United States Society on Dams

USSD

Strength of Materials for Embankment Dams

February 2007

A White Paper prepared by the USSD Committee on Materials for Embankment Dams

U.S. Society on Dams

Vision

To be the nation's leading organization of professionals dedicated to advancing the role of dams for the benefit of society.

Mission — USSD is dedicated to:

- Advancing the knowledge of dam engineering, construction, planning, operation, performance, rehabilitation, decommissioning, maintenance, security and safety;
- Fostering dam technology for socially, environmentally and financially sustainable water resources systems;
- Providing public awareness of the role of dams in the management of the nation's water resources;
- Enhancing practices to meet current and future challenges on dams; and
- Representing the United States as an active member of the International Commission on Large Dams (ICOLD).

Printed in the United States of America

U.S. Society on Dams
1616 Seventeenth Street, #483
Denver, CO 80202
Telephone: 303-628-5430
Fax: 303-628-5431
E-mail: stephens@ussdams.org
Internet: www.ussdams.org

FOREWORD

This white paper is intended to be a "living document" that presents the "state of the practice" for determining fill strength parameters for earthfill embankment dams and the static analysis of embankment dams. To provide information to update this document, please contact the Chair of the USSD Committee on Materials for Embankment Dams at www.ussdams.org/c_mater.html. This white paper was last updated in June 2006.

TABLE OF CONTENTS

CHAPTER 1 — INTRODUCTION

This white paper presents the results of a survey initiated by the Committee on Materials for Embankment Dams to determine the "state of the practice" for selecting fill strength parameters used in static and pseudostatic analyses of earthfill embankment dams. A questionnaire was sent to federal and state agencies, consulting firms, and private consultants experienced in the design and safety evaluation of embankment dams. Among the federal and state agencies contacted were: U. S. Army Corps of Engineers (USACE); U. S. Bureau of Reclamation (USBR); Federal Energy Regulatory Commission (FERC); U. S. Department of Agriculture, National Resources Conservation Service (NRCS); Tennessee Valley Authority (TVA); and California Department of Water Resources, Division of Dam Safety (DSOD).

The questionnaire asked each respondent to provide information as to their practice in determining the static strength of embankment materials and analyzing the static stability of embankment dams. The information provided included: approach to static stability analyses, loading conditions, shear strength parameters, field and laboratory testing used to determine shear strength parameters, procedures for interpreting test results, factors of safety, and methodologies used for static analyses.

The information provided by the respondents is summarized in this white paper, which is divided into the following sections:

- Loading Conditions for Embankment Dams, including End of Construction, Steady-State Seepage, Rapid Drawdown, and Earthquake.

- Determination of Shear Strengths, including subsections describing total and effective stresses, selection of shear strengths, and the use of cohesion. The information provided in this white paper regarding the determination of shear strengths applies primarily to earthfill materials: soils with a grain size less than 20 mm.

- Static Analyses of Embankment Stability, including subsections on the limit equilibrium method of slope stability analysis, types of potential failures surfaces evaluated, location of potential failure surfaces for analyses, and recommended factors of safety.

The original questionnaire was prepared by Constantine G. Tjoumas in 1997. The responses were incorporated into this paper and updated by Ralph W. Rabus in 2006. The paper was peer-reviewed by other members of the Committee on Materials for Embankment Dams.

This white paper is not intended to be a "how to" manual in selecting material strengths and the reader is directed to the references for more comprehensive discussion of the subject. An experienced geotechnical engineer should be involved in the determination of fill strengths and the performance of stability analyses.

CHAPTER 2 — LOADING CONDITIONS FOR EMBANKMENT DAMS

The stability of the upstream and downstream slopes of the dam embankment is analyzed for the most critical or severe loading conditions that may occur during the life of the dam. These loading conditions typically include:

1) *End of Construction* — when significant pore pressure development is expected either in the embankment or foundation during construction of the embankment.
2) *Steady-State Seepage* — when the long-term phreatic surface within the embankment has been established.
3) *Rapid (or Sudden) Drawdown* — when the reservoir is drawn down faster than the pore pressures can dissipate within the embankment after the establishment of steady-state seepage conditions.
4) *Earthquake* — when the embankment is subjected to seismic loading.

For the evaluation of embankment dam stability, the applicable loading conditions need to be determined. These loadings conditions are discussed in the following subsections.

2.1 END OF CONSTRUCTION

The end-of-construction loading condition is usually analyzed for new embankments that 1) include fine-grained soils, and 2) are constructed on fine-grained saturated foundations that may develop excess pore pressures from the loading of the embankment. The embankment is constructed in layers with soils at or above their optimum moisture content that undergo internal consolidation because of the weight of the overlying layers. Embankment layers may become saturated during construction as a result of consolidation of the layers or by rainfall. Because of the low permeability of fine-grained soils and the relatively short time for construction of the embankment, there is little drainage of the water from the soil during construction: resulting in the development of significant pore pressures. Soils with above optimum moisture content will develop pore pressures more readily when compacted than soils with moisture contents below optimum. Both the upstream and downstream slopes of the embankment are analyzed for this condition.

In general, the most severe construction loading condition is at the end of construction. If the embankment is constructed in stages, however, there may be some intermediate construction stages that require analysis.

Based on responses to the questionnaire, this loading condition is not usually considered for very small dams (less than 20 feet high). The NRCS, for example, seldom analyzes this condition for embankments less than 60 feet high (Reference 1). Where the foundation consists of soft saturated clays that are susceptible to excessive pore pressures developed by the loading of the dam, however, the NRCS recommends the analysis of this condition for low dams.

2.2 STEADY-STATE SEEPAGE: NORMAL POOL

After a prolonged storage of reservoir water, water percolating through an embankment dam will establish a steady-state condition of seepage. The upper surface of seepage is called the phreatic line.

It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage (or steady seepage) conditions with the reservoir at its normal operating pool elevation (usually the spillway crest elevation) since this is the loading condition the embankment will experience most. This condition is also called *steady-state seepage under active conservation pool* (USBR), *steady seepage with maximum storage pool* (USACE and FERC), *normal operating condition* (TVA), and *steady seepage – normal pool* (NRCS).

