CHAPTER IV

EMBANKMENT DAMS

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4-1 Purpose and Scope

4-1.1 General

The guidelines presented in this chapter provide staff engineers with recommended procedures and criteria to be used in reviewing and evaluating the safety of existing and proposed earth and rockfill (embankment) dams. The review performed by staff engineers will be conducted to ensure that all decisions, methods, and procedures performed by licensees/exemptees, or their consultants, are sound regarding dam safety, and to ensure that the Commission's Dam Safety Program objectives as stated in Part 12 of the Commission's Regulations are consistent with accepted, up-to-date state-of the-art procedures (the term licensees also refers to applicants for license where appropriate).

The evaluation of safety of both new and existing embankment dams presents special and unique problems. Existing dams may prove difficult to analyze especially in those instances where the dam was designed before the development of modern design and construction technology or where adequate records are not available. Even for a relatively new dam where records are extensive, evaluation can be cumbersome for the following reasons: (a) various levels of completeness of records, (b) different site conditions, (c) varying degrees of quality in design and construction, and (d) differing depth of evaluation required for each dam.

One of the main objectives set forth in this chapter is to provide systematic state of the art procedures for performing staff evaluations. It should be recognized however that the various calculation procedures discussed herein are illustrated for a given mode of failure for an embankment dam subjected to a given loading. Even though it is important to correctly utilize the most current procedures to assess the degree of safety for a given mode of failure, it has been established by study of case histories of failures that the cause of many failures have been the result of omissions in considering all possible modes of failure during design. The lack of foresight to provide appropriate zoning to preclude certain modes of piping failure has also resulted in failures. Such appropriate intelligent zoning provisions may not even require any engineering calculations. The selection of the best zoning to control seepage for a given dam and foundation condition does require a certain degree of alertness and an informed knowledge of the lessons learned from precedent.

Failures have also resulted because of the use of inappropriate shear strengths for static loads and the lack of appreciation of liquefaction potential during earthquakes for dams resting in part on alluvial foundations or for dams composed of hydraulic fill materials.

In the following section of this guideline, the causes of recent failures of embankment dams are reviewed to emphasize the most important design considerations for embankment dams. Incidents of near failure or inadequate performance are also cited to guide the reader in developing a checklist of important design considerations for use in embankment dam design and inspection. A knowledge of precedent is a necessary tool in the design and inspection of embankment dams. The calculation procedures given herein on various topics should not be used by themselves without considering the precedents of key embankment dam failures and the precedents of key dam designs which have performed successfully. The knowledge and use of key precedents supplemented by the calculation procedures given herein to arrive at an appropriate design of an embankment dam represents a processes in which the results of engineering calculations are tempered with judgement based on observations of the performance of other embankment dams. This is the general way in which the art of embankment dam design has developed in the past and is the appropriate way in which it can be improved in the future.

4-1.2 Failures and Near Failure Incidents of Embankment Dams

4-1.2.1 Causes of Failures

A failure for the purpose of this discussion is an uncontrolled release of the reservoir due to a breach of the embankment damn. An incident may be a near failure which was averted by some combination of remediation and controlled reservoir lowering, most usually as a result of keen observations of increased and uncontrolled seepage by inspectors.

The most common causes of embankment dam failures are:

1) Piping by internal erosion of fine-grained soils from the embankment dam. Piping through the foundations or abutments of the embankment dam. Piping along conduits constructed through the embankment dam.

2) Overtopping of the dam and subsequent breach by erosion due to overtopping. Overtopping has occurred due to inadequate spillway capacity and due to improper operation of spillway gates. Overtopping and a breach has also occurred on an upper reservoir of a pumped storage project without a spillway.

Failures have also resulted from the following causes which are much less common.

3) Loss of shear strength due to high pore pressures and in some cases liquefaction of loose saturated granular materials in the foundation or embankment during earthquakes.

4) High pore pressures in downstream shell due to inadequate drainage resulting in instability and failure of the downstream shell.

5) Embankment failure due to sliding on clay-shale foundations.

4-1.2.2 Examples of Failures or Inadequate Performance Due To Piping and Internal Erosion

Baldwin Hills Dam in the Los Angeles area was a 71 m high homogeneous earthfill embankment dam. The impervious member was a 5 ft thick compacted earth lining which was constructed on an asphaltic membrane. The dam failed by piping on December 14, 1963. Although there was a pea gravel and clay tile drainage system under the bottom of the reservoir, there was not a drain or filter system between the upstream slope of the embankment and the downstream slope of the homogeneous embankment. It is possible that the distortions of the embankment due to the differential settlements in the area due to oil extraction was a factor in cracking the lining which resulted in uncontrolled seepage downstream of the lining on the upstream slope of the embankment because there was no downstream drain or filter zones in the embankment.

A near failure incident was recorded at the USBR Fontenelle Dam in 1965. Fontenelle Dam is 50 m high and developed seepage at the right abutment as the reservoir reached maximum level. The area was grouted and observed. Several months later, the seepage had developed into a leak of about 11 million gallons per day and washed a hole in the downstream face of the dam that was 80 ft wide, 150 ft high and 60 ft deep. The reservoir was rapidly lowered and a breach of the dam was avoided. The dam has since been remediated and a cutoff wall was constructed.

In February 1975 the Walter Bouldin Dam in Alabama failed. The 165 ft high embankment dam just to the left of the powerhouse breached. As described by Leps (1988) this location is where a cretaceous fine sandy silts could have piped undetected into the tailrace channel from seepage lines in the foundation of the left embankment dam as there was no cutoff to bedrock beneath the left embankment dam immediately adjacent to the left side of the tailrace channel. These seepage lines were not filtered in the design and could have exited into the tailrace channel below water level where the piping would have been uninspectable. Two independent engineering panels wrote reports on the possible cause of this failure. One panel indicated that the most probable cause was an upstream slope stability failure. Another group indicated that it was most probably piping in the unprotected cretaceous fine sandy silts, in the left embankment dam foundation. Leps, 1988 makes a compelling case for the piping mode of failure.

On June 5, 1976 the 126 m high Teton Dam of the USBR failed during first filling, which had been initiated in October 1975. The failure of Teton Dam was a clear case of piping because the silt core in the rock cutoff trench in the right abutment was directly placed against open jointed rhyolite without a filter between the silt and the jointed rock. This case history has been described many times. Leps, 1988, and Sherard, August 1983. Even this very clear case of piping has been described by Hilf, August, 1995, and Fucik, August, 1985, as due to other causes. These interpretations cited above indicate how misinterpretations of case histories has the potential to set back the state of the art if there is only one small group commenting on a case history where a failure occurred.

On October 30, 1979 the Martin Co. Embankment Dam impounding a cooling pond for the Florida Power and Light, Martin Co. oil fired power plant failed by piping. This failure was 18 months after filling and occurred just 2 days after a lowering of the water level in a canal just downstream of the embankment. The canal water lowering increased the gradients through the foundation of the dam which was founded on fine sands. The dam did not have a cutoff and had a homogeneous fine sand crossection with no filter-drain system. The dam had an upstream soil cement member to act as a rip-rap to protect the sand embankment from erosion (Swiger, Hendron, Shae and Smertmann, July, 1980). The sands in the foundation could have piped to a borrow pit immediately downstream of the toe of the embankment after the tailwater was lowered because there was no filter placed on the sides of the borrow pit to prevent migration of the fine sand foundation materials and the seepage outlet could have been below the borrow pit water surface where evidence of transported materials could not have been observed during inspections.

In addition to the above cases, there have been two recent cases of piping of blanket materials into natural alluvium which served as a foundation for the blanket. In both cases the blankets were used to replace a cutoff and there was no filter placed in between the blanket and the natural alluvial material on which the blankets were placed.

In one case, Tarbela Dam, a 40 foot thick blanket was placed on top of river alluvium and there was 450 feet of head which could result in a vertical gradient of about 10 through the blanket without a filter protecting the blanket. When the reservoir was lowered because of other problems on the project hundreds of sinkholes in the blanket were observed and had to be monitored and filled with well graded materials by dumping from barges during the next filling. This case is discussed by Lowe, 1998.

Sinkholes were also observed at the Ludington Pumped Storage Project where a two foot thick blanket was used over natural glacial fluvial materials without a filter between the blanket and the foundation. For this case the vertical gradients through the blanket were on the order of 50. These two cases represent the omission of a filter on the underside of the blanket and did not involve the details of filter criteria.

4-1.2.3 Examples of Failures by Overtopping

On the Pedro Rivér, San Paulo, Brazil, two earth dams (Euclides de Cunha and Armando Salles de Oliviera) were overtopped and destroyed on January 19, 1977 (Water Power, 1977). An area of 250 km^2 downstream of the damns was inundated with considerable loss of property. It is reported that the 10,000 year flood developed in the basin.

Buffalo Creek Dam in West Virginia failed on February 26, 1972 by overtopping resulting from inadequate spillway capacity, and 118 people were killed. The dam was built from mine wastes.

4-1.2.4 Examples of Failure Due to Inadequate Static Stability of Downstream Shell

On Auguyst 27, 1993 a Concrete Faced Dam failed in China (Gouhou Dam) one day after the reservoir level reached the top of the slab. The failure was due to failure of the gravel shell. Although many CFRD's have been built from freely draining rock fill and clean gravels with no stability problems with slopes ranging from 1.3 to 1.6:1, this particular dam had a 1.5:1 downstream slope, but it was constructed with sandy gravels with about 40% of the particles finer than 5 mm. With the leakage through the face and perimeter joint, the dirty shell materials were not pervious enough to conduct the flow at low gradients and a phreatic surface raised high enough in the shell that the normal CFRD slopes could not be maintained, and the dam failed. The failure should have not been a surprise.

4-1.2.5 Examples of Failures and Near Failure Caused by Loss of Shear Strength Due to Liquefaction Under Earthquake Shaking

On February 9, 1971 a strong earthquake (6.6 Richter Magnitude) occurred with an epicenter about 8 miles northeast of Lower San Fernando Dam, California. The embankment, with a height of 142 feet was originally constructed in 1921 as a semi-hydraulic fill. The earthquake caused the development, towards the end of the earthquake shaking, of very high pore pressures in an extensive zone of hydraulic fill near the base of the embankment and upstream of the clay core so that much of this soil was in a liquefied or very low strength condition. A comprehensive dynamic analysis of the failure has been described by Seed, et al (1975). Fortunately, the reservoir storage at the time of the event was only slightly more than half full so that no water overtopped the dam and no leaks developed. Had the reservoir been filled only several feet higher, a major catastrophe might have resulted in the densely populated downstream communities.

The Sheffield Dam failed near the end of an earthquake near Santa Barbara, California in 1925, as a result of a slide of the entire embankment on a liquefied layer covering essentially the entire base; in effect, the embankment was pushed downstream by the water pressure acting on the upstream face (Seed, et al. 1969). For the conditions at the time of failure (Seed, H. B., 1987) concluded that the residual strength of the liquefied soil when sliding occurred was about 50 psf.

A study performed by the U.S. Army Corps of Engineers (1949) concluded that sliding occurred on a liquefied layer of silty sand having a relative density of about 40%. This would correspond to a value of $(N_1)_{60}$ for a clean sand of about 6 to 8.

4-1.2.6 Embankment Failure Due to Sliding on Clay-Shale Foundations

It must be remembered that the critical surface for stability may not always be contained within the materials of the compacted embankment dam; but, there may be preferred weak planes in the foundation which may control, particularly when the foundations are horizontally bedded plastic clay shales.

In 1963 the Corps of Engineers had such a stability failure at Waco Dam during construction. The dam was placed on the Pepper Shale Formation and failure took place on a bedding plane with zero cohesion and a low effective angle of shearing resistance. Fortunately the event occurred before the reservoir was filled.

In 1971 a berm was deemed necessary at Standley Lake Dam at Westminster, Colorado because of downstream movements of the slope. The foundation of the dam is on cretaceous shales and the movements stopped with the addition of the berm. This case is certainly not a failure, but a case where the Factor of Safety was marginal and spreading was occurring on weak beds in the horizontally bedded Cretaceous clay shale formations.

4-1.2.7 Example of Failure of Hydraulic Fill Dams Under Static Conditions During Construction

A major slide occurred in the upstream shell of the Fort Peck Dam, near the end of construction of this hydraulic fill structure in 1938 (U.S. Army Corps of Engineers 1939; Casagrande 1965). From the configuration of the slide material after failure, Bryant, et al. (1983) concluded that the residual strength of the liquefied sand was about 240 psf. Other studies indicate a pre-sliding driving stress of about 700 psf; a reasonably conservative value is probably about 600 psf.

It is believed that, in this case, the slide occurred due to liquefaction of sand in the foundation. Studies made by the U. S. Army Corps of Engineers, both soon after the slide occurred and during a re-evaluation of the stability of the dam in 1976 (Marcuson and Krinitzsky 1976), indicate that the relative density of the sand was probably about 45 to 50%. This would correspond to a value of $(N_1)_{60}$ for a clean sand of about 12.

A liquefaction-type slide occurred in the upstream shell of the Calaveras Dam as it approached a height of 200 ft in 1918 (Hazen 1918). The dam was a hydraulic fill structure, and it was subsequently reconstructed using rolled fill construction. From the configuration of the slide mass, the residual strength of the liquefied sand is estimated to be about 750 psf, and tests performed in recent years show that the SPT $(N_1)_{60}$ value for the hydraulic sand fill in the original structure was probably about 12.

4-1.3 Review of New or Existing Dams

4-1.3.1 Review of Existing Dams

The review of existing dams will generally not be as detailed as the procedures involved in the design of new dams. Some critical areas may require detailed review. Primarily, the review is intended to evaluate the design, analysis and observed behavior to ensure that safe and adequate embankment dams were constructed. The licensee's or its consultant's investigations and evaluations should be examined to determine if all areas of importance were considered and that appropriate design criteria have been used.

Existing dams should 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. For existing dams, an independent analysis of the embankment stability or adequacy need not necessarily be performed by staff. The data presented by the licensee should be reviewed to determine if they appear reasonable and if the latest information has been considered. The criteria used by the licensee or its consultant should be consistent with any changed conditions discovered during onsite examinations such as loadings, increased seepage, increased pore pressures in the dam or the foundation, erosion etc.

4-1.3.2 Review of New or Proposed Dams

For proposed dams, an analysis of the stability and adequacy is required unless specifically exempted by the Commission. The methods and procedures used in the evaluation of any embankment should be consistent with the latest, accepted state-of-the-art methods and criteria, and with guidance contained in this chapter of the guidelines.

For proposed or new dams, the licensee will be required to submit a design report in accordance with the Commission's Regulations. His report will be thoroughly examined to determine if all appropriate design criteria have been met.

4-1.3.3 Important Considerations to be Evaluated

During the investigation and evaluation for both proposed and existing dams, important areas to consider are as follows:

The embankment must be safe against overtopping by wave action for all operational conditions and the inflow design flood conditions.

The slopes must be stable during all conditions of reservoir operations, including rapid drawdown, if applicable.

Seepage flow through the embankment, foundation, and abutments must be controlled so that no internal erosion (piping) takes place and there is no sloughing in areas where seepage emerges.

The embankment must not overstress the foundation. Sliding stability of clay-shale foundations must be evaluated.

Embankment slopes must be acceptably protected against erosion by wave action and from gullying and scour against surface runoff.

The embankment, foundation, abutments and reservoir rim must be stable and must not develop unacceptable deformation under earthquake conditions.

The potential for liquefaction of loose alluvial foundations or old hydraulic fill embankments under earthquake must be considered.

Embankment deformations during earthquake shaking; and, post earthquake stability must also be considered.

4-1.4 References

Criteria and methods of evaluation and analysis used in reviewing licensee's reports should be based on the guidelines given herein and on criteria and procedures established in literature published by such agencies as the Corps of Engineers, U. S. Bureau of Reclamation, or other recognized engineering references. Selected references are listed in Section 4-8.

4-2 Sources of Data and Information

To properly evaluate all information and data presented in the licensee's design report or the licensee's existing dam, various available FERC reports should be reviewed. Available reports include:

- Pre-license Inspection Reports of existing dams and/or Site Inspection Reports of proposed damsites
- Operation Reports
- Construction Reports
- Independent Consultant's Safety Inspection Reports

One or more of the reports listed above should typically be available for licensed projects. If a license has not previously been issued, the staff engineer performing the review should refer to the Pre-license Inspection Report prepared by the staff engineer responsible or the project in the Regional Office.

For existing dams, additional data may be available from the facility owner, previous owners, state or local agency if the facility is a publicly owned project, and from the state agency responsible for dam safety, such as Department of Water Resources, Department of Environmental Resources, Division of Dam Safety or Department of Natural Resources. Technical information may also be available from Corps of Engineers Phase I Inspection

Reports of public or private entities having impounding structures upstream or downstream of the facility.

For proposed dams, the source of information will generally be the licensee and/or its consultants and engineers. For all proposed dams, the licensee will be required to provide staff with those data necessary to evaluate whether the design of the structure is safe and adequate.

Data that may be available from the sources referenced should include:

- Summary statement of precedents for similar dams
- Logs of drill holes, test pits, and exploratory trenches
- Site geologic reports
- Site seismicity reports
- Materials exploration and testing reports
- Reservoir area-capacity curves, rim conditions, and drainage basin information
- Dambreak analyses and reports
- Construction reports
- Correspondence that may highlight design changes or problems
- Design drawings and specifications
- Design reports including assumptions used and the reasons therefore for the assumptions

- Inspection records
- Maintenance records
- Aerial photography
- Licensee's reports
- Construction photographs
- Concrete materials and mix design
- As built drawings
- Cross-sections showing embankment, cutoff, foundation, geology, piezometric levels, and the location of other instruments, such as inclinometers.

4-3 Review of Existing Data

Appendix 4-A is a listing of various engineering data related to the design, construction, and operation of an embankment dam. Prior to review and analysis of existing data, this appendix may be useful in organizing the data as discussed in the U.S. Bureau of Reclamation's "Safety Evaluation of Existing Dams (SEED) Manual" (Ref. 2)

The engineer performing the review should examine all data to determine if problem areas have been recognized and, if appropriate methods are proposed for correction. Additionally, the data should be examined to determine if the source of any current conditions or problems, such as seepage, settlement, cracking, etc., are evident from existing data. The methodologies and criteria used in the design should be examined and compared to accepted state-of-the-art procedures and criteria.

Advances in accepted state-of-the-art methodologies may require a reevaluation of the original design by use of these guidelines. The SEED Manual discusses in greater detail specific information to look for in the reports and data that may be available.

4-4 Need for Supplemental Information

The objective of reviewing existing data is to be able to use as much information as is available to evaluate the structural adequacy of existing or proposed embankment dams. Data and analyses should be the prevalent basis for judgments on dam safety. If potentially hazardous conditions are believed or determined to exist, and the existing data are insufficient to resolve the problem, it may be necessary to request supplemental investigations, analyses, or information to complete the evaluation. The supplemental information could involve additional visual inspections, measurements, foundation exploration and testing of materials, seismic information, and hydrologic and hydraulic data. Conditions that may require supplemental information are as follows:

- Significant cracking, settlement or sloughing of an existing embankment and the potential for such in any proposed structure

- Uncontrolled seepage conditions through or under the embankment, the abutments, or at the toe area, and the potential for such in any proposed structure. Areas deserving intense scrutiny are those embankments founded over alluvium without a cutoff to bedrock.
- Available data is not adequate to perform accepted state-of-the-art analyses that are necessary
- Increase in settlement rate or horizontal movement rate either upstream or downstream
- Increases or decreases in measured seepage quantities
- Rises in internal pore pressures

4-5 Evaluation of Embankment Dams

The two principal types of embankment dams are earth dams and rock-fill dams, depending on the predominant fill material used.

a. Earth Dams - An earth dam is composed of suitable soils obtained from borrow areas or required excavation which are then spread and compacted in layers by mechanical means. Earth dams have been constructed as both homogeneous or zoned dams.

Zoned dams are generally preferred since zoning incorporates the use of drains and filters in the embankment to control seepage. Homogeneous embankments without a chimney drain and filters are usually not considered acceptable in new modern construction, but many dams of this type are in service and must be inspected and monitored very carefully.

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 (e.g. SPT blow counts, gradation analysis, phreatic surface, etc.) must be available to evaluate the liquefaction potential and stability of these dams.

b. Rock-fill Dams (Ref. 3, Chapter 1) – A rock-fill central core dam is an embankment composed largely of fragmented rockfill shells with an impervious earth core. The core is separated from the shells by a series of transition zones built of properly graded materials. The impervious core may be central or inclined. The core transition zones, drains and filters, etc. should be evaluated as discussed in Section 4-5.1.

