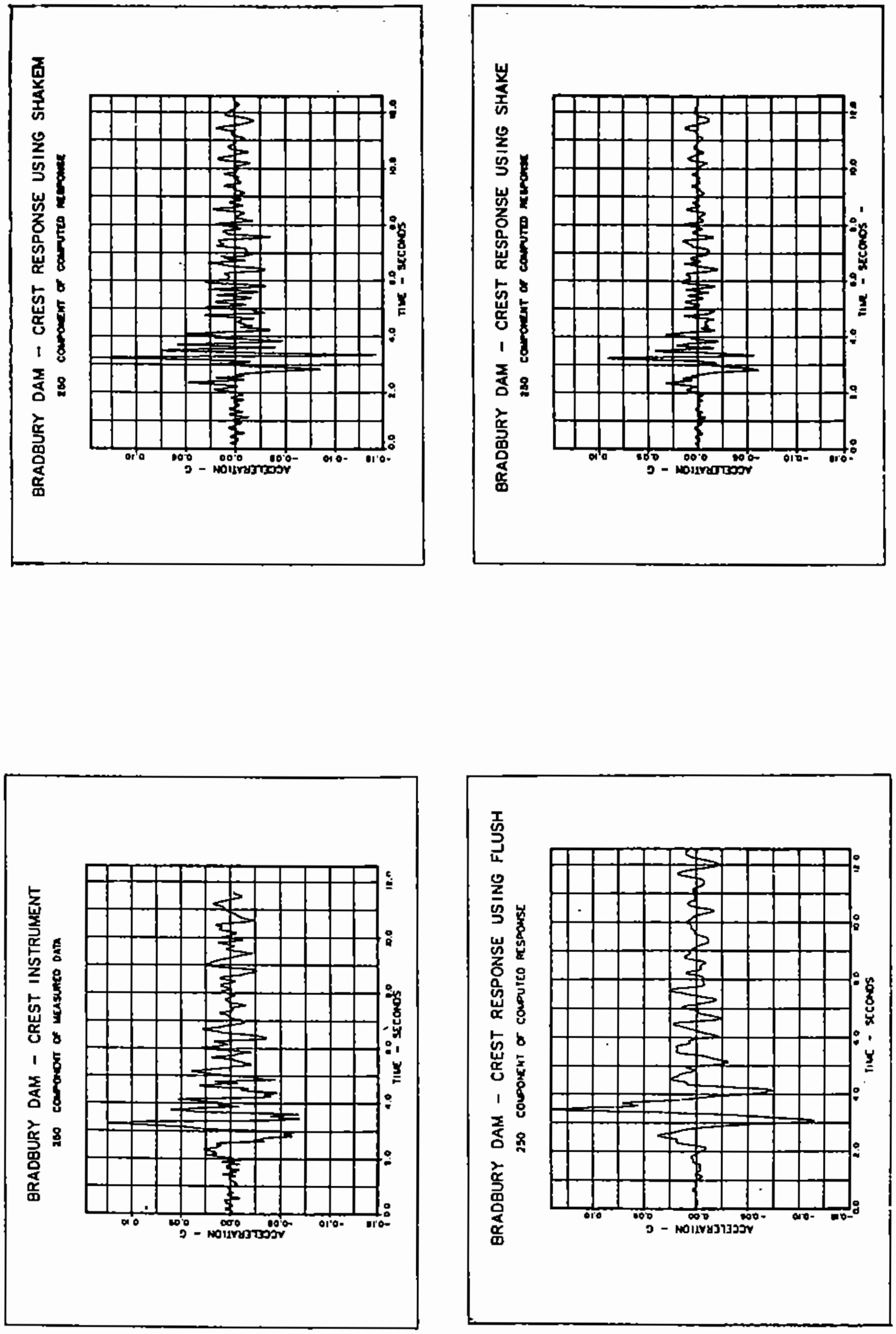


Figure 11. Time-acceleration response at the crest of the dam

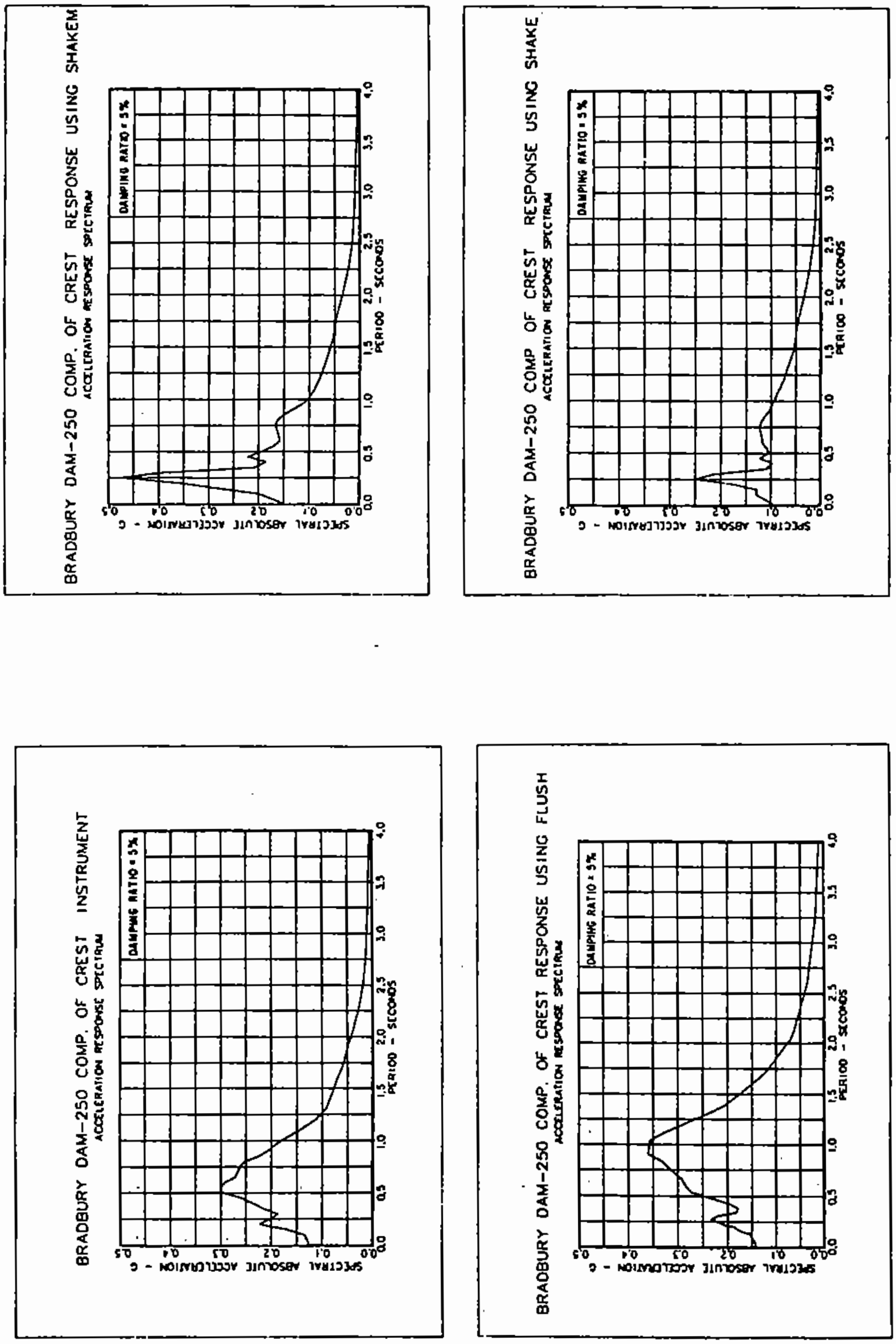


Figure 12. Acceleration response spectra for the crest of the dam