2.3 STEADY-STATE SEEPAGE: FLOOD SURCHARGE

Where the maximum flood storage elevation is significantly higher than the normal pool elevation, the effect of the raised reservoir level (or flood surcharge) on the stability of the downstream slope is normally analyzed. The flood surcharge is generally considered a temporary condition causing no additional saturation of the dam embankment; therefore, the steady-state seepage conditions developed from the normal operating pool elevation is used for this analysis. This loading condition is also called *steady-state seepage under maximum reservoir level* (USBR), *steady seepage with surcharge pool* (USACE and FERC), and *flood surcharge pool* (TVA).

2.4 STEADY-STATE SEEPAGE: PARTIAL POOL

When the reservoir is maintained at an intermediate level or during the filling of a reservoir, an analysis of the partial-pool loading condition may be required by the review agencies. This condition assumes that steady-state seepage has been established at the lower reservoir level. In addition to the downstream slope, the upstream slope is analyzed for this condition to determine the pool elevation that results in the lowest factor of safety.

2.5 RAPID (OR SUDDEN) DRAWDOWN FROM NORMAL POOL

This loading condition assumes that steady-state seepage conditions have been established within the embankment as a result of maintaining a reservoir at the normal pool elevation and that the embankment materials beneath the phreatic surface are saturated. The reservoir is then drawn down faster than the pore pressures within the embankment materials can dissipate, resulting in a reduced factor of safety. This loading condition is the normal operating case for pumped-storage reservoirs where the drawdown of the reservoir (up to 5 to 10 feet per hour) occurs daily. This loading condition is analyzed for the upstream slope of the dam.

This loading condition is called *rapid drawdown from normal pool* (USBR), *sudden drawdown from spillway elevation* (USACE and FERC), *rapid drawdown* (NRCS), and *sudden drawdown* (TVA).

2.6 RAPID (OR SUDDEN) DRAWDOWN FROM MAXIMUM POOL

Where the maximum flood storage elevation is significantly higher than the normal pool elevation, an analysis of the effect of the rapid drawdown of the reservoir on the stability of the upstream slope may be required. The maximum pool is considered a temporary condition causing no additional saturation of the dam embankment; therefore, the steady-state seepage conditions developed from the normal operating pool elevation is used for this analysis. This loading condition is called *rapid drawdown from maximum pool* (USBR) and *sudden drawdown from maximum pool* (USACE and FERC).

2.7 EARTHQUAKE (PSEUDOSTATIC ANALYSIS)

Earthquakes result in an additional loading on the dam embankment materials. In this white paper, we will discuss only the fill shear strengths used for the pseudostatic method of analyses for earthquake loading conditions. The pseudostatic analysis does not apply where there may be a loss of soil strength (e.g., liquefaction) in the embankment or foundation materials during a seismic event.

The pseudostatic analysis assumes that the earthquake causes an additional horizontal force in the direction of failure. This force is equal to a seismic coefficient times the weight of the sliding mass. The pseudostatic method of analysis is normally applied to those critical failure surfaces determined by the long-term static loading conditions (such as steady-state seepage resulting from normal reservoir pool elevation). The pseudostatic method of analysis is not usually applied to short-term to temporary static loading conditions (such as end of construction, flood storage pool, or rapid drawdown), except when this condition is the normal operating case. Where reservoir drawdown occurs on a daily cycle (such as for a pumped storage project), earthquake loading in combination with rapid drawdown is recommended by respondents.

CHAPTER 3 — DETERMINATION OF SHEAR STRENGTHS

3.1 TOTAL AND EFFECTIVE STRESSES

Before we discuss the determination of shear strengths for fill materials, it is necessary to provide background information on drained and undrained soil conditions, and total and effective stresses.

As discussed in *Soil Strength and Slope Stability* (Reference 2):

- "*Drained* is the condition under which water is able to flow into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Under drained conditions, changes in the loads on the soil do not cause changes in the water pressure in the voids in the soil, because the water can move in or out of the soil freely when the volume of voids increases or decreases in response to the changing loads.
- *Undrained* is the condition under which there is no flow of water into or out of a mass of soil in the length of time that the soil is subjected to some change in load. Changes in the loads on the soil cause changes in the water pressure in the voids, because the water cannot move in or out in response to the tendency for the volume of voids to change."

To summarize: for drained conditions, water is able to flow in or out of the soil rapidly enough in response to the loading conditions that the soil reaches a state of equilibrium and there is no increase of pore water pressures within the soil. For undrained conditions, the water is not able to flow in or out of the soil rapidly enough in response to the loading conditions, resulting in an increase or decrease of the pore water pressures.

Depending on the loading conditions and the permeability of the fill material within the embankment, an engineer could be considering drained or undrained conditions, or both (in the case of a free-draining shell material and impervious core material), in the analysis of the stability of an embankment dam.

Total and effective stresses are defined in Reference 2 as follows:

"Total stress (σ) is the sum of all forces, including those transmitted through interparticle contacts and those transmitted through water pressures, divided by the total area. Total area includes both the area of voids and the area of solid."

"Effective stress (σ') includes only the forces that are transmitted through particle contacts. It is equal to the total stress minus the water pressure (u)." The equation for effective stress is given as: $\sigma' = \sigma - u$.

All of the government agencies discussed in this report have requirements or guidelines regarding the use of total or effective stresses (or a combination thereof) in evaluating the stability of an embankment dam for different loading conditions. These will be presented later in this report.

The shear strength of a soil is a function of the cohesion of the soil (c), the internal angle of friction of the soil (Φ), and the normal stress (σ). The shear stress at failure (S) is expressed by the Mohr-Coulomb failure law as:

$S_u = c + \sigma \tan \Phi$ for a total stress analysis

$S_d = c' + \sigma' \tan \Phi'$ for a effective stress analysis

where c and c' are the cohesion intercepts and Φ and Φ' are the friction angles for the total and effective stress shear strength envelopes, respectively. Figure 1 shows the shear strength envelopes that are developed from Mohr circles for total and effective stresses.