Concrete faced rockfill dams (CFRD's) are now very common throughout the world. The concrete face serves as the impervious member and also eliminates the need for rip-rap since it is not erodible by wave action. It is essential that the downstream rockfill shell be freely draining for the steep slopes usually selected for this type of dam. Clean gravels and cobbles have also been used for downstream shells of concrete faced dams, it is important that the shell materials be free draining. The state of the art of designing CFRD's is very well given in the J. Barry Cooke Volume, 2000 (Ref.), even though the empirical approach is overdone. Static and dynamic stability analyses of CFRD's is just as necessary as for central core rock-fill dams.

Rock-fill zones should be compacted in layers, 24 to 36 inches thick by 4 to 6 passes with 10 to 15 ton steel-wheel vibratory rollers. Layer thicknesses up to 72 inches have been also used in the downstream one-half of many CFRD rockfills; this practice should be discontinued. The largest particle diameter generally should not exceed .7 of the compacted layer thickness. Dumping rock-fill is generally not acceptable for embankment dam construction today. However, the application of some water before compaction, on rock-fill, to achieve better compaction is common, but not always used. It is considered to be good practice to use about 170 liters of water per cubic meter of compacted rockfill.

The structural safety of an embankment dam is dependent primarily on the absence of excessive deformations under all conditions of environment and operation, the ability to safely pass flood flows without overtopping the embankment, and the control of seepage to prevent piping of materials and to control pore pressures and thus preclude adverse effects on stability.

To properly evaluate the stability of an embankment dam, the following areas should be reviewed.

- Embankment zoning and cross section
- Seepage control measures (Drains and filters) to control pore pressures and to preclude piping.
- Deformation, predicted and measured
- Erosion control measures such as bedded rip rap and filters to control piping by backward erosion.
- Structural stability analyses
- Liquefaction potential
- Overtopping potential and the ability to resist overtopping
- Foundation and embankment material properties and strengths
- Adequacy of freeboard

For existing dams, the review should also include summarizing the past behavior of the dam, with attention given to any problem areas noted, changes in measured seepage, changes in measured pore pressures, changes in measured settlements and horizontal movements.

4-5.1 Embankment Zoning

For zoned embankments, the zoning geometry and properties of the materials placed in the zones should be reviewed to determine: (1) the structural design, and (2) the types of internal features such as chimney drains, blanket drains, toe drains, etc., that are proposed or were used to provide for and maintain embankment stability. One should keep in mind that embankment zoning is also established for economic reasons according to the availability of materials (Ref. 1, Chapter 6). The embankment zoning should provide an adequate impervious zone, filter and drainage zones between the core and the shells, and seepage control zones. Desirable characteristics that these zones should have or provide are as follows:

- In general, the width of the core at the base of cutoff should be equal to, or greater than, 25 per cent of the maximum difference between the maximum reservoir and minimum tailwater elevations. The minimum top width of the core should not be less than 10 feet (Ref. 3, pg. 5-3). The coefficient of permeability of the core material should preferably be 10^{-4} cm/sec or less. More permeable core material may be acceptable if seepage is adequately controlled and appropriate factors of safety are still met (Ref. 1, Chapter 6).

- Transition zones must meet accepted filter criteria, e.g. see References 1, 4, & 5, to protect the adjacent zones from piping. The transition zones should be sufficiently wide to ensure that they are continuous and constructable with a minimum of contamination at the contact (Ref 4, pg. 57, 607; Ref. 1, Chapter 6). The range of gradation of the transition zones should be limited to avoid segregation of materials during placement.

- Seepage control features such as pervious drains within the embankment should be sized adequately to contain all seepage flows. The features should also be sufficiently pervious to ensure that all seepage will be intercepted and controlled without excessive pressure head losses (Ref. 1, Chapter 6, Ref. 3, pg. 5-3).

- Zoning of an embankment that places the more pervious material on each side of the core zone is preferable. This placement improves the stability of the embankment during rapid drawdown conditions and keeps the downstream slope drained for greater effective weight (Ref. 5, pg. 7). It is conservative to utilize a filter material on the upstream site of the core to act as a potential crack stopper.

Homogeneous dams should also have seepage control features such as chimney drains, blanket drains, etc., including a filter zone between the main embankment material and the drain. Desirable characteristics listed above also apply to the features of this type of structure. The homogeneous structure is generally more massive and usually has flatter slopes than a zoned embankment of the same height. These characteristics compensate for a tendency toward a higher phreatic line in the homogeneous embankment. They also tend to provide better slope stability during rapid drawdown (Ref. 1, Chap. 6).

4-5.2 Seepage Control Measures

All embankment dams are subject to some seepage passing through, under, and around them (Ref. 5, pg. 1). If uncontrolled, seepage may be detrimental to the stability of the structure as a result of excessive internal pore water pressures or by piping (Ref. 3, pg. 1-6). For existing dams, records or evidence that seepage flows have removed any significant degree of fine grained material must be evaluated. Any such records requires further field investigation.

Seepage discharge should be effectively controlled to preclude structural damage or interference with normal operations.

In the evaluation of seepage reduction or seepage control measures as they pertain to dam safety, one should review and evaluate the following:

- Protective control measures such as relief wells, weighted graded filters, horizontal drains, or chimney drains which prevent seepage forces from endangering the stability of the downstream slope (Ref. 3, p. 1-6).

- Filters and transition zones designed to prevent movement of soil particles that could clog drains or result in piping (Ref. 4, pg. 57; Ref. 1, pg. 218).

- Drainage blankets, chimney drains, and toe drains designed to ensure that they control and safely discharge seepage for all conditions. The design of these features must also provide sufficient flow capacity to safely control seepage through potential cracks in the embankment impervious zone (Ref. 3, pg. 1-6).

- Contacts of seepage control features with the foundation, abutments, embedded structures, etc., designed to prevent the occurrence of piping and/or hydrofracturing of embankment and foundation materials (Ref. 1). If conduits or pipes exist through the embankment, they should be inspected to insure that they are functional or have been properly sealed. If there is the slightest doubt about through going conduits, they should be decommissioned and replaced.

- Grouting, cut-off trenches, and impervious blankets. The use of impervious blankets or new structures in place of a cutoff is discouraged because of piping of the blanket materials into the foundation on recent projects. Blankets should only be used as a last resort element; and, then the use of a filter between the blanket and the foundation must be evaluated, not immediately omitted.

- Construction records for foundation shaping, treatment and grouting at the contact between the impervious core and foundation.

- Measures such as compaction requirements, seepage collars, placement of special materials, or other similar features to prevent internal erosion from seepage at the interface with concrete structures (Ref. 1). If seepage collars are present, special attention should be given to compaction requirements around them. The use of seepage collars is not recommended in new construction.

- For existing embankments, all seepage records compiled during the existence of the structure should be reviewed for significant trends or abnormal changes. The causes of any abnormalities should be determined as accurately as possible.

As indicated in the introductory remarks to this chapter, piping by backward erosion is one of the most common causes of embankment dam failures. The keys to controlling seepage to preclude piping are 1) the placement of filter over seepage exits from erodible soils to prevent migration of the erodible or pipeable soils; and, 2) the gradation of the filters must be appropriate to prevent migration of the erodible material into the filter, and the filter must not migrate into the drain or the bedrock downstream of the filter. It is indeed sobering that the piping failures cited at the beginning of this chapter were cases where the appropriate filter zones were absent due to omission of considering all possible seepage paths along which piping could occur. In many existing dams there are unfiltered seepage outlets which are uninspectable because they are located under the tailwater on the downstream side of the dam. An additional consideration, once it is decided to place filters at the appropriate locations, is that the filter and drain, or multiple filters be graded to serve the intended purpose of protecting the erodible materials and that the filters and drains be placed without segregation. In this respect, it has been established in a recent years that the original Terzaghi filter criteria is not appropriate for the protection of well graded or gap graded materials. A more complete discussion of filters is given in Appendix 4-B.

4-5.3 Deformation, Predicted or Recorded

The type, amount, and rate of deformation of an embankment either vertical or horizontal movement, should be estimated during the design stage and must be recorded during the operation of the structure. For proposed embankments, the structure should generally be cambered to allow for the estimated settlement during the life of the structure. For existing embankments, any evidence or records of unusual settlement, cracking, or movement should be reviewed to determine whether these conditions are detrimental to the continued safe operation of the structure. Field investigations may be required to determine the causes of these abnormalities. These investigations may involve such items as surveying the structure, installing movement detecting instruments, or excavating test pits for examination (Ref. 4, Chapter 12). The embankment history, height, foundation conditions, hazard, etc. are factors to be considered in determining field investigation needs.

As a result of deformation, cracking can develop through the impervious core section below the line of saturation which may result in piping. Adequately sized and graded filter and drain zones located downstream from the impervious core can prevent piping (Ref. 4, Chapter 11). A filter crackstopper on the upstream side of the core is also a useful zone to control seepage in the event of core cracking. Corrective measures may be needed if adequate filter zones do not exist or are not correctly located.

4-5.4 Erosion Control Measures

Upstream and downstream slopes, the toe area, groin areas of the abutments, approach and discharge channels, and areas adjacent to concrete structures should be protected against excessive erosion from wave action, surface runoff, and impinging currents. Inadequate erosion protection can result in slope instability (Ref. 3, Chapter 5). Some common types of protection used are riprap, gabions, paving (concrete or asphalt), and appropriate vegetative cover.

The slope and toe protection of all embankment dams should be reviewed to determine if the dam is adequately protected against erosive forces. If the slope protection is being continually displaced, heavier protection is required. Additionally, if embankment materials, consisting of silty and sandy soils, are being moved into the slope protection, measures must be taken to correct this condition before erosion becomes detrimental to the embankment. If riprap is required, a bedding layer must be designed according to established filter criteria and placed under the riprap protection (Ref. 1, Chapter 6).

4-5.5 Structural Stability Analyses

The evaluation of the stability of embankment dams shall be based on the available design information for proposed structures and on design and construction information and records of performance for existing embankments. The Corps of Engineers Guidelines for Safety Inspection of Dams (Ref. 6) can be used as a guide in performing the review.

Stability studies and analyses for proposed embankments will be conducted during design in accordance with methods discussed in Section 4-6.8. Quality control testing during construction will be used to confirm that the design values are being achieved. For existing embankments, the initial stability studies and analyses will normally be acceptable if they were performed by approved methodologies and if observations of the performance during reservoir operation do not suggest potential instability. It is also very important to check the design slopes against previous precedent for similar dams, in spite of the calculations. Be especially suspicious and check all assumptions when the design calculations yield a slope steeper than normal precedent. Additional stability analyses should be performed if initial design analyses do not exist or are incomplete, if existing conditions have deteriorated, if hazard potential of the project has increased, if the embankment has been subjected to loading conditions more severe than designed for, if existing analyses are not in agreement with current accepted state-of-the-art methodologies, or if assumed design parameters cannot be satisfactorily justified. Satisfactory behavior of the embankment under loading conditions not expected to be exceeded during the life of the structure should generally be indicative of satisfactory stability, provided adverse changes in the physical condition of the embankment have not occurred (Ref. 6, pg. 10).

Evidence of any adverse changes which could affect the stability of an embankment may be obtained from visual inspection and observation of available instrumentation data covering such items as changes in pore water pressures, displacements, changes in loading conditions, seepage, etc. Review of maintenance records and related information may also provide a reference to structural behavior data for a particular structure. Should a review of project records indicate possible deficiencies in the stability of an embankment, additional information may be required regarding the foundation and embankment materials. The Corps of Engineers Guidelines for Safety Inspection of Dams (Ref. 6, p. 10) and other available literature (Ref. 3, Ref. 5, Ref. 7, Ref. 8, Ref. 9,k Ref. 10, Ref. 15) can be referred to in establishing the information necessary to determine the condition and material properties of the foundation and embankment.

4-5.6 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 deformation with high pore water pressures developing, resulting in a loss of strength or resistance to deformation.

The potential for liquefaction in an embankment or its foundation must be evaluated on the basis of empirical knowledge and engineering judgment supplemented by special laboratory tests when necessary. The current state of the art for evaluation soil liquefaction potential is by using the standard Penetration blow counts to estimate the cyclic strength ratio of the sand and to compare the cyclic strength ratio with the cyclic stress ratio induced by the earthquake motions. The cyclic strength-blow count relationships have been established from case histories (Ref. 12). The induced cyclic stress ratios can be calculated by the simplified method or by the use of computer programs to calculate the induced dynamic shear stresses more accurately. Further discussion of liquefaction is presented in Section 4-7.

4-5.7 Soil Properties

Soil properties including strength and seepage parameters to be used as input data for stability analyses should be realistic and representative of the range and variation that exist in the foundation abutment, and embankment materials (Ref. 14). For information concerning the characteristics and strengths of foundation and embankment soils and rock, refer to the procedures established in the Corps of Engineers and U.S. Bureau of Reclamation Guidelines (Ref. 8, Ref. 9, Ref. 2, Ref. 15), and other literature (Ref. 16, Ref. 4, Ref. 17, Ref. 34). The selection of the proper input parameters and their correct use in a stability analysis are generally of greater importance than the specific method of stability analysis used.

4-5.8 Embankment Overtopping Potential

All embankment dams, either proposed or existing, should be evaluated for overtopping potential under the most extreme conditions expected for which the dam is determined to be a hazard to life or property. Chapter 2 of these Guidelines discusses the Spillway Design Flood and provides freeboard criteria. The maximum reservoir elevation determined for the design flood and expected wave runup are conditions that should be considered. However, a less severe storm with lower reservoir elevation but greater wave propagation may result in conditions that are more critical than those produced by the design flood. In general, overtopping of an embankment dam is not acceptable. It is not considered acceptable to prevent overtopping , for operational flows, by routinely storing water against a parapet wall on the crest of an embankment.

4-6 Static Stability Evaluation

4-6.1 General

As discussed in Section 4-1.2, a new, independent stability analysis by staff is not necessarily required for a proposed or existing embankment. Spot checks of analyses may be required to verify that application of the specific analytical approach is correct. The analysis and evaluation of the structural adequacy of an embankment dam by the licensee and/or its consultant should be reviewed based on information formulated by the licensee and information developed by the Regional Office staff from various project inspections and data requests resulting from the licensing or inspection program. For embankment dams, stability analyses should be examined to determine if the criteria used and loading conditions analyzed are appropriate. This review should be based on the above information to determine if the methods of analyses used are based on accepted state-of-the-art and that proper types of failure surfaces have been analyzed (e.g., wedge, circular, or noncircular).

An independent stability analysis should be performed by staff if actual conditions differ from those assumed in the licensee's analysis, if soil parameters are inconsistent with material types, if soil strength parameters or pore water pressures are inconsistent with the method being used, or if the critical failure surfaces do not appear to have been determined. The staff should always compare the selected design slopes of the dam in question with precedent from dams composed of similar materials on similar foundations. If the slopes of the dam in question are significantly steeper than precedent, then the assumptions and mechanics of the design calculations must be checked in detail by the staff.

Staff has several stability programs for computers available (Ref. 37 and 38). These programs may be used by staff in reviewing the results of the licensee's analyses. It should, however, be understood that the results obtained by these methods of analyses may not necessarily agree exactly with the licensee's results based on another method; however, it will provide an indication as to the adequacy of the analysis being reviewed. Staff is not limited to the use of these computer programs. Other accepted programs may also be used. The staff should verify that the licensee has checked the analysis by hand calculations for potential critical cases that have marginal factors of safety. The staff should also perform hand calculations on the critical surface as an independent check.

References are listed in Section 4-8 that analyze the various methods of stability analyses in detail. An historical development of methods of stability analyses is presented in Reference 16 (pp. 323-326).

4.6-2 Review Approach

Stability analyses should be reviewed to determine if input data appear appropriate based on a knowledge of the embankment and foundation materials, on pore pressures in the embankment and its foundation, or if the method of analysis chosen by the licensee is being used correctly. The literature provides several publications, textbooks, and other sources of information that discuss in detail the various methods of analyses available. Refer to Section 4-8 for references that can be used to obtain information for use in reviewing a particular method of stability analysis (Ref. 20, Ref. 26).

A review of the stability analyses presented by the licensee shall include an evaluation and summary of the data used in the analysis and an evaluation to determine if the critical conditions have been investigated. The items to be evaluated include:

- Densities of soils
- Shear strength parameters
- Pore water pressures, estimated or measured
- Loading conditions
- Trial failure surfaces
- Method of analysis

The soil densities and shear strengths to be used for the various loading conditions investigated can be evaluated by studying available laboratory test data and/or comparing data presented relative to that known for similar materials based on past experiences and data available from other dams consisting of similar materials and construction methods.

Pore water pressures used in the analyses of the various loading conditions investigated should be reviewed to determine if they are realistic based on available instrumentation data or estimates based on such methods as those proposed by Casagrande (Ref. 18) and Carstens and May (Ref. 19).

When field explorations and laboratory testing are required to provide additional information concerning the strength characteristics of the embankment materials, the sampling and laboratory testing procedures should be reviewed to determine if they were adequately accomplished and are representative of the conditions analyzed. Corps of Engineers and U.S. Bureau of Reclamation technical guidelines concerning sampling and laboratory testing procedures can be used to complete this review (Ref. 9, Ref. 10, Ref. 15).

4-6.3 Conditions to be Investigated

An embankment and its foundation are subject to shear stresses imposed by the weight of the embankment and by pool fluctuations, seepage, or earthquake forces.

Loading conditions vary from the commencement of construction of the embankment until the time when the embankment has been completed and has a full reservoir pool behind it. The range of loading conditions encompasses the following conditions at various stages from construction through the operational stage of the completed embankment:

- End-of-Construction
- Sudden drawdown
- Partial pool with steady seepage
- Steady seepage, normal pool
- Earthquake
- Appropriate flood surcharge pool

In all loading cases, the shear strength along any potential failure surface must be defined. The shear strength available to resist failure along any particular failure surface depends on the loading conditions applied, and the rate of change of the loading conditions.

4-6.4 Shear Strength

Generally, the shear strengths of materials used in stability analyses are determined from laboratory testing procedures which attempt to duplicate the various loading conditions to which the embankment is expected to be subjected (Ref. 1, Ref. 16, Ref. 20).

From the time construction begins until the reservoir has been filled and a state of steady seepage has been established, and during reservoir operation subsequent to establishment of the steady-state seepage condition, three different static loading conditions may act on the embankment and foundation. These are the end-of-construction, steady seepage, and sudden drawdown conditions. Shear strength values used in stability analyses for these loading conditions depend on consolidation conditions and on the shear-induced excess pore pressures generated by the loadings. Laboratory tests on specimens of embankment material which are compacted in the laboratory to the dry densities and water contents anticipated in a proposed dam, and on undisturbed samples of natural foundation soils or embankment materials from an existing dam, are conducted to simulate the conditions of consolidation and shear-induced pore pressure dissipation expected for the various loading conditions, in order to determine the appropriate shear strength values.

In general three different shear strength values are required for the static stability analyses and these can be determined by three different types of laboratory triaxial tests:

(1) <u>Unconsolidated-Undrained Strength</u>, determined by a test (referred to as UU-type) in which no consolidation is permitted under the initial confining pressure and in which no drainage is permitted during the shearing stage so that the water content

is kept constant. In other words shear-induced pore pressures are not allowed to dissipate in this test. A special type of this test in which the initial confining pressure is zero is the Unconfined Compression Test (referred to as UC-type).

(2) Consolidated-Undrained Strength, determined by a test (referred to as CU-type) in which the sample is consolidated under the initial confining pressure, and in which the drainage is not permitted during the shearing stage so that the water content is kept constant. In other words, the shearing stage of this test is the same as in the UU test, in that shear induced pore pressures are not allowed to dissipate. A variation often used in this type of test is to measure the shear-induced pore pressures during the test, in which case the test is referred to as a $\overline{\text{CU}}$-type.

(3) Consolidated-Drained Strength, determined by a test (referred to as CD-type) in which consolidation is permitted under the initial confining pressure, and complete dissipation of shear-induced pore pressure is allowed for each increment of shear stress applied in the shearing stage of the test. The consolidated-drained strength can also be determined from the $\overline{\text{CU}}$-type test as explained in the following paragraphs.

The shear strength values described in (1) and (2) are commonly referred to as undrained shear strengths, abbreviated as S_u. Whether the undrained shear strength determined in (1) or (2) is appropriate for use in stability analyses depends on the initial consolidation conditions as discussed in the following paragraphs. The shear strength described in (3) is commonly referred to as the drained shear strength. The undrained shear strength is used in stability analyses for loading conditions during which shear-induced pore pressures cannot dissipate, due to the rapidity of the application of the shear stresses by the loading with respect to the permeability and drainage boundary conditions of the materials involved. Conversely, the drained shear strength is used in stability analyses for loading conditions during which shear-induced pore pressures are zero, due to the slow application of the shear stresses by the loading with respect to the permeability and drainage boundary conditions of the materials involved.