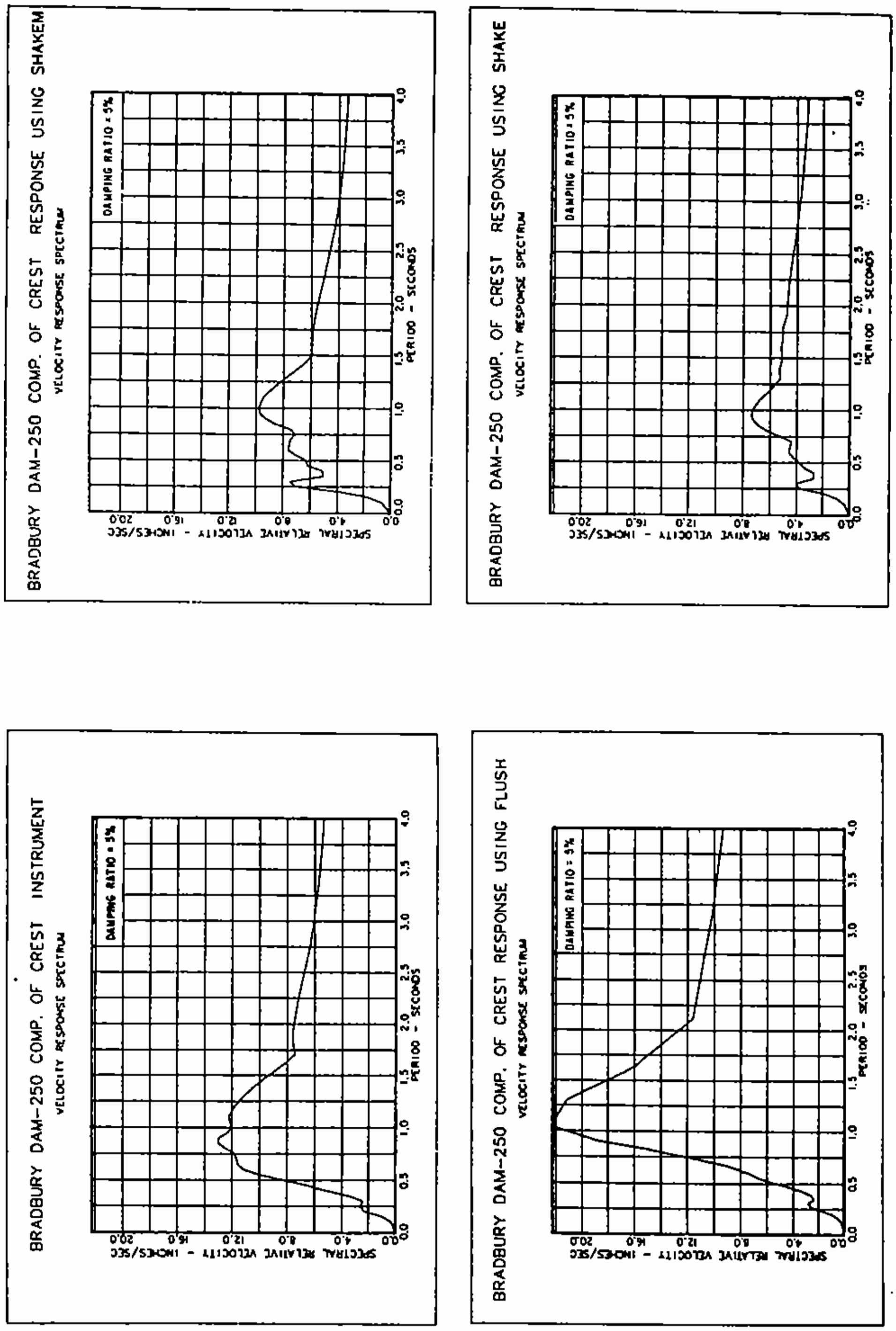


Figure 13. Velocity response spectra for the crest of the dam

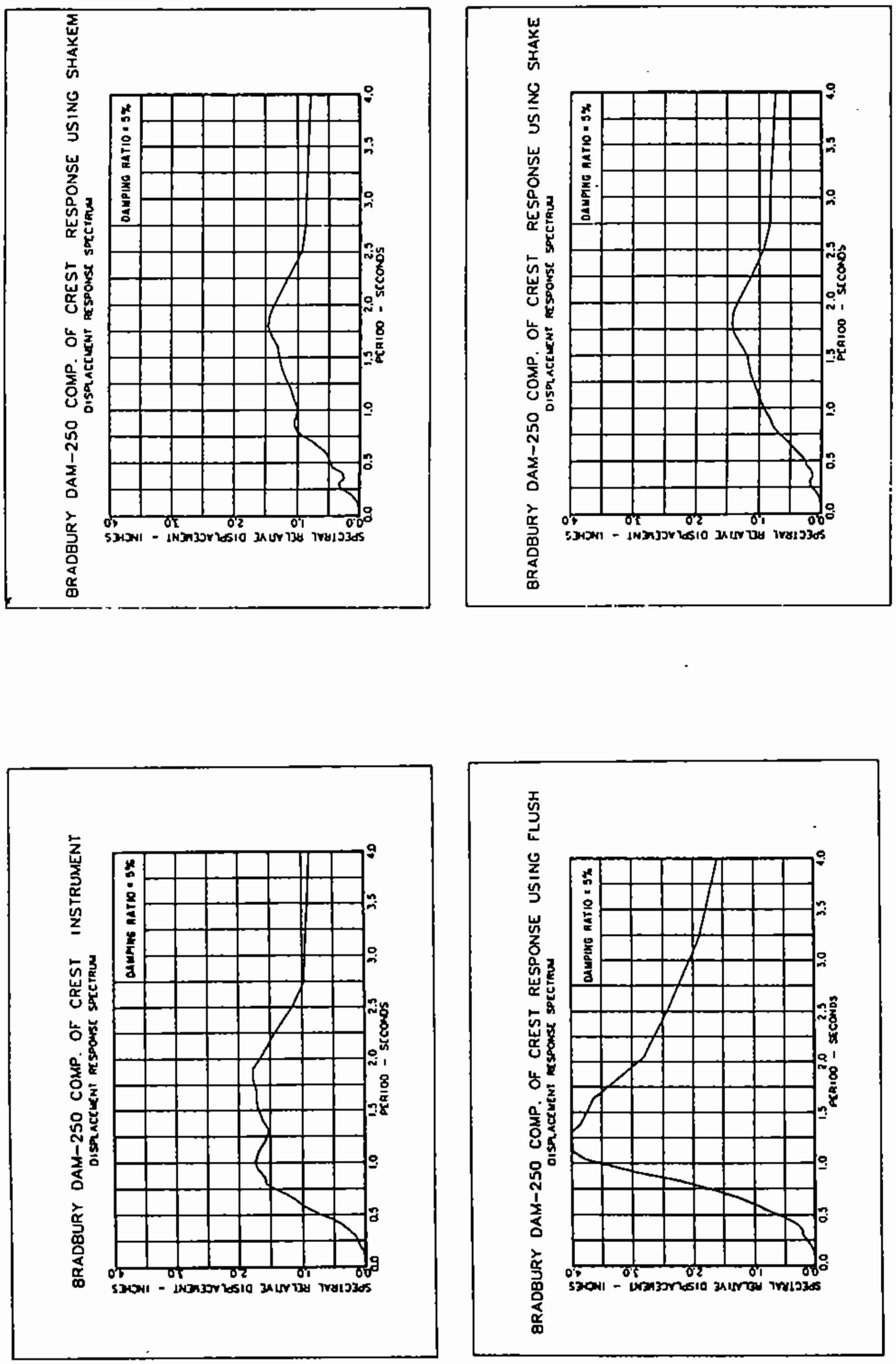


Figure 14. Displacement response spectra for the crest of the dam