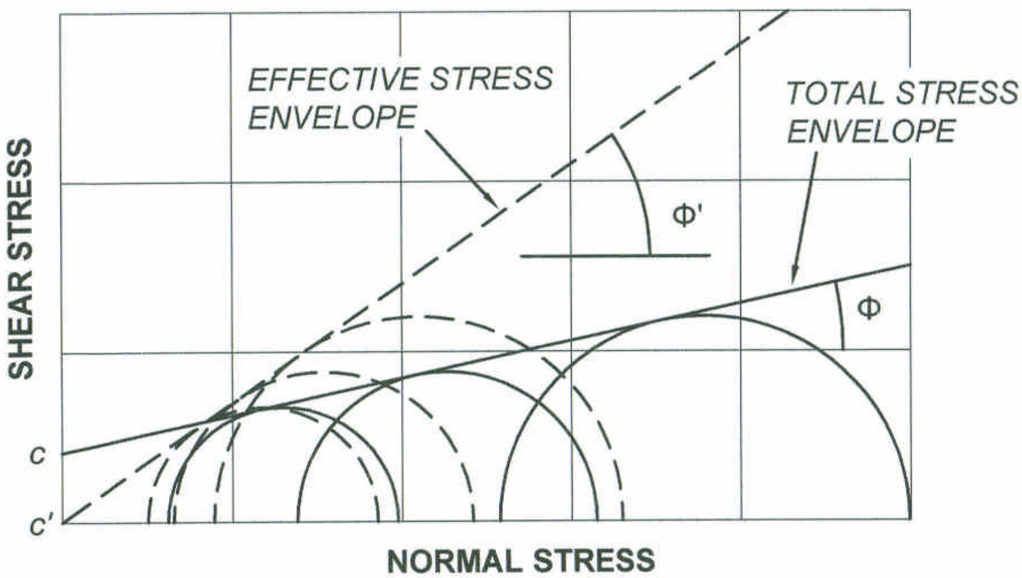

Figure 1. Shear Strength Envelopes for Total and Effective Stresses

3.2 SELECTION OF SHEAR STRENGTHS

In selecting shear strengths to be used for the analysis of slope stability, the respondents use the following methods.

3.2.1 Review of Literature

During the feasibility or preliminary design phase of a project, before a significant amount of field and laboratory testing has been performed, soil strengths are commonly selected from reviewing existing literature, reports, or other data regarding the soils in a specific area. The TVA, for example, has an extensive data base for strength parameters for soils in the Tennessee River Basin. Some consultants use the published NRCS soil maps to identify soil types likely to be present at the site. The NRCS data, however, does not contain information on shear strength parameters.

Numerous published correlations between the type of soil and average shear strengths are available, including Table 2 of the USBR's *Design Standards No. 13, Embankment Dams, Chapter 4* (Reference 3). These correlations are also used to verify the reasonableness of the laboratory test results.

Strengths for rockfill and other coarse material are commonly based on a review of literature since these materials are difficult to test in the field or laboratory.

3.2.2 Field Testing

During the field investigation, disturbed and undisturbed samples are collected of the proposed embankment and foundations materials for laboratory testing. The methods of investigation include borings, test pits, and trenches. Samples are collected through a variety of means, including split-spoon samplers, tube samples (Shelby, Denison, and Pitcher), buckets, and bags.

In situ testing of foundations materials are performed by the following methods:

- Standard Penetration Test (SPT)
- Field Vane Shear Tests
- Cone Penetrometer Tests (CPT)
- Shear Wave Testing
- Pocket Penetrometer Testing of Samples

The methods of investigation are described in numerous geotechnical engineering reference books and will not be described here. Sources that present recommended practices for geotechnical investigations include USACE EM 1110-1-1804, *Geotechnical Investigations* (Reference 14) and the USBR *Earth Manual, Part 1* (Reference 16). There are numerous published correlations between the results of the field tests and the shear strength of the materials, including J. M. Duncan's "Shear Strength Correlations for Geotechnical Engineering" (Reference 4) and most geotechnical engineering reference books.

3.2.3 Laboratory Testing

Laboratory testing is performed to: 1) determine index properties of the soil samples in order to classify the soil materials, and 2) estimate the drained and undrained shear strength properties of the fill materials. Methodologies and procedures for performing the laboratory testing are described in numerous geotechnical reference books and will not be described here. Sources that present recommended practices for laboratory testing include USACE EM 1110-2-1906, *Laboratory Testing* (Reference 15), the USBR *Earth Manual, Part 2* (Reference 17), and ASTM Standards 04.08 and 04.09, *Soil and Rock (I and II)* (Reference 18).

3.2.3.1 Index Properties. The most common index property tests performed for embankment fill materials are: 1) grain size analysis to determine the amount of gravel, sand, silt, and clay in the fill material; 2) Atterberg Limits to determine the plasticity of the soil, and 3) maximum dry density and optimum moisture content. Index properties for foundation soils include: 1) grain size analyses, 2) Atterberg Limits, and 3) *in situ* dry density and moisture contents.

3.2.3.2 Shear Strength Tests. Shear strengths are normally obtained from direct shear tests, unconfined compression tests, and triaxial tests. The shear strengths of embankment materials used in stability analyses are determined from laboratory testing procedures that attempt to duplicate the various loading conditions to which the embankment is expected to be subjected. These testing procedures include:

Unconfined Compression (UC) Test: Unconfined Compression tests can be used to estimate the undrained shear strength (S_u) of saturated, fine-grained foundation materials. This test should be limited to foundation soils classified as CL, CH, ML, MH, or CL-ML. The undrained strength of such foundation soils will generally be conservative for analysis of the end-of-construction loading condition. If the soil contains lenses, triaxial UU tests are recommended rather than UC tests.

Unconsolidated Undrained (UU or Q) Test: For the Unconsolidated Undrained test, the sample is not consolidated prior to testing, and the water content of the soil is not allowed to change either prior to or during testing. Pore water pressures are not measured during testing. This test is normally performed on saturated, fine-grained soils (clays and silts) either as an unconfined compression test or triaxial test. According to USACE EM 1110-2-1902 (Reference 5), "Q test conditions approximate end-of-construction shear strengths of embankments consisting of impervious, or of impervious zones of zoned embankments."

It is recommended that UU tests performed on embankment materials should be performed on samples remolded at the highest water content likely to be encountered during fill placement to represent the lowest embankment fill shear strength. Soils that do not exhibit undrained behavior are not normally tested by this method.