In addition to the shear strengths described in the foregoing paragraphs, the residual shear strength may be applicable if prior large shear deformations, or prior shear failure, has occurred in the foundation materials of existing or proposed embankment dams. The residual, or ultimate, condition is present in clayey materials where a reorientation of the clay minerals into a face-to-face arrangement has resulted from large shear deformation. Because the application of shear stress to a material which is at the residual condition produces no further particle reorientation, shear-induced pore pressures are zero and in this sense, the residual shear strength is also a drained strength. The residual strength is most often appropriate for analyses of foundations consisting of bedded shales, metamorphic rocks containing shear zones, faulted rocks, or colluvium.

4-6.4.1 Laboratory Testing

Testing procedures for determining the shear strengths of soils to be used in stability analyses, as well as determining other engineering properties of soils, such as density, moisture content, consolidation, permeability, gradation, etc., can be found in Corps of Engineers and U.S. Bureau of Reclamation manuals (Ref. 10, Ref. 15). When reviewing analyses of existing and proposed embankments the drained and undrained shear strengths may be applicable. In situations where embankment soils exist which may not have completely consolidated years after construction was completed, a strength envelope between the UU and CU envelopes may be appropriate in evaluating the stability of an existing embankment dam. For proposed dams, shear strength parameters obtained from the UU test will also be applicable for the end-of-construction condition. For existing or proposed dams founded on materials which have been subjected to large prior shear deformations, the residual strength, measured by direct-shear or rotation-shear tests, will be applicable.

4-6.4.2 Unconsolidated-Undrained Shear Strength (UU or UC-type tests)

Unconsolidated-undrained tests approximate the end-of-construction behavior of impervious natural foundation soils or embankment zones in which rate of consolidation is slow compared to rate of fill placement, and measure the undrained shear strength s_u. It should be noted that the terminology unconsolidated-undrained does not mean the sample is unconsolidated. An unconsolidated sample would have a consistency similar to a soil at a moisture content equal to or greater than the liquid limit. Rather, in this context the term "unconsolidated" means the sample is subjected to <u>no additional</u> consolidation after compaction.

Confining pressures used in the tests should encompass the range in total normal stresses which will act along potential failure surfaces through the impervious materials. The undrained shear strength is highly dependent on both compacted dry density and molding water content for embankment materials. Thus laboratory samples for these materials should be compacted to the dry density specified for the impervious embankment zones at water contents wet and dry of optimum within the range of compaction water content to be permitted in the Specifications. Since the undrained strength is dependent on dry density, it is desirable to obtain samples for testing by trimming cylindrical test samples from a specimen compacted in a Proctor mold. Figure 1 shows the recommended procedure.

Impervious foundation materials which are stiff and fissured may fail under drained conditions (i.e. shear-induced pore pressure equal to zero) even though they are fine-grained and subjected to short-term loading such as end-of-construction. This is because the fissures permit rapid dissipation of the shear-induced pore pressures. Therefore, an analysis using the drained strength may be more conservative and should be used if the drained strength is lower than the undrained strength. See Sections 4-6.4.4 and 4-6.6.1.

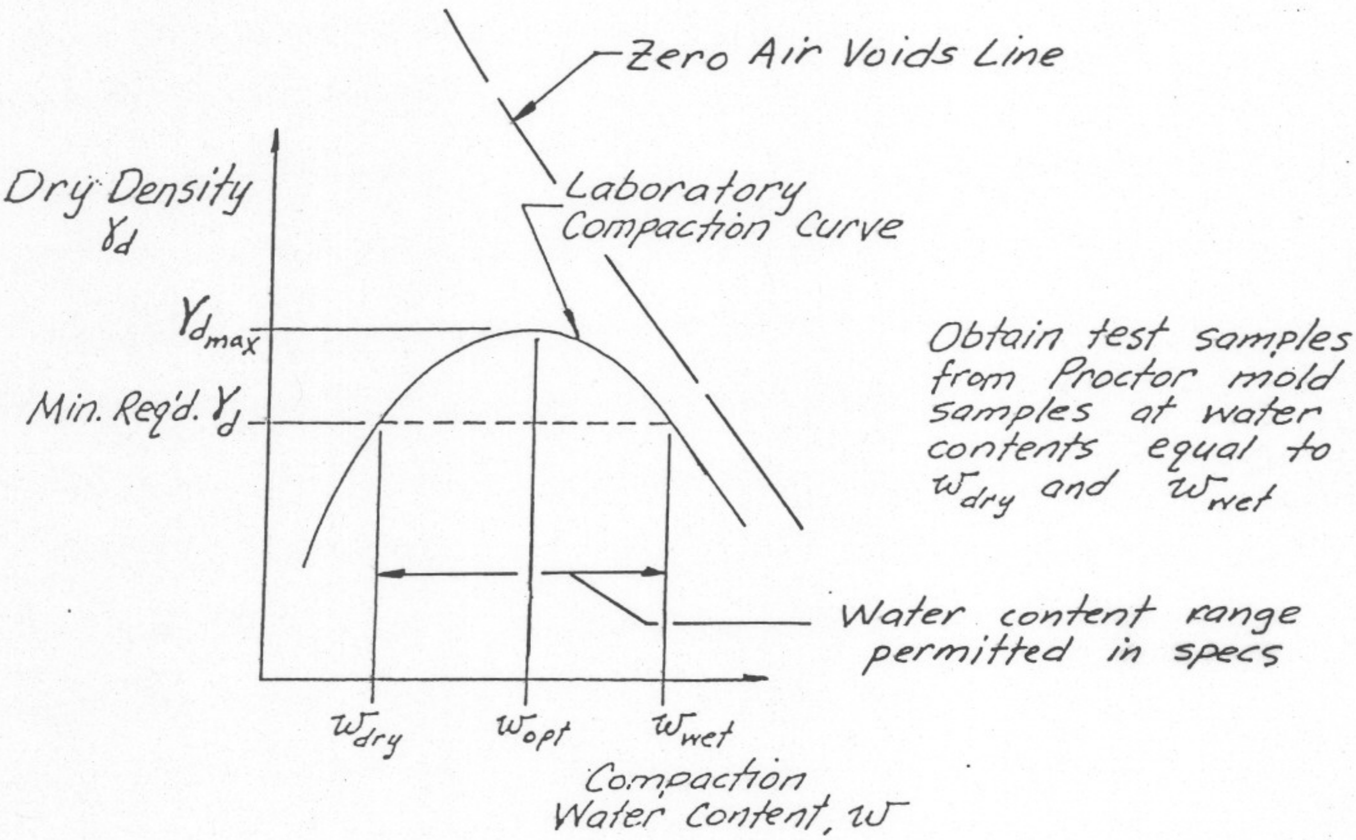
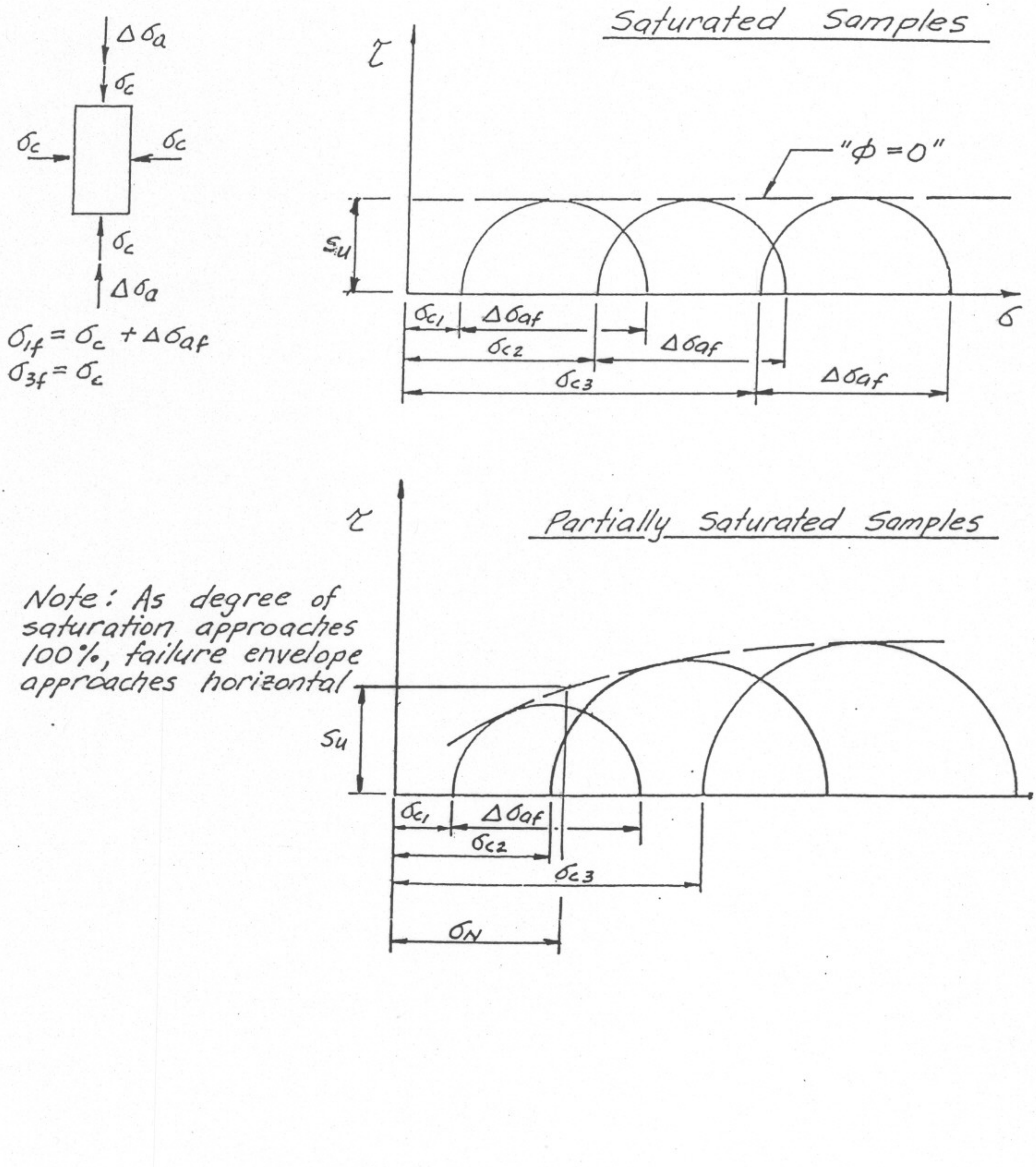

Zero Air Voids Line
Dry Density
γ_d
Laboratory
Compaction Curve
$\gamma_{d_{max}}$
Min. Req'd. γ_d
Obtain test samples from Proctor mold samples at water contents equal to w_{dry} and w_{wet}
Water content range permitted in specs
w_{dry}
w_{opt}
w_{wet}
Compaction
Water Content, w

Figure 1

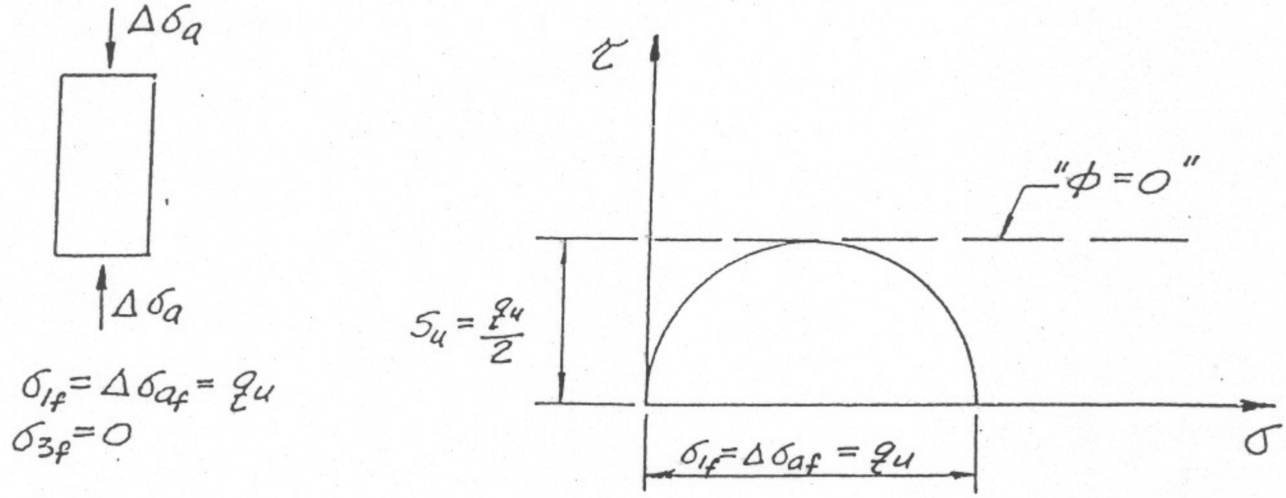
Figure 2 shows the stresses applied in the UU test and typical strength envelopes. The confining pressure σ_c is applied with the drainage line closed, followed by the application of the axial stress $\Delta\sigma_a$ with the drainage line remaining closed. The axial stress at failure is taken as the peak on the axial stress versus strain diagram or the axial stress at an axial strain of 20 percent if no peak is present. Strength envelopes for saturated samples are typically horizontal, whereas strength envelopes for partially saturated samples are curved in the low confining pressure range. In cases where foundation or embankment soils are unsaturated but will become saturated during construction, it is advisable to saturate specimens prior to application of the axial stress.

Unconfined compression (UC) tests can be used to estimate the undrained shear strength s_u of saturated, fine-grained foundation materials. The undrained shear strength of such foundation materials will generally be conservative for analysis of end-of-construction, because consolidation which occurs during construction will increase s_u and this increase is ignored when the UC results are used. Because the unconfined compression test is far simpler to conduct than a UU test, it is commonly used in practice to determine the undrained shear strength of saturated foundation materials. Figure 3 shows the stresses applied to the sample and the shear strength envelope. As for the UU test, the axial stress at failure (which is the unconfined compressive strength) is taken as the peak on the axial stress versus strain diagram, or the axial stress at an axial strain of 20 percent if no peak is present.



Δσa
σc
σc
σc
σc
Δσa
σ1f = σc + Δσaf
σ3f = σc
Saturated Samples
τ
"φ = 0"
su
σ
σc1
Δσaf
σc2
Δσaf
σc3
Δσaf
τ
Partially Saturated Samples
Note: As degree of saturation approaches 100%, failure envelope approaches horizontal
su
σc1
Δσaf
σc2
σc3
σN

Figure 2



$\Delta\sigma_a$
$\Delta\sigma_a$
$\sigma_{1f} = \Delta\sigma_{af} = q_u$
$\sigma_{3f} = 0$
τ
"$\phi = 0$"
$s_u = \frac{q_u}{2}$
σ
$\sigma_{1f} = \Delta\sigma_{af} = q_u$

Figure 3

Since the undrained shear strength is in general not constant with depth, unconfined compression tests should be conducted on samples obtained from the range of depths in the foundation materials which are likely to be involved in a potential failure. The results of these tests should be used to develop a profile of undrained shear strength versus depth in the foundation from which the available undrained shear strength along potential failure surfaces at corresponding depths can be estimated.

The unconfined compressive strength will generally provide a conservative estimate of undrained shear strength because of sample disturbance. Good undisturbed samples are necessary and should be obtained from hand-carved block samples or thin-walled tube samples having a minimum diameter of at least 2.5 inches.

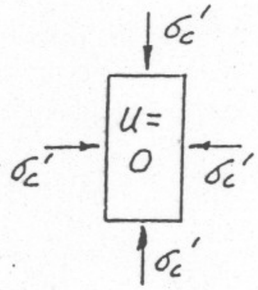
The unconfined compression test should be limited to fine-grained soils classified as CL, CH, ML, MH, or CL-ML.

Very stiff fine grained soils are often fissured and the unconfined compression test may give misleadingly low unconsolidated, undrained strengths for these materials, because premature failure will occur along fissures. Such materials are more appropriately tested in UU triaxial tests. More importantly, these materials may fail under drained conditions. See Sections 4-6.4.4 and 4-6.6.1.

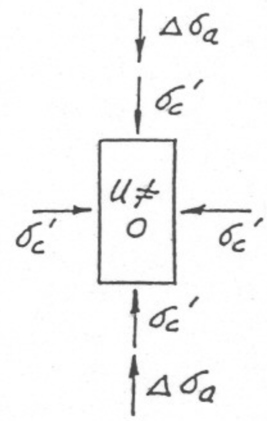
4-6.4.3 Consolidated-Undrained Shear Strength (CU- or $\overline{CU}$-type tests)

Consolidated undrained triaxial tests are referred to as CU-types or $\overline{CU}$-types if shear-induced pore pressures are measured in the test. Consolidated undrained triaxial tests with pore pressure measurements have two primary uses in stability analyses: (1) they furnish drained shear strength parameters, ϕ' and c', for use in steady-state seepage analyses or other loading conditions where shear-induced pore pressures can be taken as zero (e.g. pervious zones in rapid drawdown or end of construction); and (2) they furnish undrained shear strengths as functions of effective consolidation pressures for use in rapid drawdown analyses and in earthquake analyses. Figure 4 shows the stresses applied in $\overline{CU}$ tests. The confining pressure σ_c' is applied with the drainage line open so that the sample consolidates. The axial stress difference $\Delta\sigma_a$ is applied with the drainage line closed which causes shear-induced pore pressures to develop inside the sample; these are commonly measured with electronic pore pressure transducers inserted in the sample drainage line. As in the UU and UC tests, the axial stress difference at failure is taken as the peak on the axial stress difference versus axial strain diagram, or as the axial stress difference at 20 percent strain if a peak is not present.

Stage I
(Consolidation)



Stage II
(Undrained Shearing)



At failure: Total Stresses – $\sigma_{3f} = \sigma_c'$,
$\sigma_{1f} = \sigma_c' + \Delta\sigma_{af}$

Effective Stresses – $\sigma_{3f}' = \sigma_{3f} - u_f = \sigma_c' - u_f$

$\sigma_{1f}' = \sigma_{3f}' + \Delta\sigma_{af} = \sigma_c' + \Delta\sigma_{af} - u_f$

where u_f = pore pressure at failure induced by $\Delta\sigma_{af}$

Undrained Shear strength – $s_u = \frac{1}{2}\Delta\sigma_{af}\cos\phi'$
(see Fig. 6 for derivation)

Figure 4

Figure 5 shows the effective stress Mohr-Coulomb failure envelope, from which the drained shear strength parameters ϕ' and c' can be determined. Figure 6 shows the undrained strength envelope which is a plot of undrained shear strength as a function of the consolidation pressure used in Stage I of the test.

As discussed in Section 4-6.6.2, rapid drawdown analyses should utilize shear strengths obtained from a composite of the drained and undrained strength envelopes. The normal stress used to select the shear strength from the composite envelope should be the effective consolidation stress acting on the potential failure surface under steady state seepage prior to drawdown (i.e. the total normal stress minus the pore pressure taken from a flow net drawn for the pool level before drawdown). Figure 7 shows the composite envelope. In some cases, the drained and undrained envelopes may exhibit slight curvature; however, they can typically approximated as linear to facilitate stability calculations. This approximation does not cause significant error in the calculations.