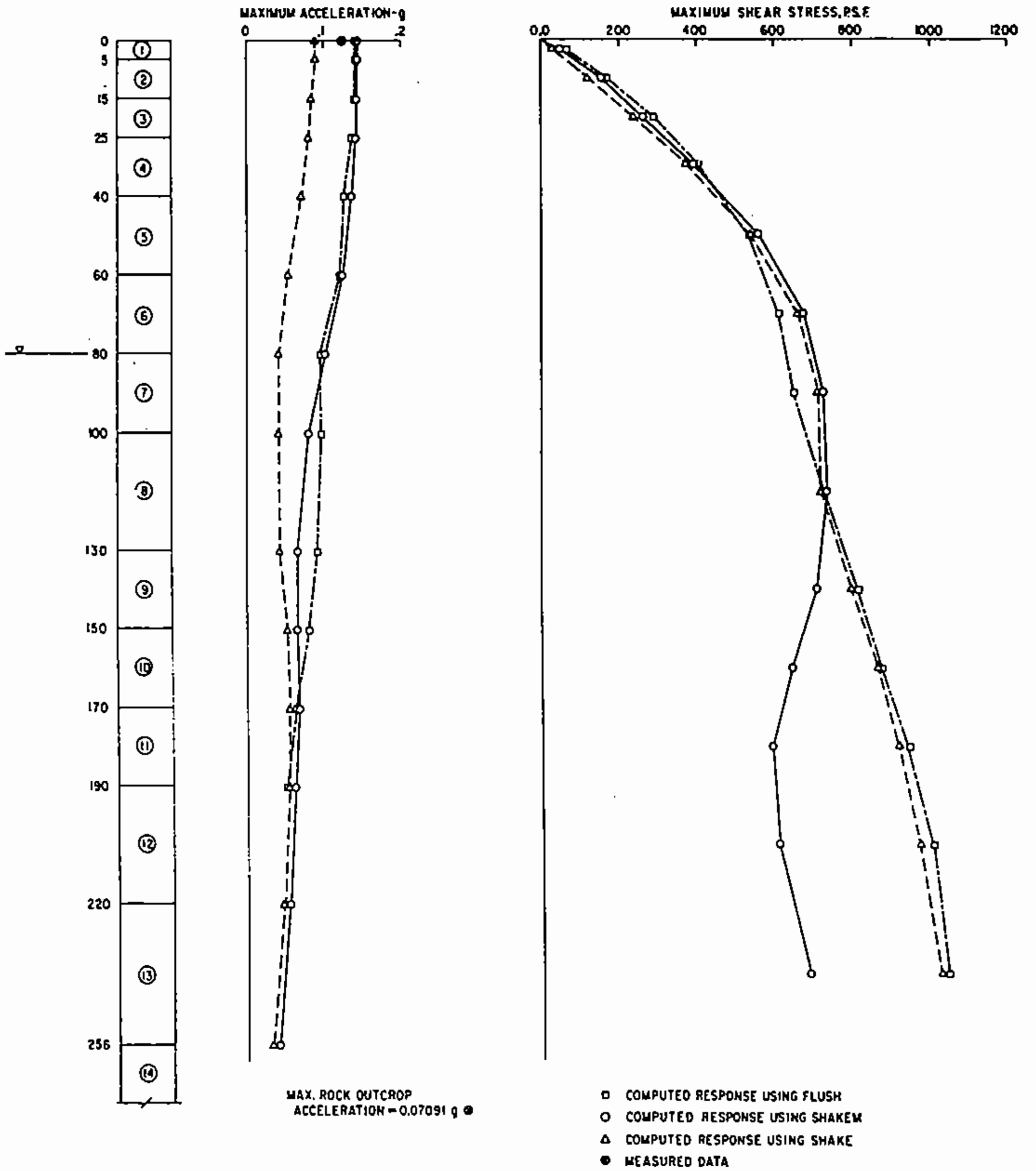


Figure 15. Comparison of maximum accelerations and maximum shear stresses using FLUSH, SHAKEM and SHAKE

FLUSH, modified one-dimensional procedure SHAKEM, and one-dimensional procedure SHAKE. The calculated time–acceleration responses for the crest of the dam using FLUSH, SHAKEM and SHAKE and the measured crest response are shown in Figure 11. The acceleration, velocity and displacement spectra for the 250° component of the crest record and the corresponding computed responses by the three procedures are shown in Figures 12, 13 and 14, respectively. Figure 15 gives the comparison of variation of maximum acceleration values and maximum shear