Consolidated Undrained (CU and $\overline{CU}$*, or R and* $\overline{R}$*) Test:* For the Consolidated Undrained test, the sample is saturated and consolidated under confining pressures that approximate field conditions. Pore water pressures during the test are either not measured to determine total stress strength parameters (CU or R); or measured to determine effective stress strength parameters ($\overline{CU}$ or $\overline{R}$). These shear strengths are normally determined by triaxial testing performed on saturated, impervious or semi-impervious soils and approximate the soil conditions experienced during steady-state seepage and rapid drawdown.

Consolidated Drained (CD or S) Test: For the Consolidated Drained test, the sample is saturated and consolidated under confining pressures that approximate field conditions; however, shear stresses are applied slowly enough to allow the dissipation of excess pore pressures during the test. Pore water pressures are not measured.

This test determines effective stress strength parameters and is normally performed on pervious or free-draining soils either by triaxial testing or direct shear testing. CD tests may also be used to measure the shear strength of relatively impervious moist soils to model the strength of materials located above the phreatic line within the embankment.

Laboratory samples of embankment materials should be compacted to the dry density and water contents specified in the design criteria or construction specifications.

3.2.4 Interpreting Test Results

In interpreting the plots of the test results and selecting design shear strength values, a moderate amount of conservatism is justified. A common practice is the use of the "two-thirds" rule, defined in Reference 5 as, "For each embankment zone and foundation layer, design shear strengths should be selected such that two-thirds of the test values exceed the design values." Many respondents also use the "two-thirds" rule. The TVA uses a shear strength where half of the test results lie above the selected line (Reference 6).

On small projects where there are usually only one or two sets of shear strength tests performed, the California DSOD will generally accept design strength obtained by conservatively drawing a line tangent to the Mohr circles. For large projects, the California DSOD recommends using the average or the "two-thirds" rule (Reference 7).

3.2.5 Shear Strengths for Design Loading Conditions

The following subsections present the design loading conditions used by the various government agencies involved in dam design.

3.2.5.1 USACE Practice. It should be noted that USACE EM 1110-2-1902, *Engineering and Design: Stability of Earth and Rock-Fill Dams* (issued in 1970), was revised in 2003 and is now entitled, *Slope Stability.* Since much of the current practice for selecting shear strengths was originally based upon the 1970 version of EM 1110-2-1902 (Reference 5), the loading conditions identified by the USACE are briefly described below:

Case I: End of Construction
Case II: Sudden Drawdown from Maximum Pool
Case III: Sudden Drawdown from Spillway Crest Elevation
Case IV: Partial Pool
Case V: Steady Seepage with Maximum Storage Pool
Case VI: Steady Seepage with Surcharge Pool
Case VII: Earthquake

For Case I, where the embankment is constructed partially or entirely of low-permeability soils placed at water contents higher than the optimum water content after complete consolidation, the USACE selected shear strengths determined from the Q (or UU) envelope since pore pressures will be induced but cannot dissipate quickly enough during construction.

For Cases II and III, the USACE selected a minimum shear strength based on the minimum of the R (CU) and S (CD) envelopes (see Figure 2) for low-permeability and semi-pervious embankment materials. For free-draining materials, the S envelope is used.

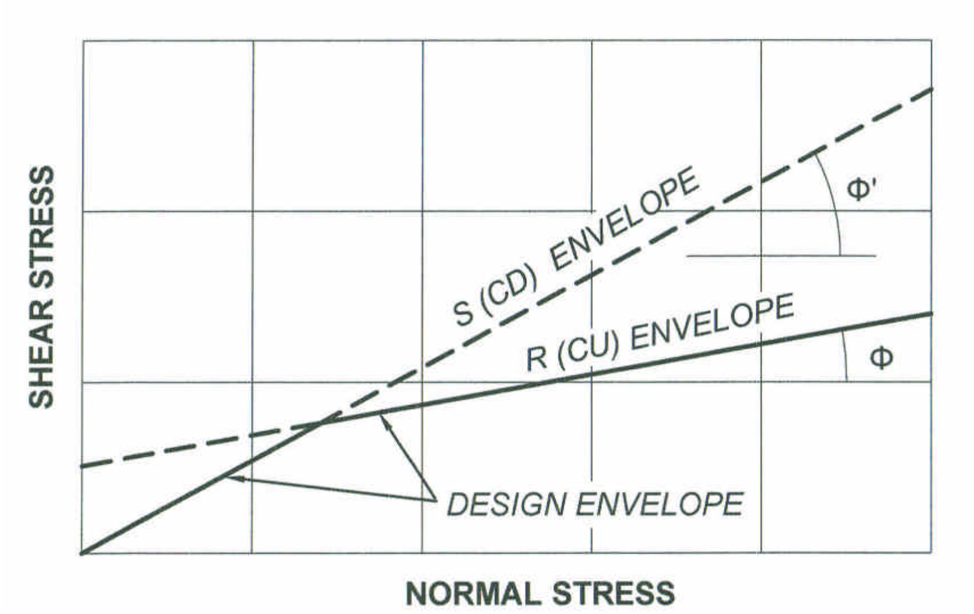

Figure 2. Design Envelope for Sudden Drawdown (USACE)

For Cases IV, V and VI, the design shear strength of impervious materials corresponded to either: 1) a strength envelope midway between the R and S envelopes where the S strength is greater than the R strength, or 2) the S envelope where the S strength is less than the R strength (see Figure 3). For free-draining materials, the S envelope is used.