Consolidation pressures used in the tests should encompass the range in effective normal stresses which will act along potential failure surfaces prior to drawdown. Since the undrained shear strength derived from the consolidated-undrained tests is highly dependent on both compacted dry density and molding water content for embankment materials, samples should be prepared as discussed for UU tests. Derivation of the drained shear strength parameters from the consolidated-undrained tests depends on the accurate measurement of shear-induced pore pressures, and the samples must be

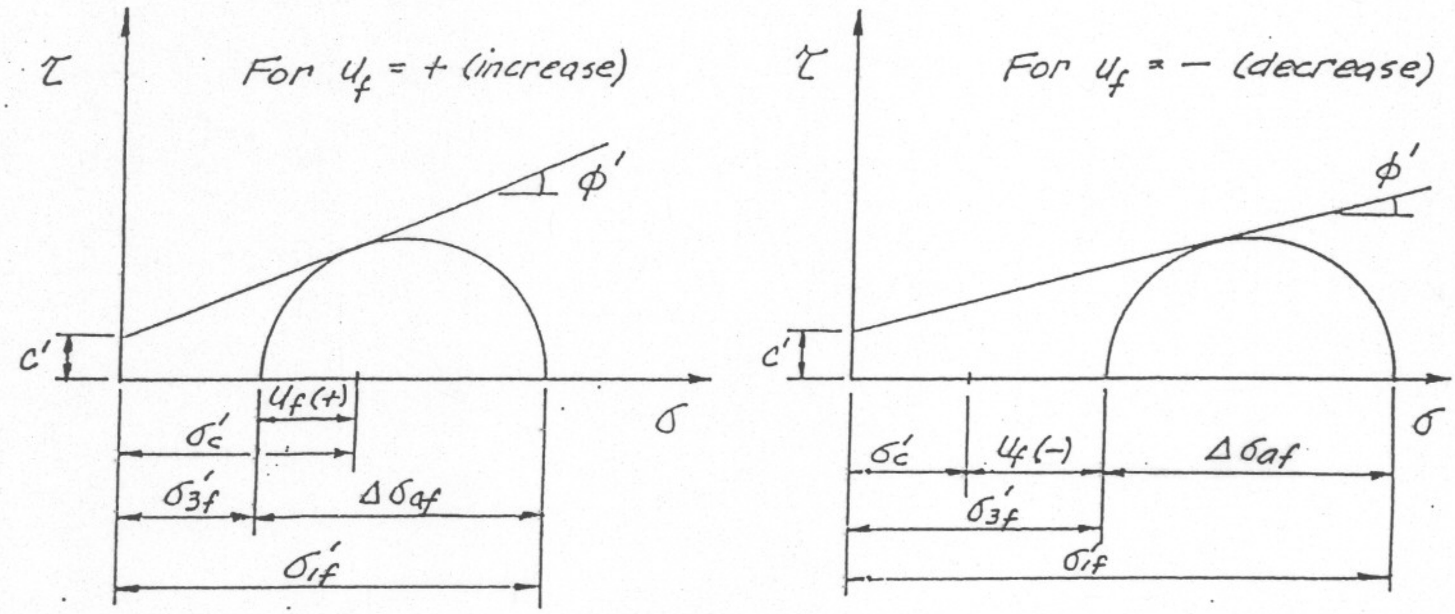
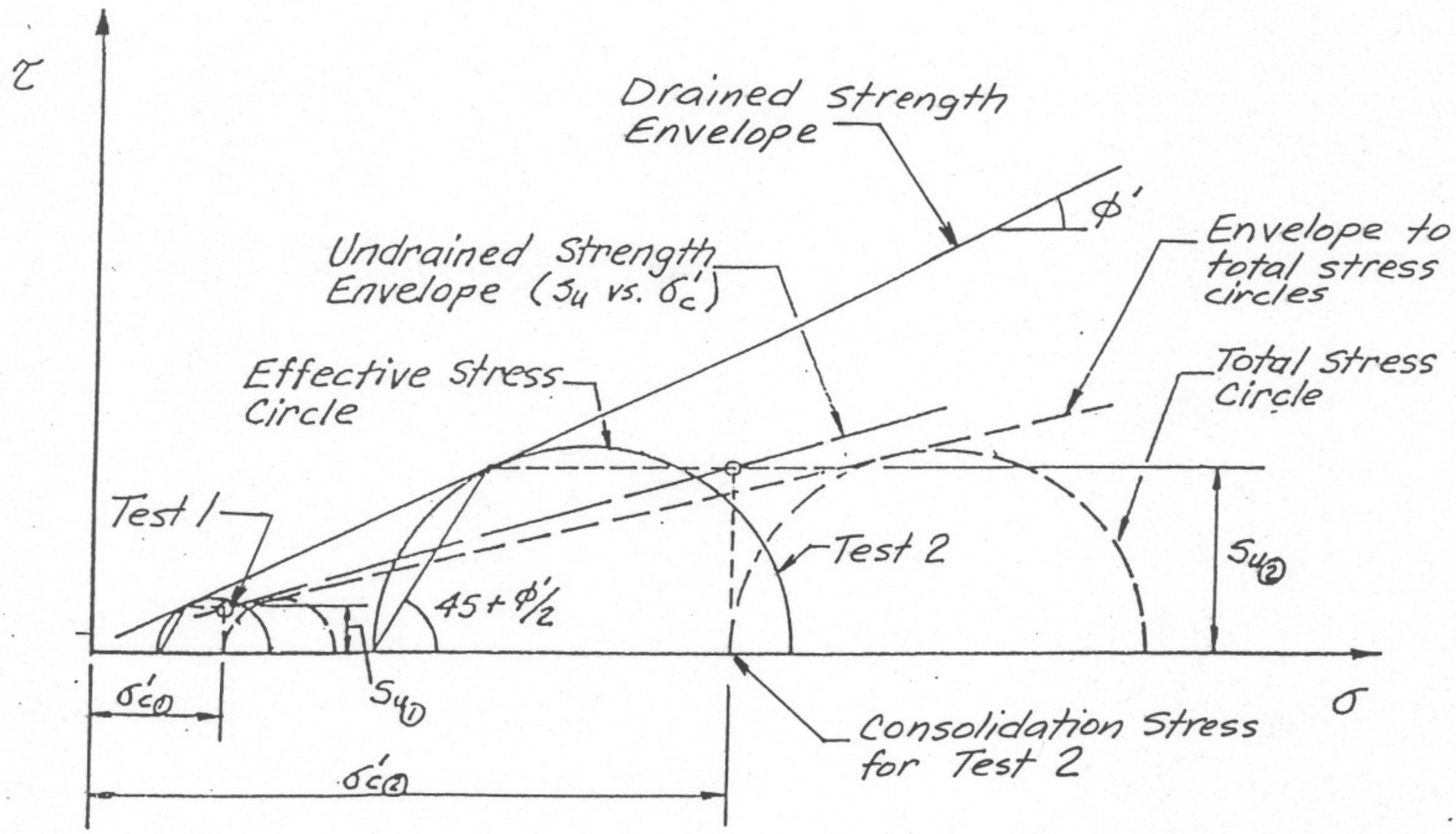
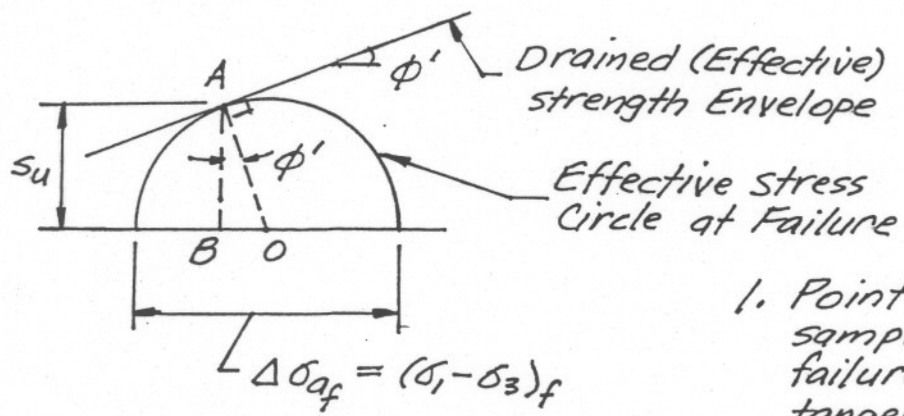

τ
For u_f = + (increase)
φ'
c'
σ
$u_f(+)$
σ'_c
σ'_{3f}
$\Delta\sigma_{af}$
σ'_{1f}
τ
For u_f = − (decrease)
φ'
c'
σ
σ'_c
$u_f(-)$
$\Delta\sigma_{af}$
σ'_{3f}
σ_{1f}

Figure 5



<u>Derivation of Equation for s_u:</u>

Consider Mohr circle for Test 2 in terms of effective stresses:



1. Point A represents plane in sample on which undrained failure occurs (= point of tangency of effective stress failure envelope)

2. OA is radius of failure circle $= \frac{1}{2}\Delta\sigma_{a_f}$

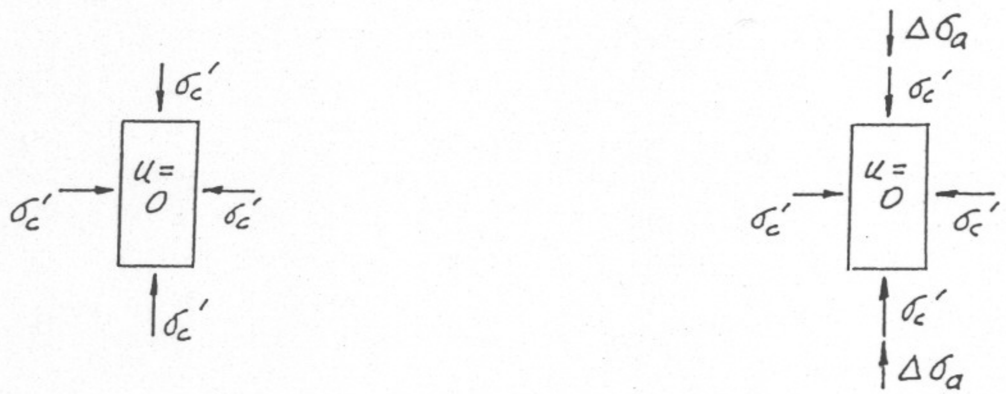
3. BA = shear stress on failure plane = s_u

4. From triangle OAB, $s_u = \frac{1}{2}\Delta\sigma_{a_f}\cos\phi'$

$$= \tfrac{1}{2}(\sigma_1 - \sigma_3)_f \cos\phi'$$
$$\cong \tfrac{1}{2}(\sigma_1 - \sigma_3)_f$$

Figure 6

Stage I (Consolidation) | Stage II (Axial Compression)



At failure (all stresses are effective stresses):

$$\sigma'_{3f} = \sigma_c'$$

$$\sigma'_{1f} = \sigma_c' + \Delta\sigma_{af}$$

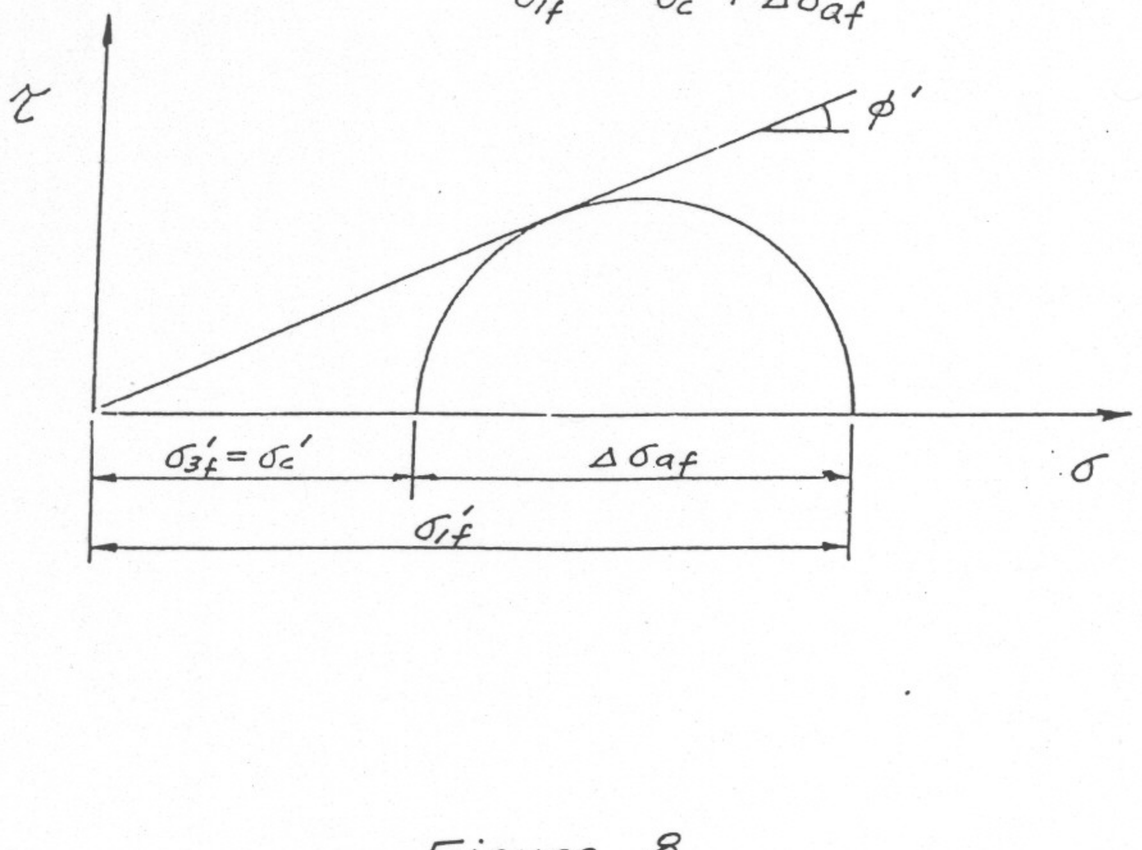


Figure 8

pressure saturated to ensure 100% saturation. Complete saturation can be verified by applying an increment of all-around pressure with the drainage line to the sample closed, measuring the pore pressure induced in the sample, and computing the B coefficient (observed pore pressure divided by increment of all-around pressure). The B coefficient should be at least 0.98 before conducting the axial loading stage of the test.

4-6.4.4 Consolidated-Drained Shear Strength (CD-type tests)

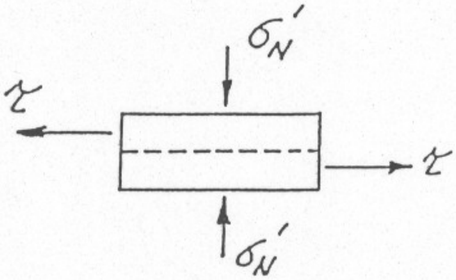
Consolidated drained triaxial tests are used to obtain the drained shear strength parameters, ϕ' and c'. Figure 8 shows the stresses applied in the test and the failure envelope. The sample drainage line is open throughout the test so that the sample is consolidated under the confining pressure σ_c' in the first stage of the test and so that the shear-induced pore pressures are zero during the second stage of the test. The drained strength envelope may exhibit slight curvature, but it may be approximated as linear with sufficient accuracy.

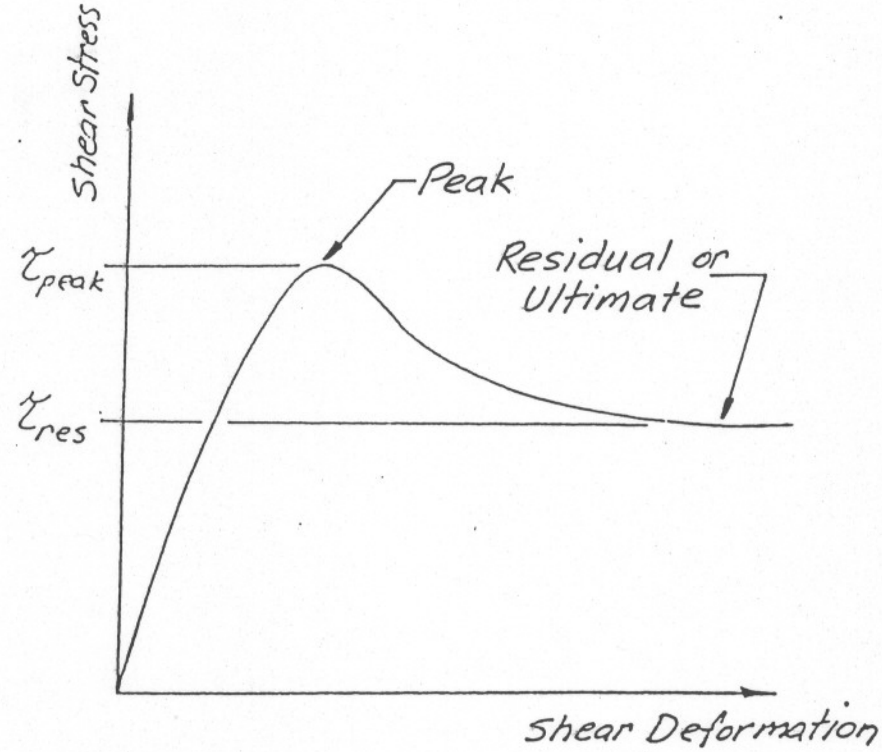
The drained strength parameters ϕ' and c' are appropriate for use in analyses for loading conditions in which the shear-induced pore pressures are zero. Examples would be steady-state seepage for all embankment and foundation soils (except for those natural soils such as overconsolidated clays or shales which may have been subject to past shear deformations and for which the residual shear strength, Section 4-6.4.5, would be appropriate), or end-of-construction or rapid drawdown loading conditions for pervious soils or for stiff, fissured impervious soils with high secondary permeability due to the fissures. The drained shear strength parameters from the consolidated-drained test can be used in lieu of the parameters determined in the consolidated-undrained test with pore pressure measurements to formulate the drained portion of the composite Mohr-Coulomb envelope for rapid drawdown analyses.

Since the drained shear strength parameters are a function largely of initial dry density for a given soil, samples should be prepared as discussed in Section 4-6.4.2. Confining pressures used in the test should encompass the range in effective normal stresses which will act along potential failure surfaces.

4-6.4.5 Residual Shear Strength

For foundations consisting of highly overconsolidated clays or shales, formations which contain shear zones or faults, or colluvium and landslide deposits, deformations along bedding planes, shear zones, faults, or landslide failure surfaces may have occurred in the geologic past such that the maximum available shear strength along these discontinuities is the residual shear strength. Under such conditions, direct shear or rotation shear tests may be used to determine the residual strength. Figure 9 shows the definition of the residual condition determined in a direct shear test and the residual angle of shearing resistance ϕ_{res}.





σ_N' = Effective stress on failure plane

τ_{peak} = Peak shear strength

τ_{res} = Residual shear strength

ϕ' = Drained angle of shearing resistance (peak)

ϕ_{res} = Residual angle of shearing resistance (ultimate)

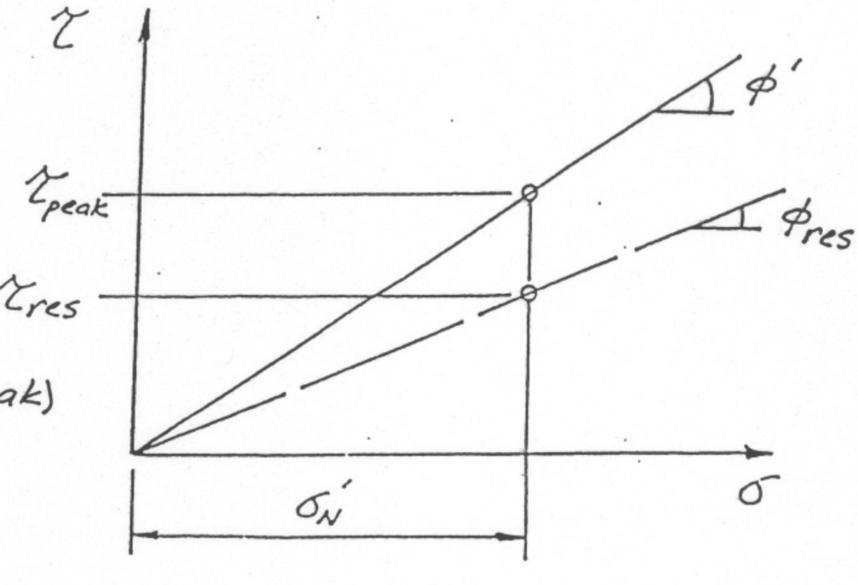


Figure 9

Samples should be oriented in the direct shear device such that shearing occurs parallel to the discontinuity so that the residual strength is measured; however, this is not always practical. Since the residual strength depends primarily on the mineralogy of the material along the discontinuity remolded samples may be used in direct shear or rotation shear devices. Remolded samples should be consolidated to normal stresses corresponding to the range in effective normal stresses which will act on the discontinuity in the problem to be analyzed. Because very large shear deformation may be required to establish the residual condition, there is sometimes a tendency in practice to stop the tests before sufficient deformation has occurred. This results in an overestimate, and hence an unconservative estimate, of ϕ_{res}. Thus, empirical correlations such as given in (Reference) should always be considered in determining the appropriate value of ϕ_{res}.

The direct shear test described in Figure 9 can also be used to determine the drained shear strength parameters ϕ' and c' if the peak shear strength is plotted versus the effective normal stress. The results are comparable to those obtained from CD or $\overline{CU}$ triaxial tests.