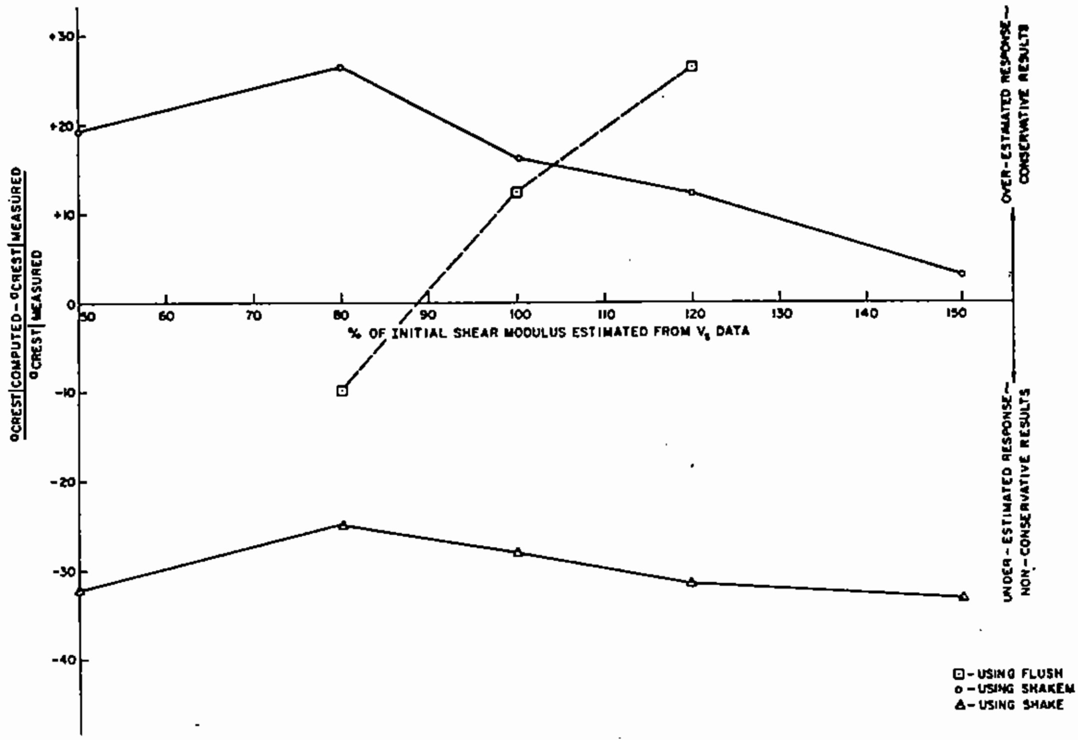


Figure 16. Comparison of percentage error in crest acceleration for ± 20 per cent and ± 50 per cent variations in shear modulus values using FLUSH, SHAKEM and SHAKE

stress values through the height of the embankment dam and the foundation, along the line passing through the centre of the dam crest, as obtained by the three procedures.

It can be seen that the analysis results obtained using FLUSH and SHAKEM compare more favourably with the observed data, as compared to the results obtained using the standard form of the SHAKE program (without modifications). This lends credibility to the use of items 1 and 2 discussed in this paper.

Perturbation analysis

The objective of this study was to study the effects of uncertainty (or variation) in the values of shear moduli on the computed response. The calculations for the dynamic analysis of the Bradbury Dam were repeated for each of the three computational procedures using uniformly perturbed values of shear moduli, all else being equal. Specifically, the values of shear moduli were changed ± 20 per cent of their initial values (assumed to be at 100 per cent), measured or computed, for the three calculation procedures and also ± 50 per cent for the one-dimensional analysis procedures. The difference between the computed peak crest acceleration and the measured peak crest acceleration expressed as a percentage is shown in Figure 16. In all cases SHAKEM results are stable, and compare reasonably well with the observed data. The FLUSH analysis results, though comparing favourably at 100 per cent of the computer average homogeneous shear wave velocity with the observed data, show an appreciably larger sensitivity to the shear modulus parameter. SHAKE (without modifications) shows relatively little sensitivity to the variations in shear modulus parameter and remains inferior to the measured data.