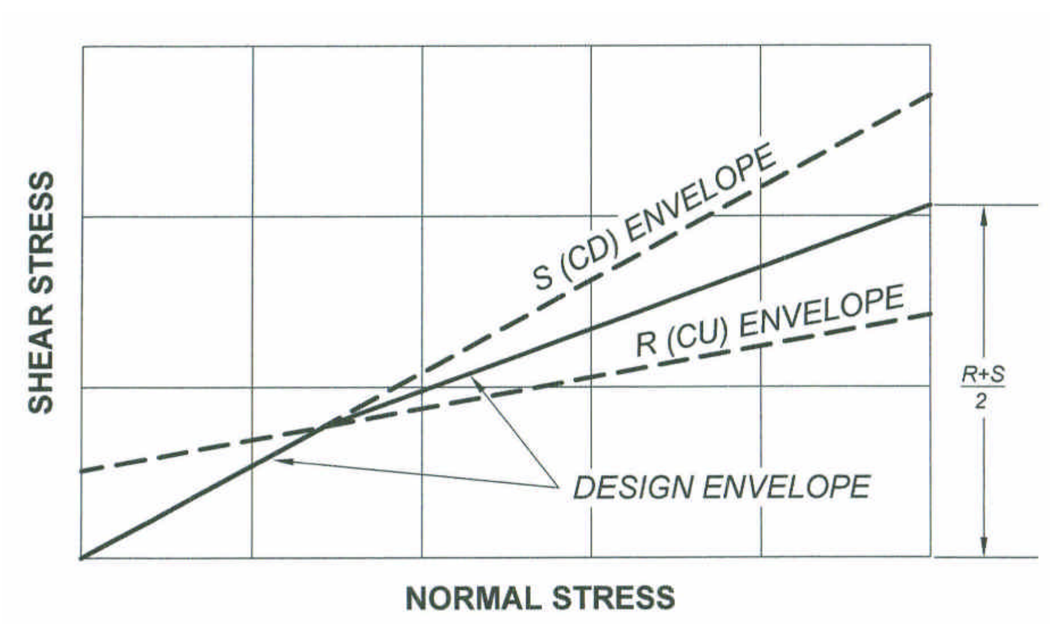

Figure 3. Design Envelope for Steady Seepage (USACE)

For Case VII: Earthquake, the USACE evaluated the most critical failure surface identified for Case VI: Steady Seepage with Surcharge Pool (with the same strength parameters) with an additional seismic loading. Sudden drawdown cases are not analyzed for seismic loading.

In the 2003 revision of EM 1110-2-1902 (Reference 8), the USACE identifies the following static loading conditions:

- During Construction and End-of-Construction: These conditions are analyzed using drained (effective) strengths for free-draining materials and undrained (total) strengths for slow-draining materials. Reference 8 states that "as a rough guideline, materials with values of permeability greater than 10^{-4} cm/sec usually will be fully drained throughout construction. Materials with values of permeability less than 10^{-7} cm/sec usually will be essentially undrained at the end of construction."

- Steady-State Seepage Conditions: All soils are considered to be fully drained in the long-term condition; therefore, drained (effective) strengths are selected for the fill materials. The pool levels considered include: 1) maximum storage pool level that is "maintained long enough to produce a steady-state seepage condition"; 2) intermediate pool levels that "exist over a period long enough to develop steady-state seepage"; and 3) surcharge pool level that is at a temporary level higher than storage pool, but "does not persist long enough to establish a steady seepage condition."

- Sudden Drawdown Conditions. Materials with permeability greater than 10^{-4} cm/sec are assumed to be fully drained; therefore; drained (effective) strengths are used for these materials. For low-permeability materials, a three-stage computation process developed by Lowe and Karafiath and modified by Wright and Duncan (1987) and Duncan, Wright and Wong (1990) is preferred by the USACE. This procedure is presented in Appendix G of Reference 8 and is summarized as follows:

  > "The purpose of the first set of computations (1^{st} stage) is to compute the effective stresses along the shear surface (on the base of each slice) to which the soil is consolidated prior to drawdown. These consolidation stresses are used to estimate undrained shear strengths for the second-stage computation (2^{nd} stage), with the reservoir lowered. The third set of computations (3^{rd} stage) also analyzes stability after drawdown, using the lower of the drained or undrained strength, to ensure that a conservative value of factor of safety is computed."

For the analyses of stability during earthquakes, the USACE is preparing an Engineering Circular, *"Dynamic Analysis of Embankment Dams,"* to provide guidance regarding analyses and design criteria for earthquake loading. At the time this white paper was prepared, the Engineering Circular was in draft form.

Table 1 summarizes the current USACE practice.

3.2.5.2 USBR Practice. The USBR examines the following loading conditions (Reference 3):

- Construction conditions, which include end of construction and, as necessary, partial completion of fill conditions (depending on construction schedule and pore pressures). Effective or total stress strengths can be used.

- Steady-state seepage conditions, which should be analyzed at the reservoir level that controls development of the steady-state phreatic surface. Effective stress parameters are used for this loading condition.

- Operational conditions, which may include:

    1. Evaluating the downstream slope for the maximum reservoir loading where the maximum reservoir elevation is significantly higher than the top of active conservation pool elevation.

    2. Evaluating the upstream slope for rapid drawdown from the top of active conservation water surface to the inactive water surface.

    3. Evaluating the upstream slope for rapid drawdown from the top of active conservation water surface to an intermediate level if there are upstream berms.

    Effective or total stress strengths can be used for operational conditions.

- Unusual conditions, other conditions (where appropriate) that may include:

    1. Plugged or partially plugged internal drainage.
    2. Drawdown due to unusually high water use demands.

    3. Drawdown for the emergency release of the reservoir.

    Effective or total stress parameters can be used for operational conditions.

USBR practice for selecting shear strengths for these loading conditions is summarized in the Table 2 (Reference 3).

3.2.5.3 NRCS Practice. The NRCS identifies the following loading conditions for embankment dams: End of Construction, Rapid Drawdown (from spillway elevation to the crest of the lowest gated or ungated outlet), Steady Seepage, and Steady Seepage with Seismic Forces. NRCS practice for selecting shear strengths is summarized in the Table 3 (Reference 9).

3.2.5.4 FERC Practice. The FERC considers the following loading conditions: End of Construction, Sudden Drawdown from Maximum Pool, Sudden Drawdown from Spillway Crest, Steady Seepage with Maximum Storage Pool, Steady Seepage with

Surcharge Pool, and Earthquake. FERC practice for selecting shear strengths is summarized in the Table 4 (Reference 10).

FERC practice previously allowed the use of the pseudostatic method of analysis in areas of low or negligible seismicity (peak ground accelerations of 0.05g or less). FERC no longer uses a pseudostatic analysis to judge the seismic stability of embankment dams. A Newmark type of deformation is required to determine the post earthquake seismic performance. This will be reflected in a revision to Chapter 4 Embankment Dams of their Engineering Guidelines that is in the process of being issued.