4-6.5 Note on Types of Stress Analyses and Terminology

In the past it was common to refer to two types of stability analyses: the "total stress", and the "effective stress" analyses. This terminology gives the erroneous impression that in the use of the so-called "total stress" analysis, one would select undrained shear strength values from the total stress envelope (as shown in Figure 6) by entering the horizontal axis with the total normal stress on the potential failure surface for which the shear strength is to be determined. If total stress Mohr circles from consolidated-undrained tests are used to plot a total stress Mohr-Coulomb envelope for these test results, then approximate undrained shear strength values could be obtained from this envelope by entering the horizontal axis with the initial <u>effective</u> normal stress (i.e. consolidation stress) on the potential failure surface for which the shear strength is to be determined; however, a preferable approach is to use the undrained strength envelope discussed subsequently. The total stress on the potential failure surface <u>should not</u> be used; this is a common mistake and is brought on by referring to this type of analysis as a "total" stress method.

It must be remembered that the only pore pressures inherently included in the "total" stress circles plotted from consolidated-undrained tests are those generated during the undrained shearing stage of the tests. The pore pressures due to static water levels or to steady-state seepage are not inherently included in these test results and this is the reason one must go into the total stress envelope with the total normal stress <u>minus</u> the water pressure due to static water level or from the steady-state seepage flow net to get the appropriate undrained shear strength.

The Corps of Engineers manual alerts the reader to this problem on pages 12 and 16 (Ref. 11). To avoid the problem, the first step is to never refer to an analysis as a total stress analysis, which in itself is a form of enticing a reader to make the wrong decision. Instead, the terminology "undrained strength analysis" should be used. The second step is to abandon the total stress Mohr-Coulomb envelope in preference for an undrained strength envelope as shown in Figure 6. The undrained strength envelope is a plot of the undrained shear strength developed on the failure plane (determined from consolidated-undrained tests) versus initial effective consolidation pressure on that same plane. Then, in an analysis in which the undrained shear strength is required, it is very straight forward to obtain the correct strength by entering the diagram at the initial "effective" consolidation stress on the potential failure surface in question.

While it is not incorrect technically to refer to an analysis utilizing the drained shear strength as an "effective" stress analysis, it is recommended that such analyses be referred to as "drained" analyses, or analyses in terms of "drained" shear strength. This notation will help to avoid the temptation to refer to the undrained strength analyses as total stress analyses. Theoretically, it is possible to carry out analyses in which failure is assumed to occur under undrained conditions (i.e. shear-induced pore pressures cannot dissipate and are therefore not zero), using effective stresses at failure along with "effective" stress shear strength parameters determined from CD or $\overline{\text{CU}}$ type triaxial tests. However, to do an analysis in this manner requires estimation of pore pressures caused by changes in all-around stress (functions of Skempton's B-coefficient), and shear-induced pore pressures (functions of Skempton's A-coefficient), as well as static and seepage pore pressures. Because of uncertainties involved in estimating field values of Skempton's A and B coefficients, this type of analysis is not recommended. The undrained strength analysis should be used instead.

4-6.6 Loading Conditions for Analysis and Selection of Shear Strength Values

As outlined in Section 4-6.3, an embankment may be subjected to several loading conditions during its life, ranging from construction to full pool operation. The loading conditions for which an embankment must be analyzed and the shear strength values appropriate for use in the analyses are presented in detail in the following paragraphs.

4-6.6.1 End of Construction Loading Condition

At the end of construction, an embankment dam may still be undergoing internal consolidation under its own weight. For homogeneous dams or for zones in dams constructed of impervious materials, pore water pressures will be built up during construction if the rate of consolidation of the embankment materials is slow compared to the rate of fill placement. Low permeability, natural foundation layers which are too thick to consolidate a significant amount during construction will also develop excess pore pressures caused by the weight of the embankment.

When such conditions are present, applicable shear strength values are determined from UU tests. For embankment materials, tests should be performed on laboratory samples compacted to specified dry densities at water contents within the range of compaction water contents permitted in the specifications, as discussed in Section 4-6.4.2. Confining pressures used in the tests should encompass the range in total normal stresses which would be present along potential failure surfaces in the dam.

For natural foundation soils, shear strength values can be determined from UU tests or unconfined compression tests on undisturbed samples, as discussed in Section 4-6.4.2. Shear strengths so determined will usually be conservative provided the foundation soils are normally consolidated or slightly overconsolidated such that embankment loading causes the maximum previous consolidation stress to be exceeded, because some consolidation and gain in strength will occur during construction. Evaluation of the time rate of consolidation and resulting gain in strength may show that staged construction would result in a significant increase in foundation strengths during construction and permit a more economical embankment design.

Overconsolidated natural foundation soils which are stiff and fissured should be tested in UU tests to determine end-of-construction shear strength because unconfined compression tests will give misleadingly low strengths for fissured materials. This is because premature failure will occur along the fissures under the condition of zero confining pressure. More importantly, these materials may fail under drained conditions (shear induced pore pressures are zero), even though the intact material between fissures may be of low permeability. The drained shear strength can be determined from CD or $\overline{CU}$

type tests and may be less than the strength determined from UU tests. In this case it will be more conservative to use the drained shear strength as the end-of-construction strength for such materials. See Section 4-6.4.2.

For pervious zones in the embankment where drainage can occur rapidly, drained shear strengths should be used in the analysis. These zones would include drains, filters, rock fill shells, and gravel shells.

The analysis that applies for this loading condition is the undrained strength analysis (or the drained strength analysis for stiff fissured materials or pervious zones as discussed in the preceding paragraphs). Because of the difficulty in estimating pore pressures within the embankment during this stage of loading, an effective stress analysis is not recommended. (See Section 4-6.5). An effective stress analysis may, however, be conducted using pore pressure responses in previously constructed dams that used materials, construction methods, and construction schedules similar to those for the proposed dam. In this type of analysis, shear strength parameters determined from CD or $\overline{CU}$ tests are used and effective stresses are calculated using the pore pressures observed in the similar previously constructed dams.

If there are any serious questions about stability during or at the end of construction, the only positive method to determine the stability is to install piezometers and evaluate the stability during construction using an effective stress analysis. The results of such an analysis may require alteration of the construction schedule or other methods to ensure stability.

4-6.6.2 Rapid Drawdown Loading Condition

In the rapid drawdown loading condition the embankment has been subjected to a prolonged high pool during which time a steady seepage condition has been established through the embankment. The soil in the embankment below the phreatic surface is in a completely saturated state and is fully consolidated under the weight of the overlying material. If subsequently the reservoir pool is drawn down faster than pore water can escape, excess pore water pressures and unbalanced seepage forces develop. Consequently, the factor of safety following a reservoir drawdown is reduced (Ref. 16, p. 370). This is usually the critical condition for design of the upstream portion of the embankment. Analyses for the rapid drawdown condition are based on the conservative assumptions that (1) pore pressure dissipation does not occur during drawdown, and (2) the reservoir water surface is lowered instantaneously from maximum pool, spillway crest, or top of gates to the minimum pool (Ref. 11, pg. 16). For core materials of low permeability the drawdown may take place during a period of days, or even weeks, and still be termed rapid or "sudden". The assumption of no drainage at all in the core material during drawdown is frequently a close approximation of actual conditions and is conservative (Ref. 40, pg. 538).

For embankments composed of impervious materials, the shear strength available to resist failure after drawdown is governed by the effective consolidation stresses along the potential failure surface for steady state seepage prior to drawdown. The shear strengths to be used should be the smaller of the drained or undrained strengths taken from the combined envelopes shown and explained in Section 4-6.4.3 (Ref. 11, pg. 16). This is conservative and ignores the cases where the undrained shear strengths are higher than the drained shear strengths because of negative pore pressures generated during shear.

Shear strengths of free-draining shell materials, defined as those in which drainage of pore water can proceed concurrently with lowering of the pool or with only a minor time lag, should be taken from the drained strength envelope (Ref. 11, pg. 16).

As discussed by Lowe and Karafiath, it has been found that the consolidated undrained shear strength of a clay soil depends on the principle stress ratio at the time of consolidation (Ref. 40, page 539). Principle stress ratios along a potential failure surface can be estimated for the high reservoir level immediately before drawdown. Anisotropically consolidated undrained triaxial tests can then be conducted at these principle stress ratios to determine the undrained shear strength for use in the after drawdown stability analyses. The reader is referred to Reference 40 for a detailed discussion of this method.

Duncan, et al. (Ref. 41) have developed a procedure for estimating the undrained shear strength resulting from anisotropic consolidation by interpolation from the results of isotropically consolidated tests. Because anisotropically consolidated tests are more difficult and expensive to perform than isotropically consolidated tests, the interpolation procedure makes the use of undrained shear strength resulting from anisotropic consolidation easier to use in practice, if it is desired to take into account the effects of anisotropic consolidation. The reader is referred to Reference 41 for a detailed discussion of the method.

Where the sudden drawdown loading condition controls the design of the upstream slope, and where the rapid drawdown assumption appears to be excessively conservative, consideration of possible drawdown rates along with permeabilities of relatively incompressible embankment materials can be used to construct flow nets. The flow nets can be used to determine pore pressures from which effective consolidation stresses are calculated to determine shear strengths. An approximate procedure for this type of analysis is given in Appendix III of Reference 11.

4-6.6.3 Steady Seepage Loading Condition

Steady seepage develops after a reservoir pool has been maintained at a particular elevation (e.g., maximum storage pool) for a sufficient length of time to establish a steady line of saturation through the embankment. The seepage forces which develop in the steady state condition act in a downstream direction, and therefore the condition of steady seepage through an embankment may be critical for downstream slope stability (Ref. 11, pg. 19). The upstream slope need not be analyzed for this condition if the upstream slope was designed for rapid drawdown. The seepage forces can be conservatively estimated by assuming a horizontal phreatic line from the elevation of the pool to the downstream limit of the impervious zone. However, high abutment groundwater tables may cause the phreatic surface to be higher in the vicinity of the abutments. In homogeneous impervious embankments, or when the assumption of a horizontal phreatic line in the impervious zone is overly conservative, the line of seepage can be estimated by various methods (Ref. 18, Ref. 19, Ref. 5). If sufficient instrumentation is available for an existing embankment dam, piezometer levels in both the embankment and foundation can be reviewed and phreatic surfaces can be developed accordingly.

The pore water pressures which exist within an embankment at any given time are the result of (1) static water levels or gravity seepage flow, and (2) changes in pore volume produced by changes in all-around stress or shear stress. In analysis of the steady seepage loading condition, pore water pressures due to changes in all-around stress are assumed to be zero (i.e. consolidation of the impervious embankment zones is assumed to be complete) and it is also likely that shear-induced pore water pressures in impervious zones are zero. This is because the steady seepage condition is assumed to develop slowly enough for the second category of pore water pressures to have dissipated. Thus the only pore water pressures assumed to be present in the embankment are those due to static water levels or gravity seepage flow.

Under these assumptions the shear strength available would be the drained shear strength, whereas if shear-induced pore pressures were not assumed to be zero, the undrained shear strengths would be used. For design purposes, however, the Corps of Engineers generally use the shear strength for impervious soils from a composite strength envelope midway between the drained and undrained envelopes, for effective normal stresses in the range where the drained strength is greater than the undrained strength (Ref. 11). This approach allows for the condition where positive shear-induced pore pressures in impervious embankment zones may not be completely dissipated and is conservative. For the range in effective normal stresses on a potential failure surface where the drained shear strength is less than the undrained shear strength, the drained shear strength is used. Figure 10 shows the composite design shear strength envelope. It is emphasized that the normal stress to be used is the effective normal stress on the potential failure surface. The shear strength of freely draining cohesionless soils should be taken from the drained strength envelope determined from CD or $\overline{\text{CU}}$ tests as discussed in Sections 4-6.4.3 and 4-6.4.4.

In the case of an old existing dam, where the steady seepage condition has been present for a long period of time under a more or less static reservoir level, it is likely that shear-induced pore pressures have completely dissipated. Use of the design envelope

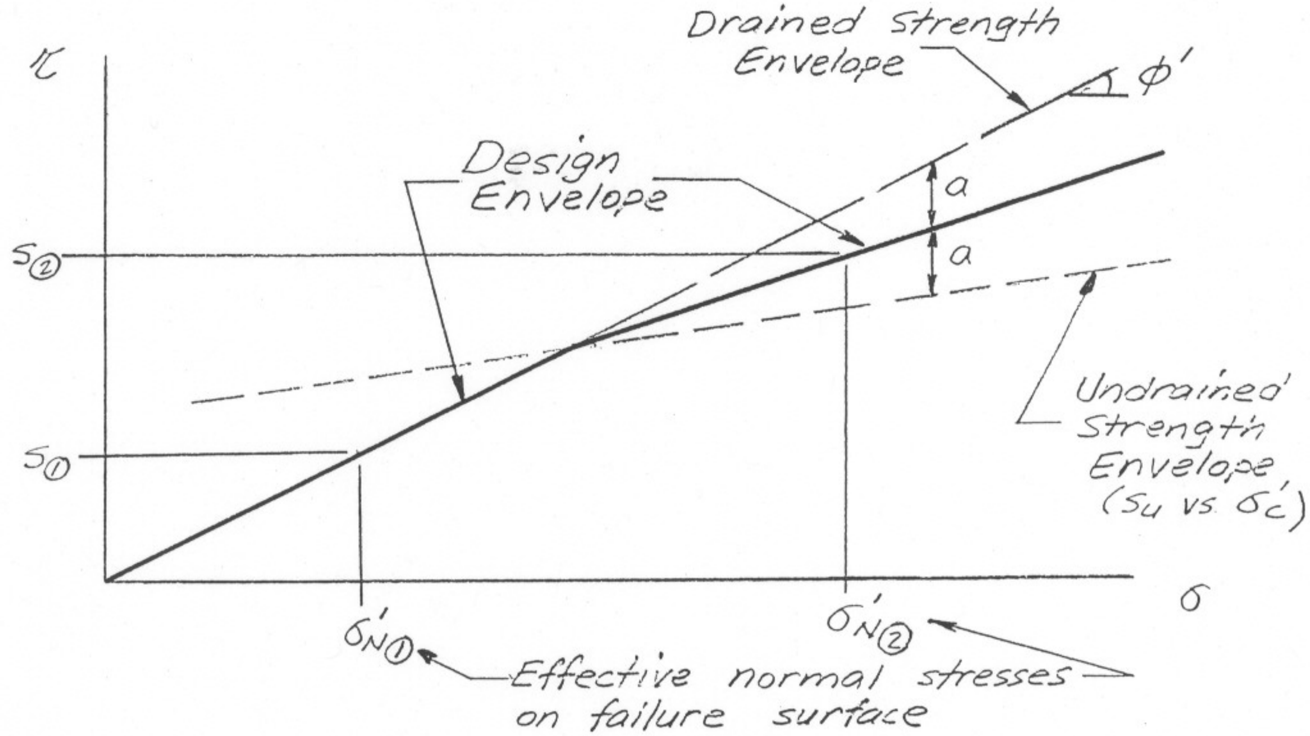

Drained Strength Envelope
φ'
τ
Design Envelope
a
a
s②
s①
Undrained Strength Envelope, (s_u vs. σ_c')
σ
σ'N①
σ'N②
Effective normal stresses on failure surface

Figure 10

shown in Figure 10, midway between the drained and undrained envelopes is probably overly conservative for compacted embankments. Thus, the drained shear strength envelope should be used for evaluating stability of dams in this category.

In the case where a steady seepage condition exists in an embankment, an additional horizontal thrust in the downstream direction may be imposed by a surcharge pool up to the probable maximum pool elevation. The surcharge pool should be considered as a temporary condition causing no saturation of impervious materials above the steady seepage phreatic line established under normal reservoir conditions. The shear strengths to be used in the stability analyses should be the same as used for the steady seepage condition with maximum storage pool.

4-6.6.4 Partial Pool Loading Conditions

Analyses of the upstream slope for intermediate reservoir stages should be conducted assuming that a steady seepage condition has developed at the intermediate stages. The shear strengths to be used should be the same as for the steady seepage condition for the downstream slope, discussed in Section 4-6.6.3. Stability analyses should be performed for several pool elevations and the factors of safety plotted as a function of reservoir level to determine the minimum factor of safety.

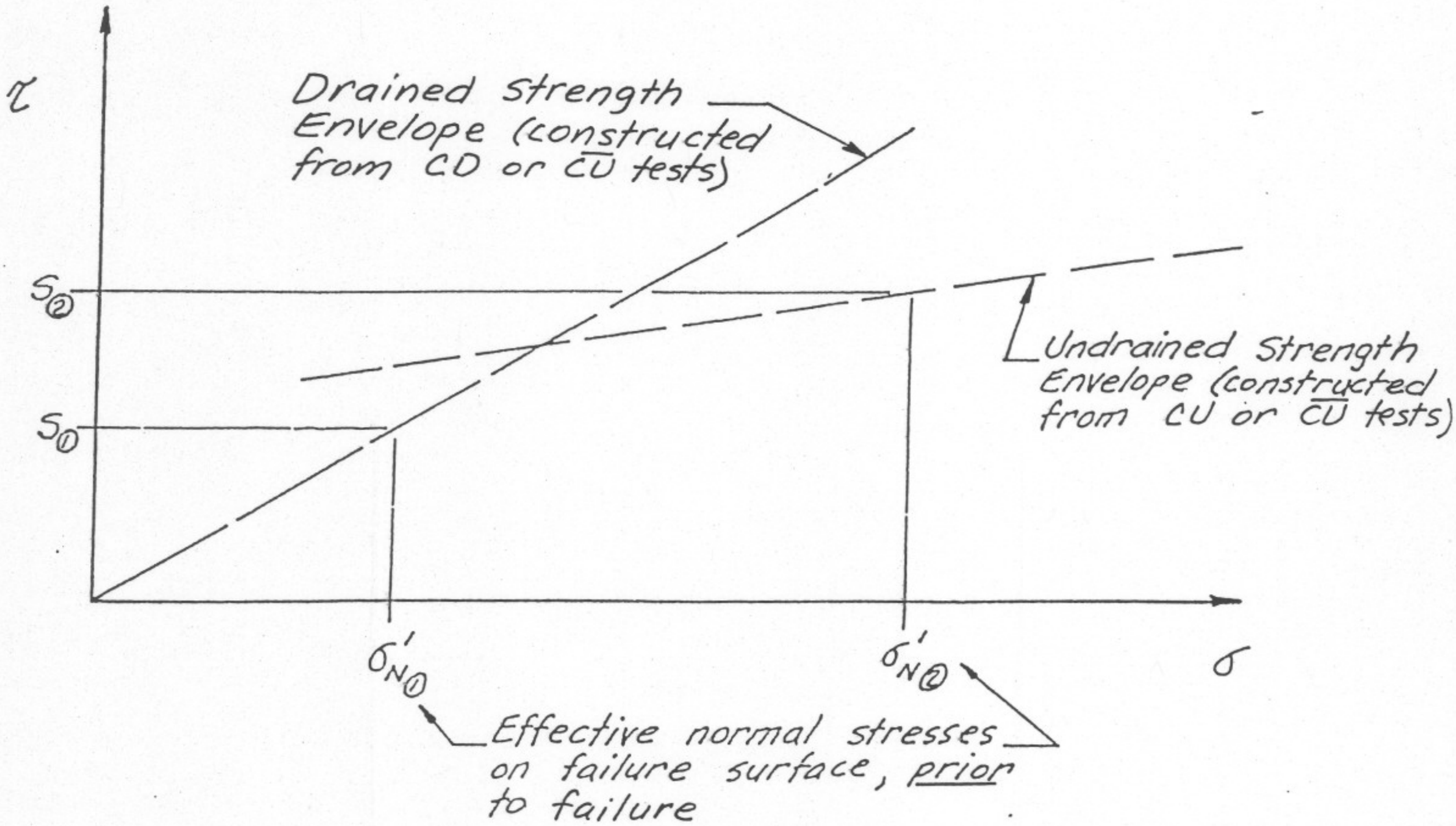

τ
Drained Strength Envelope (constructed from CD or C̄Ū tests)
S②
S①
Undrained Strength Envelope (constructed from CU or C̄Ū tests)
σ'N①
σ'N②
σ
Effective normal stresses on failure surface, prior to failure

Figure 7

Where pore water pressures developed during construction are not dissipated before a partial pool condition can develop, the reduced effective normal stress along a potential failure surface must be accounted for in the analysis.

4-6.7 Factors of Safety

The factor of safety provides a margin of safety to guard against ultimate failure, to avoid unacceptable deformations, to reduce the risk of progressive failure, and to cover uncertainties associated with the measurement of soil properties, the loading, or the analysis used (Ref. 22, pg. 48). In selecting a minimum acceptable factor of safety an evaluation should be made on both the degree of conservatism with which assumptions were made in choosing soil strength parameters, pore water pressures, and loading conditions; the consequences and the influence of the method of analysis which is used (Ref. 16, pp. 369-371). The latter concerns the method of calculation in which side earth forces are considered and how assumptions of directions of side earth forces affect stability analysis results. It also concerns assumptions relative to the dissipation of shear-induced pore pressures and whether anisotropic consolidation is accounted for in the determination of undrained shear strength.