GENERAL COMMENTS

There are sizeable simplifications involved in the one-dimensional computational procedure discussed in this paper as compared to the physical problem, e.g. the procedure considers only one horizontal component of ground shaking, whereas in reality the level ground is simultaneously subjected to three components of shaking; the one- or pseudo-two-dimensional representation of an embankment and its foundation as used may be too simple an approximation of the actual three-dimensional geometry, etc. Also, because of the non-linear behaviour of embankment dam materials, the response at smaller vibration levels may not necessarily be a true indication of the dam's performance during strong earthquakes. These and other important differences (such as neglecting the reflection and scattering of waves by the inclined free surfaces of the side slopes) in the numerical model and the actual physical situation under study should be kept in focus in using the results of any computational procedure.

Some of the simplifications required in the one-dimensional analysis procedure are not needed in the two-dimensional finite element analysis procedure. The effects of material variability, the three-dimensional geometry and other parameters of the physical problem on the embankment response, to some scale, got considered in the determination of one average shear wave velocity for the entire cross-section of the dam from the observed records at the baserock outcrop and dam crest for the Santa Barbara earthquake.

Table I gives a comparison of the computed responses of the Bradbury Dam for the Santa Barbara earthquake of 13 August 1978, using the FLUSH computer program, the modified SHAKE computer program and the standard SHAKE computer program (without modifications) and with the observed response data. A review of these results indicates the following:

1. The calculated response using the computer program SHAKEM or the computer program FLUSH is generally closer to the observed response than the computed response using the standard SHAKE computer program (without modifications).
2. Using the outcrop record as input motion and the SHAKEM computer program, or the computed baserock motion as input and the FLUSH computer program, the calculated peak accelerations at the crest of the dam are somewhat higher than the measured peak acceleration at the crest of the dam. This result is slightly conservative. The same is not true for the standard SHAKE results.
3. The velocity response spectrum for the computed crest time–acceleration using the SHAKEM computer program and the measured crest time–acceleration data show that peak spectral velocity for the computed data is somewhat less than for the observed data or the FLUSH analysis results, and so are the spectral intensities.[33] The spectral intensity, which is the area under the velocity response spectrum, was calculated for values in period 0·1 to 2·5 s. The corresponding difference using the standard SHAKE computer program is relatively large.
4. The variation in computed values of maximum shear stress with depth as obtained from using the SHAKEM computer program has a reversal in the bottom half of the total depth as opposed to the essentially monotonic increase of the maximum shear stress with depth as obtained from the FLUSH and SHAKE computer programs (Figure 15). However, the equivalent numbers of uniform stress cycles for each of the layers, calculated at 0·65 of the maximum shear stress, for SHAKEM results are higher than for SHAKE results. This information was not available for the FLUSH analysis results. The maximum shear stresses as obtained by the SHAKE computer program (without modifications) are in excellent agreement with those obtained by the FLUSH analysis. A similar finding is reported in Reference 8.

CONCLUSIONS

The proposed modifications to the use of the SHAKE computer program allow a better representation of the actual finite length of the layers within the embankment dam in the computational procedure. This should, in general, give a better computed response (better correlation with the observed or actual response). The particular analysis, with all its attendant limitations, reported in this paper tends to support this argument. Additionally, the modified procedure can be used to analyse horizontally layered deposits of infinite lateral extent by making the lengths of all the layers equal, i.e. the results of SHAKEM become identical to those of SHAKE.

ACKNOWLEDGEMENTS

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NOTATION

G_i = shear modulus of layer i
h_i = thickness of layer i
L_b = length of the layer representing the base of the dam
L_i = length of layer i
u_i = horizontal displacement in layer i
V_s = shear wave velocity
X_i = local co-ordinate for layer i
β_i = damping factor for layer i
γ_i = weight density of layer i
ρ_i = mass density of layer i
τ_i = shear stress at the interface in the layer with length L_i

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