3.2.5.5 TVA Practice. The TVA identifies the following loading conditions for embankment dams: End of Construction, Sudden Drawdown, Steady Seepage with Normal Operating Condition, and Steady Seepage with Flood Surcharge Pool. TVA practice for selecting shear strengths is summarized in the Table 5 (Reference 6).

3.2.5.6 California DSOD Practice. As stated in their response (Reference 7), the California DSOD does not require that dam owners and consultants follow the same approach as the DSOD, and "occasionally depart from our standard approach and requirements if the owner/consultant provides good rationale for doing so." The approach of the California DSOD in selecting shear strengths is summarized in Table 6.

3.2.6 Cohesion

The questionnaire asked "when cohesion is justified and why?" The Table 7 presents the criteria of the federal and state agencies, and responses from consulting firms regarding the inclusion of cohesion in shear strengths for embankment fill materials. Based on the responses, it is a common practice to include cohesion for impervious or semi-pervious fine-grained soils.

CHAPTER 4 — STATIC ANALYSIS OF EMBANKMENT STABILITY

4.1 LIMIT EQUILIBRIUM METHODS OF SLOPE STABILITY ANALYSIS

The most common two-dimensional methods of analysis of the static stability of embankments use the limit equilibrium method. The limit equilibrium method assumes the following five conditions (Reference 12):

1. "Each point within the soil mass must be in equilibrium…
2. The Mohr-Coulomb failure condition must not be violated at any point…
3. The strains that occur must be related to the stresses through a stress-strain relationship suitable for the soil.
4. The strains that occur at each point must be compatible with the strains at all surrounding points.
5. The stresses within the soil must be in equilibrium with the stresses applied to the soil."

Common stability analysis procedures that use the limit equilibrium method are the Ordinary Method of Slices, Modified Bishop, Janbu, Spencer, and the Morgenstern-Price method. As discussed by FERC in Reference 10, "different procedures use different assumptions. Some methods do not satisfy all conditions of equilibrium, such as moment equilibrium or vertical and horizontal force equilibrium." Table 8 presents the equilibrium conditions satisfied by the different stability analysis procedures.

4.2 TYPES OF POTENTIAL FAILURE SURFACES EVALUATED

Failure surfaces within embankment dams fall into the following three categories:

1. Circular
2. Non-circular, or Wedge
3. Infinite Slope

In general, the respondents agree that fill embankments will most often be analyzed using circular failure surfaces. Slope stability computer programs can quickly search for the most critical failure surfaces within a fill embankment.

Non-circular failure surfaces are used where there are weak zones in either the embankment or foundation. Examples include rock foundations with horizontal or nearly horizontal weak clay seams, alluvium underlying a dam embankment, the interface between embankment zones with significant strength differences, or potentially liquefiable layers within the embankment.

The Infinite Slope method is usually used to evaluate the near-surface stability of saturated slopes with seepage. This is generally a concern for granular materials with low cohesion.

4.3 LOCATION OF POTENTIAL FAILURE SURFACES FOR ANALYSES

Locating the potential failure surfaces within an embankment takes consideration and experience. Factors include the zoning of embankment materials, fill strengths, and the location of the phreatic surface. As stated in Reference 2, "Locating the critical circle requires a systematic search in which the center point of the circle and its radius are varied. Such searches are usually performed using a computer program in which the search is automated."

In Reference 3, the USBR lists slip surfaces that should be examined:

A. "Slip surfaces that may pass through either the fill material alone or through the fill and the foundation materials and which do not necessarily involve the crest of the dam.

B. Slip surfaces as in preceding paragraph A. that do include the crest of the dam.

C. Slip surfaces should be examined which pass through major zones of the fill and the foundation.

D. Slip surfaces that involve only the outer portion of the upstream or downstream slope. In this case, the infinite slope analysis may be appropriate for cohesionless materials."

One respondent stated that the "most severe case would entail a failure surface cutting the impervious element of the dam at or below the water, so that the reservoir would be breached" (Reference 13).

A designer should not just accept, however, what the computer program determines is the most critical slip surface from its search pattern. "In some cases the slip surface with the minimum factor of safety may not be the slip surface of greatest interest… failure along the shallow surface in the embankment would consist simply of raveling of material down the slope and might, at the most, represent a maintenance problem. In contrast, failure along the deeper surface might require reconstruction of the embankment." (Reference 2).

Verification of the results of the slope stability program "should be commensurate with the level of risk associated with the structure and should include one or more of the following methods of analyses using" (Reference 8):

1. Graphical (force polygon) method.

2. Spreadsheet calculations.

3. Another slope stability computer program.
4. Slope stability charts.
5. Stability verification performed by different personnel.

4.4 RECOMMENDED FACTORS OF SAFETY

The factor of safety (FS) is calculated as the ratio of the total available shear strength (or resistance) available (s) along a failure surface to the total stress (or driving force) mobilized (τ) along the failure surface:

$$FS = s \div \tau$$

For stability analyses of embankment dams, the recommended factors of safety will vary with loading conditions. Long-term loading conditions (i.e., steady seepage) require higher factors of safety while short-term loading conditions (i.e., rapid drawdown) will require lower factors of safety. Presented in Table 9 is a list of federal and state agencies and their recommended criteria for factors of safety for the different loading conditions.

Embankment dams for pumped storage projects may require special consideration because the upstream slope is experiencing rapid drawdown loading conditions on a frequent basis. One consultant recommends minimum factors of safety of 1.5 for an upstream slope under rapid drawdown conditions and 1.1 for an upstream slope under rapid drawdown conditions with earthquake loading (Reference 11). The USACE recommends minimum factors of safety of 1.4 to 1.5 for the upstream face where rapid drawdown is a routine operating condition (Reference 8), and goes on to recommend that "if consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered."

Table 1. USACE Practice

Design Condition		**Shear Strength Parameters**
During Construction and End of Construction	Free-draining soils	Drained (effective) strengths[1]
	Low-permeability soils	Undrained (total) strengths[2]
Steady-State Seepage Conditions		Drained (effective) strengths[1]
Sudden Drawdown Conditions	Free-draining soils	Drained (effective) strengths[1]
	Low-permeability soils	1st stage computation: Drained (effective) strengths[1] 2nd stage computation: Undrained (total) strengths[2] 3rd stage computation: Lower of drained and undrained strengths

Notes: 1) Effective stress parameters are obtained from consolidated-drained (CD, S) tests or consolidated-undrained (CU, R) tests with pore pressure measurements.