An estimate of the factor of safety can be obtained by examining conditions of equilibrium when incipient failure is postulated, and comparing the shear stress necessary to maintain equilibrium with the available shear strength of the soil (Ref. 23).

Therefore, the slope stability analysis of embankments requires measurements of the shear strength and computation of the shear stress. Appropriate minimum values of factors of safety to be required in the stability analysis of a slope depend on the measurement of shear strength, likelihood of the assumed loading, consequences of failure, and assumptions in the method of analysis. Factors influencing the selection of minimum factors of safety include:

- Reliability of laboratory shear strength testing results
- Embankment height
- Storage capacity
- Thoroughness of investigations
- Construction quality, construction control of embankment fills
- Judgment based on past experience and knowledge of precedents
- Design conditions being analyzed
- Predictions of pore water pressures used in effective stress analyses

Minimum factors of safety generally required by FERC are listed in Table 1. The minimum Factors of Safety given in Table 1 are to be used in conjunction with analyses using peak shear strengths. If certain wedges involving foundation shales are analyzed using residual or near residual strength, then lower Factors of Safety than those given in Table 1 for the static cases are generally acceptable. The specific values being dependent on the percentage of the failure surface assumed to be at the residual strength.

Final accepted factors of safety will also depend upon the degree of confidence in the engineering data available. In the final analysis, the consequences of a failure with respect to human life, property damage, and impairment of project functions are important considerations in establishing factors of safety for specific investigations.

4-6.8 Static Stability Analyses

Various analytical methods for evaluating the static stability of an embankment dam exist. The method utilized in the licensee's analysis should be consistent with the anticipated mode of failure, dam cross section, and soil test data.

Many methods of stability analyses exist that use the same general approach of employing the "limit equilibrium method" of slope stability analysis. In this type of approach an estimate of factor of safety can be obtained by examining the conditions of equilibrium when incipient failure is postulated, and comparing the shear stress necessary to maintain equilibrium with the available shear strength of the soil. The factor of safety (F.S.) is thus defined as the ratio of the shear strength available (s) on the potential failure surface to the shear stress mobilized τ along the failure surface in order to maintain equilibrium(Ref. 24). This can be expressed as

$$\text{F.S.} = \frac{s}{\tau} \tag{1}$$

Table 1

Loading Condition	Minimum Factor of Safety	Slope to be Analyzed	Shear Strength Envelope
End of construction condition	1.3	upstream and downstream	
Sudden drawdown from maximum pool	>1.1*	upstream	
Sudden drawdown from spillway crest or top of gates	1.2*	upstream	
Steady seepage with maximum storage pool	1.5	upstream and downstream	
Steady seepage with surcharge pool	1.4	downstream	
Earthquake (for steady seepage conditions with seismic loading using a pseudo static lateral force coefficient)	> 1.0	upstream and downstream	

A state of limiting equilibrium exists when the shear strength mobilized is expressed as:

$$\tau_m = \frac{1\ (s)}{F.S.} \tag{2}$$

F.S. is the factor of safety with respect to shear failure and 1/F.S. is a measure of the degree of mobilization of the shear strength. It may be shown that the definition of F.S. given by equation (1) is equivalent to the one used in the Ordinary Method of Slices, where the factor of safety is defined as the ratio of the resisting moment to the driving moment (Ref. 25, page 784).

The drained shear strength of a soil is expressed by the following expression:

$$s = c' + \sigma_N' \tan \phi'$$

in which c' and ϕ' are the cohesion intercept and slope of the Mohr-Coulomb diagram and σ_N' represents the effective normal stress on the failure surface.

The undrained shear strength is given by the undrained shear strength envelope (Figure 6) which is a plot of s_u as a function of σ_c', where σ_c' is the effective consolidation stress on the failure surface, prior to the undrained failure. Thus, to determine the shear strength along a potential failure surface, the effective normal stress or consolidation stress on the failure surface must be known.

In analyzing both force and moment conditions of equilibrium it becomes apparent that the problem of determining the distribution of the effective normal stress or consolidation stress on the failure surface is statically indeterminate, that is, there are more unknowns than there are equations of equilibrium (Ref. 25). An approach to this situation is to make assumptions to reduce the number of unknowns in order that the problem is statically determinate, such as is done in the "limit equilibrium" analysis procedure. Different procedures use different assumptions. Some methods do not satisfy all conditions of moment and force equilibrium. Table 2 shows equilibrium conditions satisfied by various methods of analysis.

Studies have been performed to examine the accuracy of the equilibrium methods of slope stability analysis (Ref. 25, pp. 783-791, Ref. 26, pp.475-498). Except for the ordinary Method of Slices, which gives the lowest and hence most conservative values for the F.S., and the differences are probably within the range of uncertainty in the shear strength parameters.

Table 2

Equilibrium Conditions Satisfied

Procedure	Overall Moment	Individual Slice Moment	Vertical Force	Horizontal Force
Ordinary Method of Slices	Yes	No	No	No
Bishop's Modified Method	Yes	No	Yes	No
Corps of Engineers Modified Swedish Method	No	No	Yes	Yes
Janbu's Generalized Procedure of Slices	Yes	Yes	Yes	Yes
Spencer's Procedure	Yes	Yes	Yes	Yes
Morgenstern and Price	Yes	Yes	Yes	Yes

In Appendix 4C a step by step illustration is given for Stability Analyses for Steady State Seepage and Rapid Drawdown using the Corps of Engineers Modified Swedish Method. Actual calculations from a high central core, rockfill dam are shown in Appendix 4D.

Everything from p. 69-86
on is considered only a
very beginning draft.
Expect to change greatly.
ajH

4-6.9 Earthquake

Evaluation of seismic effects for embankments located in areas of low seismicity (0.10 or less) may be accomplished using the seismic coefficient in the pseudostatic method of analysis. Seismic coefficients at least as large as shown in figures 6, 6a, 6b, and 6c of Reference 11 shall be employed as applicable (Ref. 11, change 1, dated 17 February 1982). The pseudostatic method assumes that the earthquake causes additional horizontal forces in the direction of potential failure. This investigation need only be applied to those critical failure surfaces found in analyzing steady state seepage. An analysis of earthquake loading is never necessary in conjunction with sudden drawdown stability analysis for nearly all dams. 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. The selection of shear strengths to be used in the analysis are discussed in Section 4-7.

For embankments located in areas of strong seismicity, a dynamic analysis of embankment stability should be performed based on present state-of-the-art procedures. Refer to Corps of Engineers ER 1110-2-1806, "Earthquake Design and Analysis for Corps of Engineers Dams," for the earthquake loading to be used in dynamic analyses and for guidance in performing seismic evaluations.

In general, an embankment dam should be capable of retaining the reservoir under conditions induced by the maximum credible earthquake where failure would cause loss of life. The following investigations should be accomplished for all proposed and existing embankments, with the exception that existing confirmed "low" hazard potential dams may be exempted from these investigations.

- A seismic stability investigation using a dynamic analysis for proposed and existing dams located in Seismic Zones 3 and 4 of Reference 33.

- An evaluation of the liquefaction potential for all dams that have or will have liquefiable materials either in the embankment or foundation.

- A geological and seismological review of existing dams in Seismic Zones 2, 3, and 4 of Reference 33, to locate faults and ascertain the seismic history of the region around the dam and reservoir.

- A seismic stability investigation of existing dams by dynamic analyses, regardless of the seismic zone in which the dam is located where capable faults or recent earthquake epicenters are discovered within a distance where an earthquake could cause significant structural damage.

4-7 <u>Seismic Stability Evaluation</u>

4-7.1 General Approach

Various methods of analyses are available for evaluating the seismic stability of an earth dam. These may be classified as:

- Pseudostatic methods
- Deformation analyses for non liquefiable soils
- Evaluation of potentially liquefiable soils

Regardless of the method of analysis, the final evaluation of the seismic safety of the embankment should be based on all pertinent factors involved in the investigation and not solely on the numerical results of the analysis (Ref. 21). References presented in the Corps of Engineers ER 1110-2-1806 can be used in determining the scope of analysis required for properly assessing the seismic stability of an embankment dam.

- Analyses for earthquake loading should begin with simplified procedures and proceed to more rigorous methods of analyses as a particular situation may warrant. Projects with well compacted embankments and dense foundation soils located in Seismic Zones 1 or 2, (Ref. 33), and all confirmed low hazard potential projects, may be evaluated by the pseudostatic method using the seismic coefficient assigned to the seismic zone in which the project is located.

- In areas of severe and/or frequent seismic loading such as in seismic Zones 3 and 4 or where foundation liquefaction potential exists, more rigorous dynamic methods of analyses may be necessary (Ref, 33). Site specific seismic evaluations will be performed for all projects not covered in the paragraph above. These studies will identify earthquake source areas, the maximum credible earthquake, and the distance from the site of each relevant source area. Potential for fault rupture in the dam foundation and in the reservoir will be assessed. The modes of failure that need to be investigated and the appropriate methodology are described in the following subsections.

4-7.2 Modes of Failure:

a. Loss of Stability

The dam becomes unstable as a result of loss in strength in the dam or foundation when they are composed of loose constructive sands. – Liquefaction Slide – typical examples: Lower San Fernando Dam and Ft. Peck Dam. In the case that liquefaction may be triggered by earthquake motions, the post earthquake stability must be assessed.

b. Excessive Deformations

The dam remains stable during and after the earthquake; however, deformations can accumulate during earthquake shaking. 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. These deformations should be estimated by either the Newmark or Seed-Makdisi methods, for all dams in Seismic Zones 3 and 4.

c. Other Mechanisms

- Overtopping due to seiches
- Movements along a fault passing under the dam
- Landslides in abutments causing direct damage to the dam or due to a wave in reservoir caused by an earthquake induced slide.

4-7.3 Methods of Analyses

a. Pseudostatic Analysis Procedures

For many years the standard method of evaluating the safety of embankment dams against sliding during earthquakes has been the pseudostatic method of analysis. This approach should not be used for those cases where potentially liquefiable materials are estimated to be triggering along the potential failure surface.

In general, therefore, earthquake analyses using the seismic coefficient method may be performed only for structures proposed or existing in Seismic Zones 1 and 2. Seismic coefficients at least as large as shown in the Corps of Engineer ER 1110-2-1806, should be employed in the analysis (Ref. 33). In the analyses conducted with the seismic coefficient, the undrained shear strength should be used consistent with initial effective normal consolidation pressures determined from the conditions for steady state seepage at normal pool. The Factor of Safety computed using the seismic coefficient in a pseudo static analysis with the appropriate undrained strength should be at least 1.0, as indicated in Table 1.

b. <u>Deformation Analysis - Non Liquefiable Soils</u>

For those dams located in Seismic Zones 3 and 4 and for which neither the embankment or the foundations are composed of potentially liquefiable soils, the dynamic deformations should be calculated by either the Newmark Method (Ref.) or the Seed Makdisi Method (Ref.). For both methods, one common element or step in the calculation of the dynamic inelastic displacements, is the calculation of the yield acceleration of the sliding mass being evaluated. The yield acceleration is the minimum value of a pseudo static acceleration times the mass of the failure wedge which will just bring the potential failure mass to a Factor of Safety of 1.0 (i.e. the inertia force required to fully develop the undrained strength on the potential failure surface). In the evaluation of the yield acceleration the consolidation stresses on potential failure surfaces should be determined for the steady state seepage case at normal high reservoir. The undrained shear strengths selected should be consistent with these consolidation pressures along the potential failure surface.

The calculations of yield accelerations for various cases are illustrated in Appendix 4-~~D~~E. The deformations calculated for potential failure masses by the Newmark or Seed Makdisi Methods should normally not exceed 2 feet and should never be greater than about one-half of the filter thickness. Some investigators have made deformation calculations according to the methods described herein and have taken the free board as the allowable permissible deformation. Although keeping the calculated displacement less than the free board is a necessary condition, it is not a sufficient condition to assure safe performance of the dam. Loss of uncontrollable seepage and erosion is a possibility due to cracking at displacements smaller than the free board. The allowable inelastic displacements calculated by this method are about two feet and should not be more than one-half the thickness of the filter zones in any case.

Calculations of inelastic deformations utilizing the Newmark Method are given in Appendix 4-~~D~~E.

c. Evaluation of Potentially Liquefiable Soils

Embankments and/or foundations composed of sands, silty sands, non plastic silts, sandy silts and sandy gravels should be evaluated for the triggering of liquefaction for the design earthquake motions from the magnitude of earthquake selected for design. It is preferred that the evaluation for triggering be made on the basis of standard penetration test blow counts, as indicated by Seed and Idriss (Ref.), Seed et al. (Ref.), Seed and Harder (Ref.), and Youd et al (Ref.). In the past this method of relying on a correlation between the standard penetration values and liquefaction behavior has been erroneously referred to as a simplified procedure. In fact, when the term was first used (1971), the correlation between actual field behavior and standard penetration blow counts was the only credible method available correlating natural properties of the materials at key sites where liquefaction was observed with the ground motions measured or estimated from known magnitude earthquakes and the epicentral distance from the earthquake to the point where liquefaction behavior was observed. At that time some investigators and regulators thought that it was more appropriate and rational to obtain samples of sand, test them in cyclic triaxial test, and compare the cyclic strength ratios from the tests on samples with the cyclic stress ratios induced in the field as calculated from the design ground motion.

To appreciate these philosophical differences some history on the subject of liquefaction evaluation and soil mechanics is necessary.

Soil Mechanics has given Civil Engineers the tools for calculating the Factors of Safety for stability problems and the means for estimating deformations, seepage and porepressures in terms of the shear strength, compressibility, and permeability properties of the soil materials. The classical approach to many design problems in geotechnical engineering has consisted of:

1) obtaining representative samples
2) conducting the appropriate laboratory tests on the representative sample to obtain the strength, compressibility or permeability
3) analyzing the problem within the framework of soil mechanics and rock mechanics, utilizing the material properties obtained from samples in order to reach a conclusion helpful for design or evaluation.

Although this classical approach has worked where "representative" samples of the mass can be obtained and tested, this approach is not appropriate for applied problems in sands. The classical approach is not directly applicable for these cases because "representative" samples can not be reliably obtained without changing the density. For these cases the use of precedent in the form of key case histories is the most direct approach and soil mechanics is used to back-calculate strength, stiffness, and permeabilities from the actual field performance of soils. This approach is often labeled as an "empirical" method and wrongfully relegated to a second class status by those who favor the classical approach but fail to perceive that the "representative" sampling problem cannot be overcome.

The evaluation of liquefaction problems in the design or evaluation of Civil Engineering structures founded on sands sometimes results in a confrontation of the two approaches. The confrontation usually develops because one group of engineers believes that "representative" samples of sands can be obtained and tested to yield meaningful strength properties. Another group of engineers associated with the same project may favor an engineering decision based upon correlations between penetration tests and back-calculated strengths from key case histories or precedents. In Appendix 4-? two case histories are reviewed which illustrate the nature of the conflict; one concerns the design of a nuclear plant on sands, the other involves the evaluation of an existing dam in an area with earthquake potential.

Experience indicates that this philosophical choice occurs with great regularity on many civil engineering problems. The use of precedents in geotechnical problems however assumes a greater importance than in other areas of civil engineering because many of the idealized assumptions which are made in the classical approach may not be realized in practice. Therefore the "classical" approach should not be used, no matter how logical it may seem, unless it is substantiated by field behavior.

The current consideration of liquefaction potential in the design of embankment dams is largely due to the emphasis given by Professor Seed to the liquefaction failures observed in Chile caused by the earthquake of 1960. The failure of Ft. Peck Dam during construction in 1935 was due to the liquefaction of recently placed hydraulic fill, without the aid of an earthquake for a trigger; but, the significance of this event was not seriously incorporated into dam engineering practice from 1935 to 1964, as evidenced by the significant number of embankment dams constructed in earthquake areas during this period with the shells placed over loose alluvial sands. The observation of widespread liquefaction in the great Alaskan earthquake of 1964, and the Niigata earthquake of 1964 led to the more formal considerations given to liquefaction potential in the design of dams as we know them today.

In an effort to quantify the effects of the factors influencing liquefaction potential, Seed and Lee (1966, Ref.) published results from cyclic triaxial tests on Sacramento River sand which showed that the three most important variables affecting "initial" liquefaction, or the development of porepressures approaching the initial consolidation pressure, were:

1) the ratio of the dynamic cyclic shear stress, τ_d, to the initial consoliudation pressure, $\overline{\sigma}_o$.
2) the relative density of the sand, and
3) the number of cycles of the dynamic shear stress.

For relative densities less than 80%, and for 10 cycles, the relationship between the relative density and the cyclic stress ratio required to produce initial liquefaction was found to be linear and was given as:

$$\tau_{10} / \bar{\sigma}_o = \frac{R.D.\ \%}{200} \qquad \text{Eq. 1}$$

Seed and Idriss (1967, Ref.) [2] were the first to analyze the liquefaction cases at Niigata in terms of available standard penetration data and within the framework implied by Eq. 1. If relative densities were estimated from the blow counts, according to Gibbs and Holtz [3], it was shown by Seed and Idriss [4] that the strengths inferred from Eq. 1 had to be multiplied by ½ in order to predict the liquefaction that had occurred in the heavy damage zone. This correction factor later came to be known as, C_r, and was an empirical adjustment necessary to obtain agreement between field observations and cyclic triaxial tests on Sacramento River sand at the same relative density.

At that time, (1968), it is important to note that there was not one case history where samples were obtained and cyclically tested, from locations where earthquakes had shaken sand deposits and liquefaction-non-liquefaction areas observed. Yet it is a fact that on many projects the engineers were conducting liquefaction potential analyses for reactor foundations on sands based on cyclic triaxial tests of undisturbed samples obtained from borings. This procedure in fact would have been reasonable and logical in terms of soil mechanics if it could be assumed that "representative"undisturbed samples could be obtained. However, due to the fact that sampling tends to densify loose samples and dilate dense samples, the direct results of tests on undisturbed sand samples were suspect. In addition, the direct sampling and cyclic testing of unsaturated samples did not have a single precedent relating to areas affected by liquefaction at that point in time.

During that period, 1967-1970, the relationship shown in Fig. 1 for assessing liquefaction potential was used to assess liquefaction potential for nuclear reactor projects, Hendron (1967, Ref.). The plot was initially based upon the standard penetration tests obtained at the boundaries of liquefaction damage at Niigata in 1964 and was supplemented later by standard penetration tests obtained at the boundaries of liquefaction damage and no liquefaction observed from the Takachioki earthquake of 1967. In Fig. 1, τ_{max} was computed from

$$\tau_{max} = \frac{a_{max}}{g} \sigma_v \qquad \text{Eq. 2}$$

where σ_v is the total vertical stress, a_{max} is the peak acceleration of the ground motion at the ground surface, and g is the acceleration of gravity. The relative density in Fig. 1 is determined from the standard penetration values and the vertical effective stress by means of the Gibbs-Holtz correlations [3] as was done by Seed and Idriss [2]. The relationship given in Fig. 1 has the same form as Eq. 1, but the absolute value of the cyclic strength ratio as a function of relative density is determined from key locations where liquefaction-non-liquefaction behavior was observed and where ground motion measurements made it possible to estimate the levels of dynamic shear stresses. Although this plot was simple and did not contain many case histories, it did provide a framework for evaluation a new site by the use of the standard penetration test blow counts, which was the only index property available from key locations affected by liquefaction.

The summary of standard penetration data and of field performance observations from 35 locations by Seed and Idriss in their 1971 paper entitled "A Simplified Procedure for Evaluating Soil Liquefaction Potential," [5], represented a significant step forward. The heart of the simplified procedure was the use of the standard penetration test values, the known or approximated ground accelerations, and the Gibbs-Holtz correlation to interpret the observed behavior from the case histories.

A new site could then be evaluated from standard penetration test results, by the "Simplified Method." The name of the "simplified" method unfortunately implied that there was a more exact sophisticated method. This led some engineers at this time to believe that the cyclic strength ratio of a soil in the field was most appropriately determined by multiplying the triaxial cyclic strength ratio from tests of undisturbed samples by a C_r value ranging from 0.55 for low relative densities to 0.68 for high relative densities.