2) Total stress parameters can be obtained from unconsolidated-undrained (UU, Q) tests or estimated from consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the R or total stress envelope and associated c and Φ from CU/R tests should not be used.

Table 2. USBR Practice

Loading Condition	**Stress Parameters**	**Shear Strength Tests**
End of Construction	Effective stress parameters when pore pressures can be monitored during construction. Total (undrained) stress parameters when laboratory pore pressures approximate expected field pore pressures.	$\overline{\text{CU}}$ tests for clays and silts; CD tests for free-draining materials. UC and UU tests for fine-grained soils.
Steady-State Seepage – Active Conservation Pool	Effective stress parameters.	$\overline{\text{CU}}$ tests for clays and silts; CD tests for free-draining materials.
Operational Conditions – Maximum Pool Level	Effective or total stress parameters.	$\overline{\text{CU}}$ tests for impervious and semi-pervious materials; CD tests for free-draining materials.
Operational Conditions – Rapid Drawdown from Normal or Maximum Pool	Effective or total stress parameters.	$\overline{\text{CU}}$ tests for pervious and semi-pervious materials; CD tests for free-draining materials. Shear strengths for total stress analysis to be based on minimum of combined CD and CU shear strength envelopes.
Unusual	Effective or total stress parameters.	$\overline{\text{CU}}$ tests for pervious and semi-pervious materials; CD tests for free-draining materials. Shear strengths for total stress analysis to be based on minimum of combined CD and CU shear strength envelopes.

Table 3. NRCS Practice

Design Condition		**Shear Strength to be Used**
End of Construction	Impervious materials at water contents at the highest probable placement water content, or at saturation for foundations soils.	UU test.
	Pervious embankment and foundation materials.	$\overline{CU}$ or CD tests.
Rapid Drawdown		Use lowest shear strength from composite envelope of CU and $\overline{CU}$.
Steady Seepage	Impervious soils	For CD>$\overline{CU}$, midway between $\overline{CU}$ and CD. For CD<$\overline{CU}$, use CD envelope.
	Free-draining soils	CD test.
Steady Seepage with Seismic Forces		Use: 1) 80 percent of CU shear strength for impervious soils and 2) CD shear strength for pervious soils or those that will not develop pore pressures from loading.

Table 4. FERC Practice

Loading Condition	Type of Analysis	Shear Strength Tests
End of Construction	Total Stress	Q test for homogeneous dams or zones of impervious materials; S test for pervious materials.
Sudden Drawdown	Total Stress	Perform R test to obtain undrained (total) shear strength values; S test for free-draining materials.
	Effective Stress	Perform $\overline{R}$ test to obtain drained effective shear strength values; S test for free-draining materials.
Steady Seepage	Effective Stress	For impervious materials, use composite of $\overline{R}$ and S tests; For free draining materials, use S test.
Earthquake (Pseudostatic)[1]	Effective Stress	For impervious materials, use composite of $\overline{R}$ and S tests; For free draining materials, use S test.

[1]FERC requires a Newmark type of deformation analysis along the critical failure plane.

Table 5. TVA Practice

Loading Condition	Type of Analysis	Shear Strength Tests
End of Construction	Total Stress	Q test for impervious materials; S test for free draining materials.
Sudden Drawdown	Total Stress	R test.
Normal Operating Condition	Total Stress	Use composite of R and S tests.
Flood Surcharge Pool	Total Stress	Use composite of R and S tests.

Table 6. California DSOD Practice

Loading Condition	**Type of Analysis**	**Shear Strength Test**
Post Construction	Total Stress	UU test.
Steady Seepage (Full and Partial Pools)	Effective Stress	$\overline{CU}$ test.
Sudden Drawdown	Total Stress	CU test.
Earthquake (Pseudostatic)	Total Stress	CU test.

Table 7. The Use of Cohesion

Agency/Company	**Criteria Regarding Cohesion**
USBR	Use cohesion for fine grained soils.
NRCS	Significant cohesion values are commonly measured on total stress CU tests on clay soils. Use minimum value of cohesion from composite envelope of CD and CU tests.
TVA	Cohesion used in impervious and semi-pervious materials; no cohesion for free draining material.
California DSOD	Use minimum value of cohesion from composite envelope of CU and $\overline{CU}$ tests. Common practice is to curve the low end of the strength envelope into the origin or a small cohesion value.
Consultants	1. Will use cohesion when the data indicates that it is appropriate. 2. Cohesion is generally used for CU strengths; generally not for CD strength.

Table 8. Equilibrium Conditions Satisfied[1]

Procedure	**Overall Moment**	**Individual Slice Moment**	**Vertical Force**	**Horizontal Force**
Ordinary Method of Slices	Yes	No	No	Yes
Modified Bishop	Yes	No	Yes	No
Janbu	Yes	Yes	Yes	Yes
Spencer	Yes	Yes	Yes	Yes
Morgenstern – Price	Yes	Yes	Yes	Yes

[1]From FERC Chapter IV, Embankment Dams, Table 2, (Reference 10).

Table 9. Factors of Safety for Embankment Dams

Agency	Loading Condition	Stress Parameter	F.S.
USACE	During Construction and End of Construction	Total and Effective	1.3
	Long-term (Steady seepage, max. storage pool, spillway crest or top of gates	Effective	1.5
	Max. Surcharge Pool	Effective	1.4
	Sudden Drawdown from Max. Surcharge Pool	Total and Effective	1.1
	Sudden Drawdown from Max. Storage Pool	Total and Effective	1.3
	Sudden Drawdown when Routine Operating Condition (Pumped storage facility	Total and Effective	1.4-1.5
USBR	End of Construction – Pore pressures in embankment and foundation with laboratory determination of pore pressure and monitoring during construction.	Effective	1.3
	End of Construction – Pore pressures in embankment and foundation with no laboratory determination and no monitoring during construction.	Effective	1.4
	End of Construction – Pore pressures in embankment only with or without field monitoring and no laboratory determination.	Effective	1.3
	End of Construction	Undrained (Total)	1.3
	Steady-State Seepage from Active Pool	Effective	1.5
	Operational – Max. Pool Level	Effective or Undrained	1.5
	Operational – Rapid Drawdown from Normal Pool	Effective or Undrained	1.3
	Operational – Rapid Drawdown from Max. Pool		1.3
	Unusual		1.2