The observation of case histories interpreted according to the simplified method ultimately took the form shown in Fig. 2 as published by Seed, Arango, and Chan [6] and was refined to Fig. 3 by Seed, Idriss and Arango, [7], and by Seed, Tokimatsu, Harder and Chung [8]. Note that in Fig. 3 that the standard penetration test data are corrected to 1 tsf based on tests by Marcuson and Bieganousky, [9], and that the corrections to the blow counts to 1 tsf in Fig. 2 are based on tests by Gibbs and Holtz [3]. In Figs. 2 and 3 τ_{ave} is equal to 0.65 τ_{max}.

Figure 3 is considered to represent the state of the art at present for determining the liquefaction in sands and the build-up of porewater pressures leading to the onset or triggering of liquefaction where r_u approaches 100% for level ground conditions. The method was extended to include embankments and sloping ground conditions as given by Seed [10]. In this publication H. B. Seed introduced the correction factors K_σ and K_α to account for the stress level and the slope of the embankment, respectively.

Even if liquefaction is "triggered" and the r_u values from cyclic strains approach 100%, the shear strength of the sand is not zero if the driving stresses due to a slope or due to the foundation loads of a structure cause monotonic strains after triggering. As has been correctly pointed out by Poulos, Castro, and France [11], there is a finite undrained steady-state shear strength in a contractive soil which has been triggered. Theoretically, if stability is the only concern, and not deformation, then an embankment or slope can be evaluated by comparing the driving shear stresses with the undrained steady-state shear strength of the sand. As it turns out in practice however, the evaluation of the steady state undrained shear strength is a sensitive problem greatly affected by the changes in void ratio which occur during sampling. Large corrections must be applied to the undrained steady-state shear strength measured to account for those void ratio changes. (It is also possible that in natural deposits a re-distribution of water content takes place within sand layers in the field leading to lower strengths along some boundaries even though on the average the layer is "globally" undrained. This possibility was discussed by Seed [12].

For structures which can take deformations, such as embankment dams, Castro et al [13] and Poulos et al [11] suggested that if stability is the concern the Factor of Safety for a triggered, contractive sand can be computed. A comparison of the driving stresses and the undrained-steady state shear strength can be made. Poulos et al [11] further suggest that the appropriate method for determining the undrained steady-state shear strength is by means of sampling, testing and correcting the tested strength for void ratio changes.

Because of the large corrections which need to be made to the actual test results, it is a big step to accept the procedures suggested by Poulos et al [11] for such a critical structure as an embankment dam, even if it is capable of taking significant deformations. Seed [12] has analyzed a group of Case Histories where the undrained residual strength was back-calculated from case histories where major movements occurred due to liquefaction; the results of that study were reported in a recent re-evaluation of the Lower San Fernando dam by Seed, Seed, Harder and Tong [14] as shown in Fig. 4, where the undrained residual strength back-calculated from the liquefaction failures is plotted versus

N_1. It should be noted that the back-calculated strengths are very low. A comparison of the back-calculated strength and laboratory strengths from undisturbed samples corrected to the 1971 pre-slide condition are shown in Fig. 5. The general trend is for the back-calculated strength to be lower than the strengths determined from the testing of undisturbed samples.

Although more research may change the state of the art, it is apparent that most of our present experience for the "triggering" of liquefaction and for the available undrained residual strength after triggering is in terms of the standard penetration test. Professor Seed and his co-workers have combined precedent and soil mechanics in a most meaningful way in order to make adequate engineering judgements in these problems. The "classical" approach in these problems is hampered by the difficulty in recovering undisturbed representative samples of sand, a fact recognized by Terzaghi and Peck [15] in their initial treatment of the subject of static settlement of foundations on sands.

Following a detailed study of embankment dam performance during earthquakes, (Ref. 26, pg. 227), Seed observed that the seismic resistance of dams constructed of clayey soils is much higher than that of embankments constructed of saturated sands or other cohesionless soils. Thus for embankments which do not involve saturated cohesionless soils, the pseudostatic method of analysis may still be used; alternatively, methods for evaluating deformations in such dams have been developed. The computed displacements can be compared to allowable displacements to determine the adequacy of the embankment (See 4-7.3.d). Methods for evaluating deformations during shaking have been developed by Seed and Newmark (Ref. 28, Ref. 29).

When embankments and/or their foundations are composed of loose sands, silts, or gravels, the pseudostatic method may not be applicable. Therefore, analyses must be performed to determine (a) if liquefaction potential exists and (b) whether such a liquefied condition can lead to failure or excessive deformations of an embankment. There are various simplified methods available for evaluating soil liquefaction potential (Ref. 29, Ref. 30, Ref. 13) based on empirical correlations between in situ behavior of sands and standard penetration resistance.

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Reference list is being supplemented

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APPENDICES

APPENDIX 4-A

ENGINEERING DATA

Being Revised

APPENDIX 4-A

ENGINEERING DATA

This appendix lists engineering data which should be collected relating to the design, construction, and operation of an embankment dam to be used in establishing the adequacy of embankment structures.

1. General Project Data

   a. Construction dates.

   b. Design of structures.

   c. As-built drawings indicating plans, elevations, and sections of embankment and appurtenant structures.

   d. Information on any modifications made, if applicable, such as dam raising.

2. Geotechnical Data

   a. Regional and site seismicity.

   b. Foundation data and geological features including logs or borings, geological profiles and cross sections, and reports of foundation treatment.

   c. Engineering properties assigned to construction materials and the foundation for design purposes including results of laboratory tests, field permeability tests, construction control tests, and assumed design properties for materials.

3. Construction History

   a. Construction procedures and methods used.

   b. Properties and characteristics of construction materials.

   c. How was quality control measured and maintained?

   d. Final foundation and embankment reports.

4. <u>Operation and Maintenance Records</u>

   a. Performance record to date based on instrumentation observations and surveillance reports.

   b. Comparison of conditions to which embankment has been subjected, to those assumed in the original design.

   c. Remedial measures undertaken during life of project.

   d. Known deficiencies and any work underway to correct deficiencies.

5. <u>Inspection History</u>

   a. Operation inspections reports.

   b. Safety inspections reports.

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APPENDIX 4-B

Filter Requirements and Criteria

This is still in
progress
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APPENDIX 4-C

General Step by Step Procedures

1) Steady State Seepage

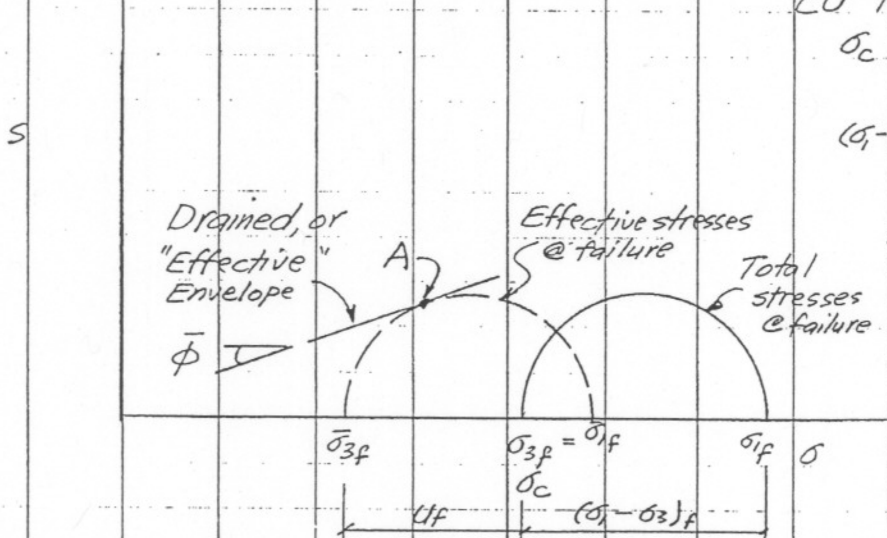
2) Rapid Drawdown

APPENDIX 4-C

Steady State Seepage

Stability Analysis Procedure – Steady Seepage

1. Shear strength envelopes:
   - Pervious materials, including shells, filters, transitions, use <u>drained</u> <u>envelope</u> with shear strength parameters $\bar{\Phi}$ and $\bar{c}$
   - Impervious core: construct <u>drained</u> envelope with shear strength parameters $\bar{\Phi}, \bar{c}$ and on same axes, construct <u>undrained</u> envelope, s_u vs σ_c, where σ_c is the consolidation pressure used in a $\overline{CU}$ triaxial test and s_u is as defined below:



$\overline{CU}$ triaxial test:

σ_c = consolidation pressure in Stage I

$(\sigma_1 - \sigma_3)_f$ = stress difference at failure, equal to peak on $(\sigma_1 - \sigma_3)$ vs ϵ curve, or if no peak, $(\sigma_1 - \sigma_3)$ at $\epsilon = 20\%$

u_f = shear-induced pore pressure at failure, ie at ϵ corresponding to $(\sigma_1 - \sigma_3)_f$

(Note $u_f = +$ is shown in diagram. u_f may also be $-$)

At failure: $\sigma_{3f} = \sigma_c$

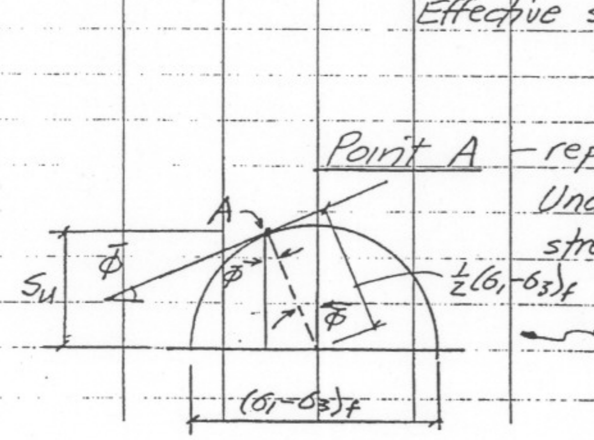
$\sigma_{1f} = \sigma_3 + (\sigma_1 - \sigma_3)_f$

Effective stresses: $\bar{\sigma}_{3f} = \sigma_{3f} - u_f = \sigma_c - u_f$

$\bar{\sigma}_{1f} = \bar{\sigma}_{3f} + (\sigma_1 - \sigma_3)_f$

(If u_f is $-$, then $\bar{\sigma}_{3f} = \sigma_{3f} - (-u_f)$, $\bar{\sigma}_{3f} = \sigma_{3f} + u_f$)

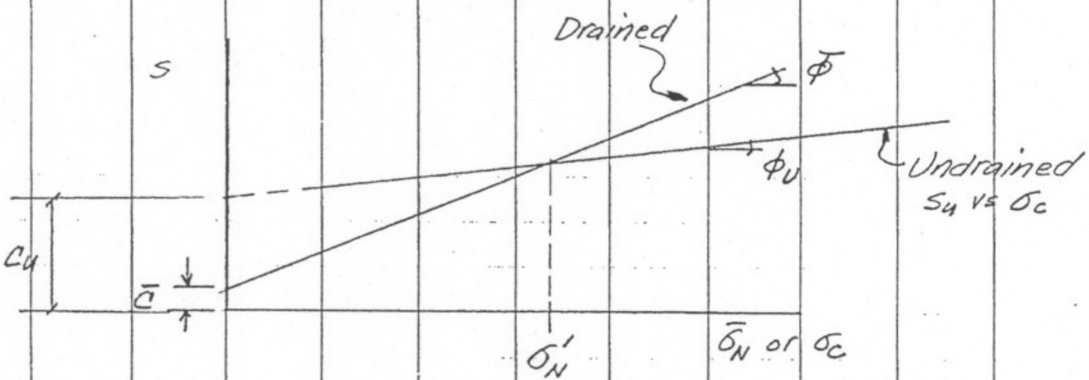
<u>Point A</u> – represents failure plane in sample. Undrained shear strength s_u is shear stress on the plane represented by pt A



From geometry: $s_u = \frac{1}{2}(\sigma_1 - \sigma_3)_f \cos \bar{\Phi}$

Undrained envelope is s_u vs σ_c. Note that σ_c is the effective normal stress on the failure plane <u>prior</u> to failure (for isotropic consolidation).

Drained and undrained envelopes:



Note that drained and undrained envelopes will intersect at normal stress $= \sigma_N'$

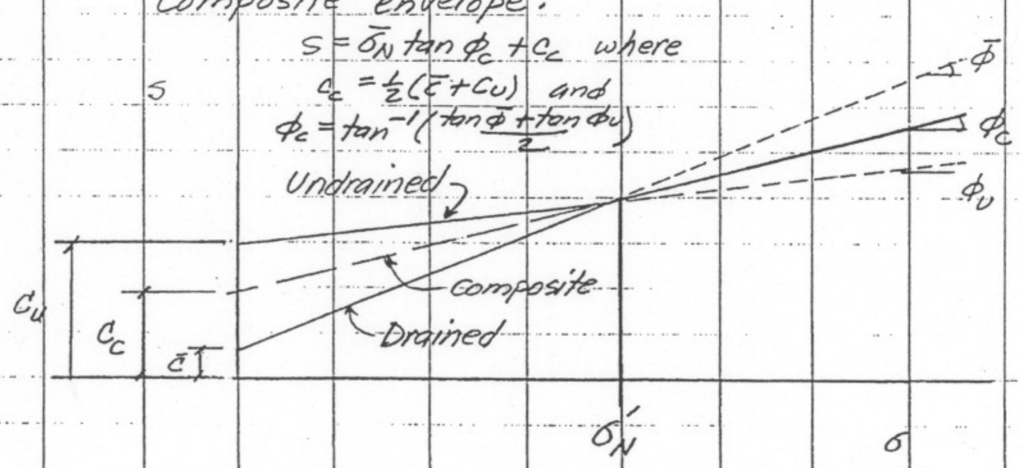
- Prepare composite envelope for impervious using drained envelope for all $\bar{\sigma}_N < \sigma_N'$ and <u>average</u> of drained and undrained envelopes for $\bar{\sigma}_N > \sigma_N'$. Use of the composite envelope for $\bar{\sigma}_N > \sigma_N'$ (in the range where the drained strength is larger than the undrained) in stability analyses allows for the possibility of incomplete dissipation of shear induced pore pressures.

<u>Composite envelope</u>:

$s = \bar{\sigma}_N \tan\phi_c + c_c$ where

$c_c = \frac{1}{2}(\bar{c} + c_u)$ and

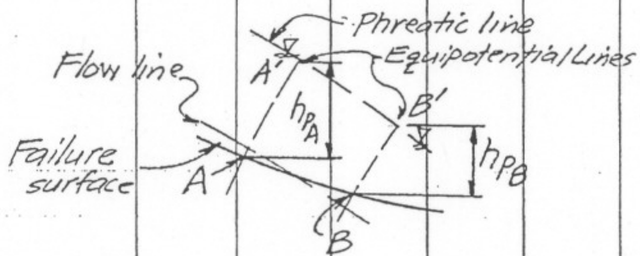
$\phi_c = \tan^{-1}\left(\frac{\tan\bar{\phi} + \tan\phi_u}{2}\right)$



2. Porewater pressures due to steady seepage:

- Obtain pore pressures on failure surface from steady-state flow net or from reservoir or tailwater level

From flow net:

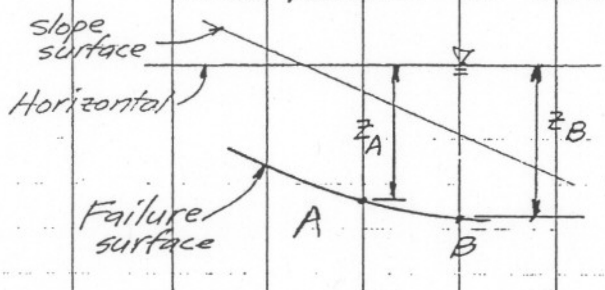


h_{p_A} & h_{p_B} are pressure heads at pts A & B, where A & B are any two points on failure surface at which u is desired.

h_p at any point such as A or B is the difference in elevation between the intersection of an equipotential line with the point, and the intersection of the same equipotential line with the phreatic line (e.g. A' or B' in sketch)

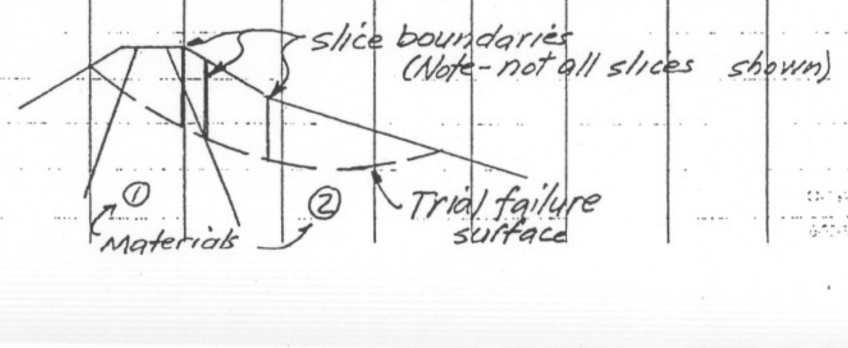
$$u = h_p \cdot \gamma_w$$

From reservoir or tailwater level:



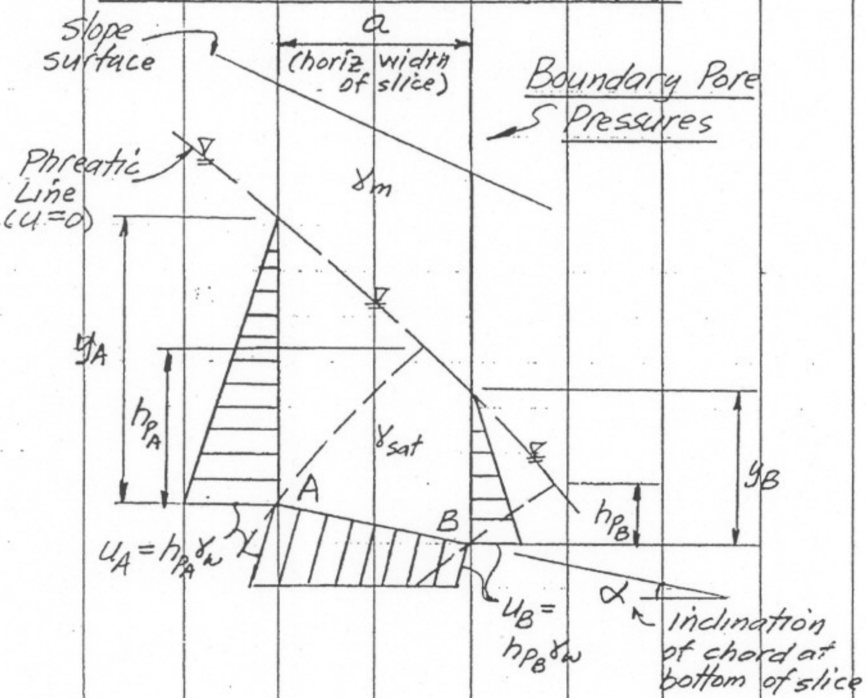
$u = z\gamma_w$ where z = vertical distance from horizontal water level to point where u is desired

3. Divide slope to be analyzed into vertical slices having approximately the same widths. Typically, 8 to 12 slices is sufficient. Take vertical slice boundaries at changes in material properties along the trial failure surface and, to facilitate weight calculations, at changes in the slope angle:



4. Find resultant of total weight and pore pressures acting on boundaries of each slice:

slices subject to steady seepage:



- Find pore pressures acting on boundary of slice (see #2.)
- Pore pressure equals zero at phreatic line
- Pore pressures act normal to slice boundaries
- Typically it is sufficiently accurate to assume linear variation in u from phreatic line to lower slice corners, and linear variation in u across bottom of slice

- Find resultant forces on slice boundaries due to pore pressures: (see sketch)

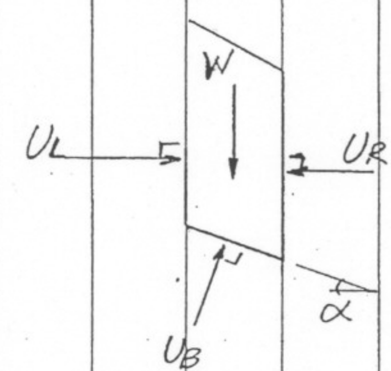
U_L = porewater force on left side of slice
$= \frac{1}{2} h_{P_A} \gamma_w z_A$

U_R = porewater force on right side of slice
$= \frac{1}{2} h_{P_B} \gamma_w z_B$

U_B = porewater force or bottom of slice
$= \frac{1}{2}(h_{P_A} + h_{P_B})\gamma_w \cdot \frac{a}{\cos\alpha}$ (Note $\frac{a}{\cos\alpha}$ = length of chord at bottom of slice)

- Find total weight of slice
W_{TOT} = (Area above phreatic line)$\cdot \gamma_m$ + (Area below phreatic line)$\cdot \gamma_{sat}$

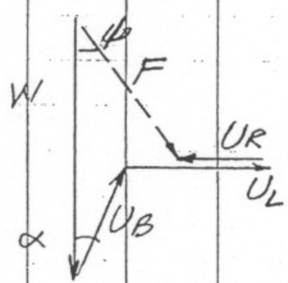
- Resultant forces on slice :



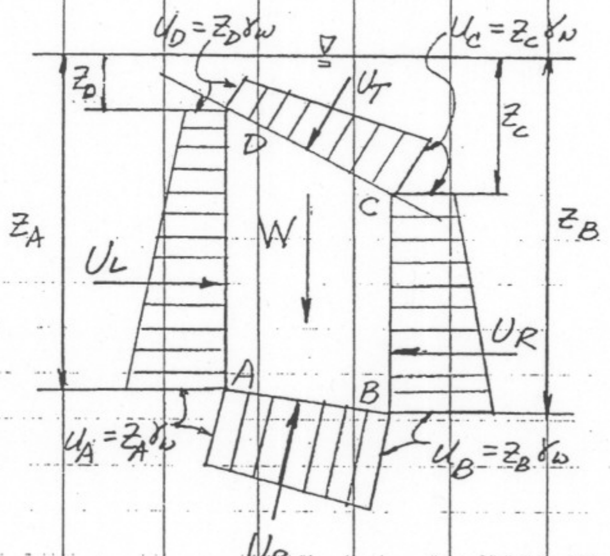
(Note porewater forces U_L, U_R, U_B act normal to slice boundaries)

Find resultant of W, U_L, U_R, U_B by ΣF_H and ΣF_V, or from force polygon. Resultant is "F" acting at "ψ" to vertical.