Table 9. Factors of Safety for Embankment Dams (continued)

Agency	Loading Condition	Stress Parameter	F.S.
NRCS	I. End of Construction	Total for impervious; effective for pervious	1.4
	II. Rapid Drawdown	Composite	1.2
	III. Steady Seepage – Normal Pool	Composite	1.5
	IV. Steady Seepage with Earthquake	Total	1.1
FERC	End of Construction	Total	1.3
	Sudden Drawdown from Max. Pool	Effective and Total	1.1
	Sudden Drawdown from Spillway Crest	Effective and Total	1.2
	Steady Seepage – Max. Storage Pool	Effective and Total	1.5
	Steady Seepage – Surcharge Pool	Effective and Total	1.4
	Earthquake – Steady Seepage	Effective and Total	> 1.0
TVA	End of Construction	Total	1.3
	Sudden Drawdown	Total	1.2
	Steady Seepage – Normal Operating Condition	Total	1.5
	Steady Seepage – Flood Surcharge Pool	Total	1.25
California DSOD	Post Construction	Total	1.25
	Steady Seepage (Full Pool)	Effective	1.5
	Steady Seepage (Partial Pool)	Effective	1.5
	Sudden Drawdown	Total	1.25
	Pseudostatic	Total	1.1

REFERENCES

1. Van Klaver, R. (1997). "NRCS Practice on Static Strength and Analyses," Natural Resources Conservation Service, correspondence.

2. Duncan, J. M. and Wright, S. G. (2005). *Soil Strength and Slope Stability,* Wiley, New Jersey.

3. USBR (1987). *Design Standards No. 13, Embankment Dams, Chapter 4. Static Stability Analyses;* available from the National Technical Information Service.

4. Duncan, J. M., Horz, R. C. and Yang, T. L. (1989). "Shear Strength Correlations for Geotechnical Engineering," Virginia Tech, Department of Civil Engineering.

5. USACE (1970). *Engineering and Design: Stability of Earth and Rock-Fill Dams,* Engineering Manual 1110-2-1902, Department of the Army, Corps of Engineers, Washington, D. C.

6. Hall, W. D. (1997). "Analyses and Static Strength of Embankment Materials, TVA Design Approach and Criteria," Tennessee Valley Authority, correspondence.

7. Persson, V. H. (1997). "Response to Analysis and Static Strength of Embankment Materials," California Department of Water Resources, Division of Safety of Dams, correspondence.

8. USACE (2003). *Slope Stability,* Engineering Manual 1110-2-1902, Department of the Army, Corps of Engineers, Washington, D. C.; available at www.usace.army.mil/inet/usacoe-docs/eng-manuals/em1110-2-1902/entire.pdf

9. NRCS (2005). *Technical Release No. 60, Earth Dams and Reservoirs,"* Natural Resources Conservation Service; available at www.info.usda.gov/CED/ftp/CED/TR_210_60_Second_Edition.pdf

10. FERC (1991). *Chapter IV, Embankment Dams,* Federal Energy Regulatory Commission; available at http://www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide/chap4.PDF

11. Kleiner, D. E. (1997). "Report – Analysis and Static Strength of Embankment Materials," Harza Engineering Company (now part of MWH), correspondence.

12. Lambe, T. W. and Whitman, R. V. (1969). *Soil Mechanics,* Wiley, New York.

13. Jansen, R. B. (1997). "Analyses and Static Strength of Embankment Materials," correspondence.

14. USACE (2001). *Geotechnical Investigations,* Engineering Manual 1110-1-1804, Department of the Army, Corps of Engineers, Washington, D. C.; available at www.usace.army.mil/inet/usacoe-docs/eng-manuals/em1110-1-1804/entire.pdf

15. USACE (1970, revised 1986). *Laboratory Testing,* Engineering Manual 1110-2-1906, Department of the Army, Corps of Engineers, Washington, D. C.; available at www.usace.army.mil/inet/usacoe-docs/eng-manuals/em1110-2-1906/entire.pdf

16. USBR (1998). *Earth Manual, Part 1*; available from the National Technical Information Service and at www.usbr.gov/library/BRreclamation.html

17. USBR (1990). *Earth Manual, Part 2*; available from the National Technical Information Service

18. ASTM 04.08 *Soil and Rock (I)* and 04.09 *Soil and Rock (II).*

QUESTIONNAIRE RESPONDENTS

Adhya, Ron; Pacific Gas & Electric; March 17, 1997.

Bingham, William B.; Vice President; Gannett Fleming; April 1, 1997.

Blohm, H. L; Vice President; Montgomery Watson; January 16, 1997.

Chugh, Ashok; U. S. Bureau of Reclamation; March 7, 1997.

Collins, Sam F; Executive Vice President and General Manager; Sabine River Authority of Texas; February 13, 1997.

Duncan, James Michael: University Distinguished Professor; Virginia Polytechnic Institute; January 10, 1997.

Findlay, R. Craig; Director of Engineering; Northrop, Devine & Tarbell, Inc.; March 16, 1997.

Gupta, Don; Black & Veatch; March 14, 1997.

Hall, W. Hall, Manager, Fossil and Hydro Engineering; Tennessee Valley Authority; March 5, 1997.

Harlan, Richard C.; May 6, 1996.

Jansen, Robert B.; February 24, 1997.

Kleiner, David E.; Chief Geotechnical Engineer; Harza Engineering Company; August 19, 1997.

Persson, Vernon H.; Chief, Division of Safety of Dams, California Department of Water Resources; January 3, 1997.

Rizzo, Paul; Paul C. Rizzo Associates, Inc.; April 1, 1997.

Sharma, Ram P.; Morrison Knudsen Corporation; April 2, 1997.

Van Klaven, Richard, Director, Conservation Engineering Division; Natural resources Conversation Service; February 17, 1997.

Walz, Arthur H. Jr.; Chief, Geotechnical and Materials Branch; U. S. Army Corps of Engineers; January 6, 1997.