Example force polygon:



Horizontal Water Level:



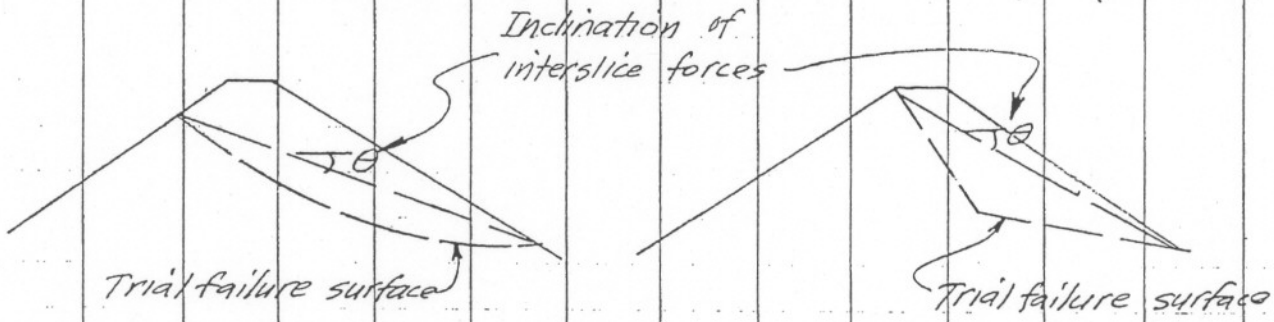
U_T = porewater force on top of slice $= \frac{1}{2}(u_D + u_C) \cdot \overline{DC}$

Other forces as defined above

Find resultant of W, U_T, U_L, U_R & U_B as explained above

Note that for hydrostatic condition where water level above slice is horizontal, resultant of the total weight and the total boundary porewater pressures is the submerged weight, ie $F = W'$ and $\psi = 0$ where W' = submerged weight of slice = area of slice γ' (where $\gamma' = \gamma_{sat} - \gamma_w$)

5. Assume interslice forces act an inclination given by the slope of a chord (or line) connecting the intersections of the trial failure surface and the slope surface:



6. Draw force polygon for assumed factor of safety:
   - Assume a factor of safety and compute ϕ_m and C_m for each slice, where:

$\phi_m = \tan^{-1}\left(\frac{\tan\bar{\phi}}{FS}\right)$ where FS = assumed factor of safety & ϕ_m = mobilized friction angle at the base of the slice

$C_m = \bar{c}\,\frac{a}{\cos\alpha}\cdot\frac{1}{FS}$ where C_M = mobilized cohesion at the base of the slice, $\bar{c}$ = unit cohesion intercept, a = horizontal width of the slice, α = inclination of the chord at the base of the slice ∴ $a/\cos\alpha$ = length of the chord at the base of the slice.

Note that FS is defined as T_{MAX}/T_{eq} where T_{max} is the maximum shear force available along the failure surface and T_{eq} is the shear force required for equilibrium.

For one slice, $T_{max} = \bar{N}\tan\bar{\phi} + \bar{C}$ where $\bar{N}$ is the normal force at the base of the slice and $\bar{C}$ is the cohesive force

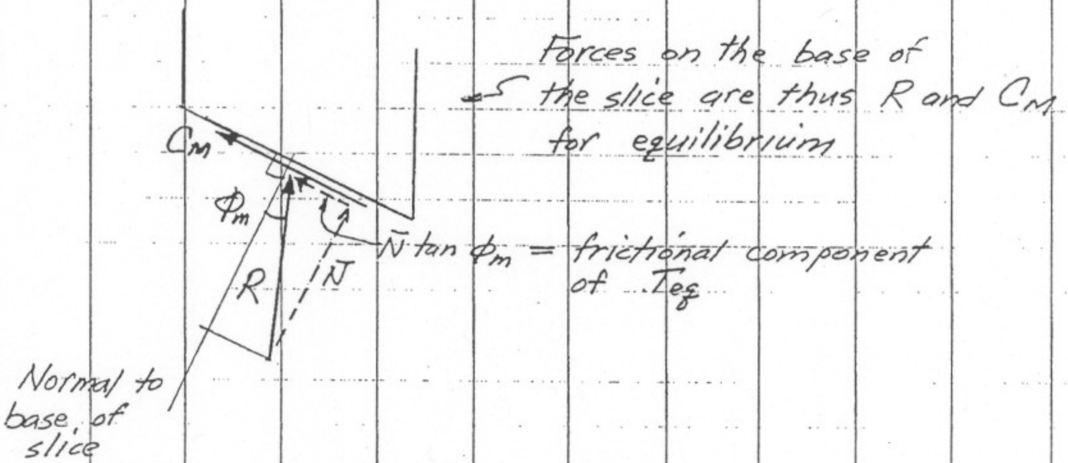
$$T_{eq} = \frac{T_{MAX}}{FS} = \bar{N}\frac{\tan\bar{\phi}}{FS} + \frac{\bar{C}}{FS}$$

or $T_{eq} = \bar{N}\tan\phi_m + C_m$ where ϕ_m & C_m are as defined above

- Define R as the resultant of $\bar{N}$ and the "frictional" component of T_{eq} (ie $\bar{N} \tan \phi_m$)

  Thus R must make the angle ϕ_m with the normal to the base of the slice

  C_m acts parallel to the base of the slice



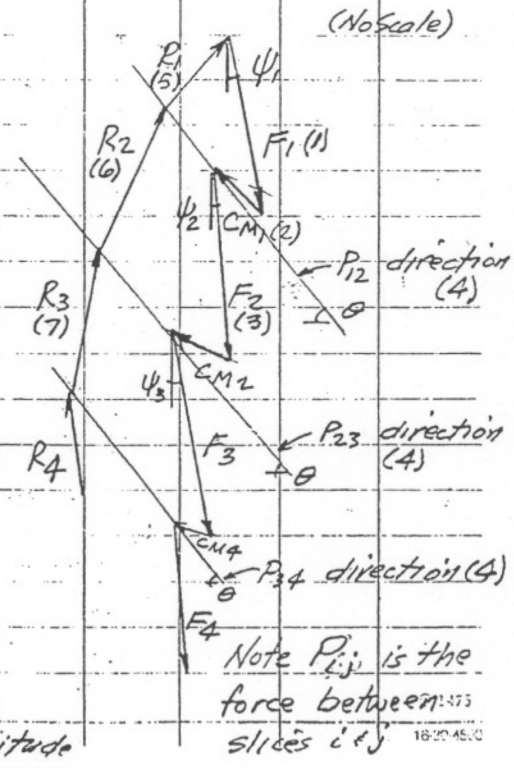
(1) - Begin force polygon by drawing F_1 vector at ψ_1 to vertical for slice 1

(2) - Draw C_{M_1} for slice 1 (if any) parallel to the chord at the base of slice 1, starting at the head of the F_1 vector

(3) - Draw F_2 at ψ_2 beginning at the head of the C_{M_1} vector

Continue untill all the F & C_M vectors are drawn

(4) - Draw the interslice forces P at the head of each C_M vector, or, if C_M is zero for a slice, at the head of the F vector (except for the last slice) Note the magnitude of the P vectors are unknown



Note P_{ij} is the force between slices i & j

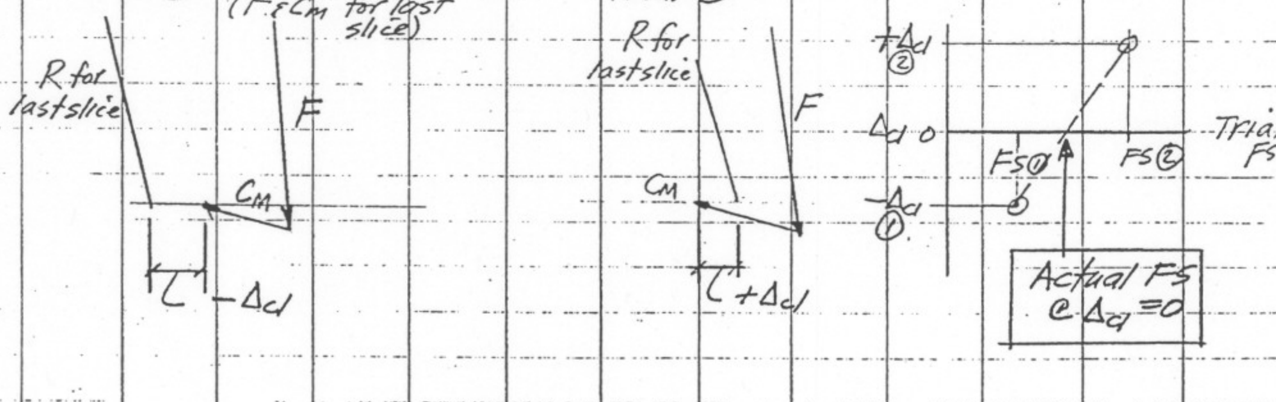
(5) - Begin with slice 1 and draw the R_1 vector for slice 1 at ϕ_{m_1} to the normal to the base of slice 1. The head of the R_1 vector begins at the tail of the F_1 vector and extends until it intersects the interslice force vector P_{12}.

(6) - At the intersection of the R_1 and P_{12} vectors, begin the R_2 vector at ϕ_{m_2} to the normal to the base of slice 2. Continue the R_2 vector until it intersects the interslice force vector P_{23}.

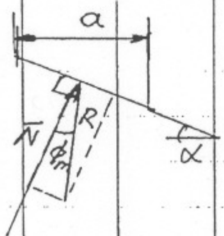
(7) - Continue drawing the R vectors in this manner until the last R vector is drawn.

If the R vector for the last slice just intersects the head of the C_M vector for the last slice, or the head of the F vector for the last slice if C_M is zero for the last slice, then the assumed FS is the actual FS. If the force polygon does not close, then assume a new trial FS and repeat until the force polygon closes.

A convenient way to interpret the results and to select the trial FS, after at least two trials are completed, is to define an error of closure Δ_{cl} and to plot Δ_{cl} vs FS:



7. After closed force polygon is obtained assuming the drained envelope controls ($\bar{\Phi}, \bar{c}$), determine the normal effective stress at the base of each slice in the impervious core to determine if the composite drained & undrained envelope may control. (see I. "Composite Envelope). The normal effective stress at the base of a slice, $\bar{\sigma}_N$, can be found by



$$\bar{\sigma}_N = \frac{R \cos \phi_m}{a/\cos\alpha}$$

where R & ϕ_m are from the closed force polygon and $R\cos\phi_m$ is the resultant effective normal <u>force</u> and $a/\cos\alpha$ is the chord length

If $\bar{\sigma}_N$ is larger than σ_N' for any slice in the impervious material, use the shear strength parameters for the Composite Envelope for those slices and a new FS. For the slices where the composite envelope controls, Φ_m will be determined from Φ_c and C_M from C_c

Make a final check of the closed force polygon to make sure that $\bar{\sigma}_N$ for the slices in the impervious material is still greater than σ_N' so that the composite envelope is still appropriate.

APPENDIX 4-C

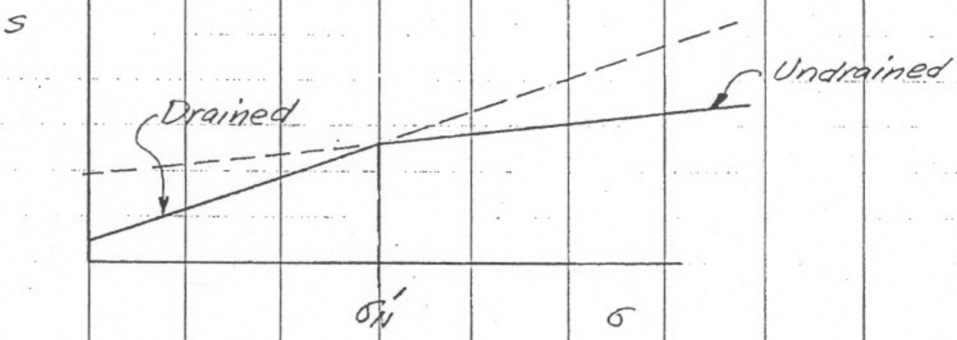
Rapid Drawdown

Stability Analyis Procedue – Rapid Drawdown

1. Shear strength envelopes:

   – Pervious materials, use drained envelope $(\bar{\phi}, \bar{c})$

   – Impervious core, construct drained and undrained envelopes as explained for steady seepage case;



2. Determine the FS for the full reservoir condition by the closed force polygon, as explained for the steady seepage case, except use the drained shear strength envelopes for all materials, including the impervious materials (i.e. do not use the composite envelope). Also assume that the full reservoir level extends horizontally into the embankment so that pore pressures along the failure surface will be hydrostatic. Thus the F vectors will equal the sum of the total weight above the water level and the submerged weight below the water level, and ϕ will be zero.

3. From the closed force polygon, determine the normal effective stress $\bar{\sigma}_N$ $(= R\cos\phi_m/(a/\cos\alpha))$ at the base of each slice in the impervious material. This is the "consolidation" pressure on the failure surface, prior to failure. For those slices which have $\bar{\sigma}_N$ less than $\bar{\sigma}_N'$ (diagram above), the drained envelope is assumed to give the shear strength available after drawdown, and for those slices which have $\bar{\sigma}_N$ greater than $\bar{\sigma}_N'$, the undrained envelope is assumed to give the shear strength available after drawdown.

4. After drawdown force polygon:

- Assume a factor of safety and prepare the force polygon for the after drawdown reservoir level. The F vectors after drawdown are equal to the sum of the total weight above the drawdown water level and the submerged weight below the drawdown water level, and $\psi = 0$.

- For slices in <u>pervious zones</u>, the drained envelope gives the shear strength available after drawdown and ϕ_m and C_m are determined for $\bar{\phi}$ and $\bar{c}$ as explained for the steady seepage case. The R vector acts at ϕ_m to the normal to the base of the slice and the C_m vector acts parallel to the base of the slice. The shear strength for these slices is a function of the <u>after drawdown effective normal stress</u>, because pore pressures in these materials are assumed to decrease as the reservoir level drops during drawdown.

- For slices in the <u>impervious zones</u>, the effective <u>normal stress after drawdown</u> is assumed to be the <u>same as before drawdown</u>. That is, the low permeability of these materials is assumed to prevent the pore pressures from adjusting to the lowered reservoir level resulting from the drawdown.

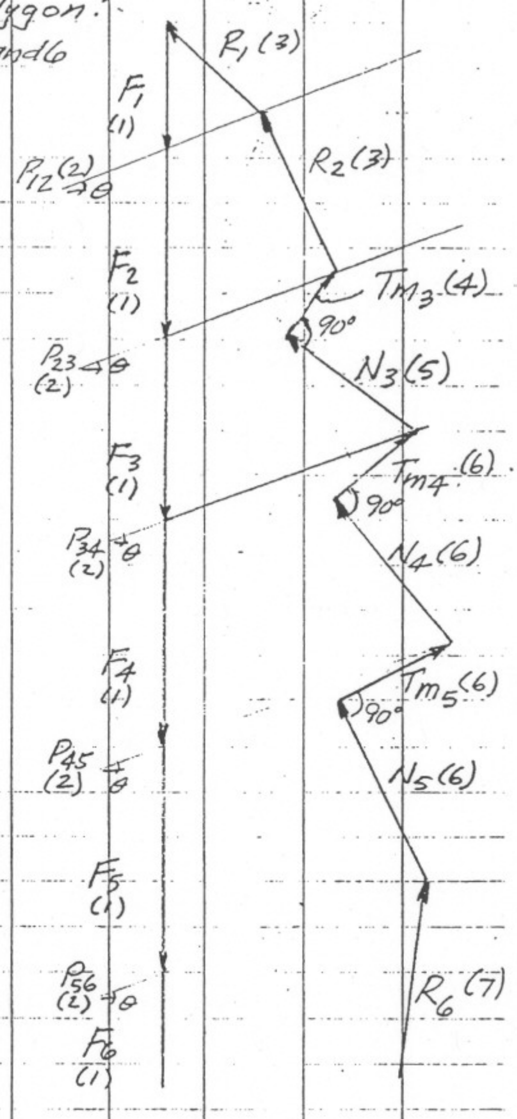
  Thus, $\bar{\sigma}_N$ determined from the before drawdown force polygon is used to determine the shear strength from combined drained and undrained envelopes.

- For slices in the impervious materials, calculate $T_{max} = S_{max} \cdot \frac{a}{\cos\alpha}$ where S_{max} is the shear strength from the combined drained-undrained envelopes at the <u>before drawdown</u> $\bar{\sigma}_N$.

($S_{max} = \bar{\sigma}_N \tan\bar{\phi} + \bar{c}$ for $\bar{\sigma}_N < \sigma'$ and $S_{max} = \bar{\sigma}_N \tan\phi_U + c_U$ for $\bar{\sigma}_N > \bar{\sigma}_N'$)

- For the assumed FS, compute $T_m = \frac{T_{max}}{FS}$ for the slices in the impervious materials where T_m is the mobilized shear force at the base of the slice.

- Procedure for drawing force polygon:
(Note – in this example, slices 1 and 2 and 6 are assumed to be in pervious materials with $\bar{\Phi}$ and $\bar{c} = 0$)



(1) Draw F vectors equal to sum of total weight above drawdown pool and submerged weight below drawdown pool. ($\psi = 0$)

(2) Draw interslice force directions as explained for steady seepage case

(3) Draw R_1 vector beginning at the tail of F_1, at Φ_m to the normal to the base of the slice, until it intersects with the interslice force vector (P_{12}). Begin the R_2 vector at the intersection of R_1 and P_{12} and draw it at Φ_m to the normal to the base of slice 2, extending until it intersects the interslice force vector P_{23}.

(4) Draw the Tm_3 vector for slice 3 parallel to the base of slice 3 with the head at the tail of the R_2 vector

(5) Draw the normal force vector N_3 from the tail of Tm_3 until it intersects the interslice force vector P_{34}. N_3 is at 90° to Tm_3.

(6) Draw the Tm_4 and N_4, and Tm_5 and N_5 vectors in a similar way to Tm_3 and N_3

(7) Draw the R_6 vector beginning at the intersection of N_5 and the interslice force vector P_{56}. R_6 is at an angle ϕ_m to the normal to the base of slice 6 and extends until it intersects the interslice force vector P_{67} (not shown on sketch previous page).

Continue until the last R vector is drawn. If the force polygon is closed ($\Delta_d = 0$), then the assumed factor of safety is the actual factor of safety. If it does not close, assume a new factor

APPENDIX 4-D

Actual Calculations

High, Central Core Rockfill Dam
550 ft high

Calcs. finished but drawings need to be reduced in size but need to be large enough to be readable.

SGNT

Calcs. are for steady state Seepage, U/S & D/S slopes.

Rapid Drawdown - U/S slope

APPENDIX 4E

Instructive Calculations for "yield" Acceleration