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Slope Stability during Rapid Drawdown

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INTRODUCTION

The writers had the great pleasure of working with Professor Seed on the research described in this paper. He played an active role in these studies,which were begun in the early 1980s (Wong, Duncan and Seed, 1983), and continued later under a contract for the U. S. Army Corps of Engineers (Wright and Duncan, 1987). We are pleased to acknowledge Professor Seed's great influence on and contribution to this work, and to dedicate this paper to his memory.

RAPID DRAWDOWN SLOPE STABILITY PROBLEMS

When the water level outside a slope drops, the stabilizing influence of the water pressure on the slope is lost. If the water level subsides so rapidly that the pore pressures within the slope do not have time to change in equilibrium with the drop in external water level, the slope is made less stable.

Many failures of dam slopes and natural slopes have occurred during rapid drawdown. These include, for example, Pilarcitos Dam south of San Francisco (W. A. Wahler & Associates, 1970), Walter Bouldin Dam in Alabama (Whiteside, 1976), and a number of river bank slopes along the Rio Montaro in Peru (Lee and Duncan, 1975). Rapid drawdown imposes very severe loading on slopes, and is frequently the design condition that controls the steepness of the upstream slopes of dams. Analysis of rapid drawdown is therefore an important consideration for design of dams, and for design of other slopes that will be submerged and subject to drawdown.

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Materials of high permeability can drain during rapid drawdown, but materials with clay content cannot. To analyze stability during rapid drawdown, it is necessary to determine which materials will drain and which will not. In doubtful cases it must be assumed that drainage either will or will not occur, whichever will result in the lowest shear strength within the particular zone, and the lowest factor of safety for the slope.

For some dams rapid drawdown from a high reservoir level is an improbable event, and represents an extreme loading case. For such dams it is appropriate to permit low factors of safety for the rapid drawdown condition. For other dams, notably pumped storage projects, episodes of rapid drawdown occur frequently, and they represent a regular operating condition. In these cases it is appropriate to require higher values of safety factor for the drawdown condition, as for other design conditions that have high probabilities of occurrence.

METHODS OF ANALYSIS

Stability during rapid drawdown has been analyzed in two basically different ways: (1) using effective stress methods, and (2) using total stress methods, in which the undrained shear strength of the soil is related to the consolidation pressures in the slope prior to drawdown. Both methods treat free draining materials in the same way. Their strengths are analyzed in terms of effective stresses, and it is assumed that the pore pressures within them are hydrostatic.

Effective Stress Methods

The advantage of effective stress analyses is that it is relatively easy to evaluate the required shear strength parameters. The effective stress shear strength parameters for soils are readily determined by means of isotropically consolidated undrained (IC-U) triaxial tests with pore pressure measurements. This type of test is well within the capability of modern soil mechanics laboratories.

The disadvantage of effective stress analyses is that it is difficult to estimate with accuracy the pore pressures that will exist within non-free draining materials during drawdown. The pore pressure changes during drawdown depend on the changes in stress that result from the changing water loads on the slope, and the undrained response of the soils within the slope to these changes in load. While the changes in stress can be estimated with reasonable accuracy, particularly at shallow depths beneath the surface of the slope, the undrained response of the soil is much harder to estimate. The changes in pore pressure would be expected to be considerably different for materials that tend to dilate during shear and those that do not. While in principle it is possible to estimate these pore pressures using Skempton's pore pressure parameters (Skempton, 1954), in practice this is difficult.

In most cases effective stress analyses of stability during rapid drawdown have used the assumptions regarding pore pressures that were suggested by Bishop (1954) and later used by Morgenstern (1963). These assumptions have been justified on the basis of the fact that they are conservative in most cases. They have been found to result in reasonable values of factor of safety for dams that suffered rapid drawdown failures: Wong, et al. (1983) found that the values of safety factor calculated using Morgenstern's assumption were $F = 1.2$ for Pilarcitos Dam and $F = 1.0$ for Walter Bouldin Dam, both of which failed.

It seems likely that Bishop and Morgenstern's assumptions for pore pressures during drawdown may be more accurate for soils that do not tend to dilate during shear than for those that do tend to dilate. Thus, although these assumptions may show reasonable correspondence with failures of slopes in materials that are not densely compacted and do not tend to dilate during shear, they are likely to be unduly conservative for better-compacted materials that do tend to dilate during shear. Use of effective stress analyses based on the Bishop and Morgenstern assumptions would treat all fill materials alike with respect to pore pressures during drawdown, regardless of how well they are compacted or how strongly they might tend to dilate during shear.

Terzaghi and Peck (1967) suggested that pore pressures during drawdown in well-compacted silty sands could be estimated using flow nets. Several investigators (Browzin, 1961; Brahma and Harr, 1963; Newlin and Rossier, 1967; Desai and Sherman, 1971; Desai, 1972; Desai, 1977) used theoretical methods to analyze the non-steady flow conditions following drawdown. Like the Bishop and Morgenstern pore pressure assumptions, these methods do not consider the behavior of the soil with regard to dilatancy, and are thus not capable of representing all of the important factors that control the pore pressures during drawdown.

Svano and Nordal (1987) and Wright and Duncan (1987) used procedures for estimating pore pressures during drawdown that reflect the influence of dilatancy on the pore pressure changes. Svano and Nordal used two-stage stability analyses and iterated to achieve consistency between the calculated factor of safety and the values of the pore pressures. Wright and Duncan used finite element analyses to estimate the stress changes during drawdown, and Skempton's pore pressure parameters to calculate the pore pressures. These studies indicate that it is possible to estimate realistic pore pressures for effective stress analyses, but it is more cumbersome and difficult than using total stress analyses as described in the following sections.

Terzaghi and Peck (1967) summarized the problems in estimating pore pressures during drawdown in these words:

> "... in order to determine the pore pressure conditions for the drawdown state, all the following factors need to be known: the location of the boundaries between materials with significantly different properties; the permeability and consolidation characteristics of each of these materials; and the anticipated maximum rate of drawdown. In addition, the pore pressures induced by the changes in the shearing stresses themselves .... need to be taken into consideration. In engineering practice, few of these factors can be reliably determined. The gaps in the available information must be filled by the most unfavorable assumptions compatible with the known facts."

By using undrained, total stress analyses as described in the following sections, many of the problems associated with estimating pore pressures for effective stress analyses can be avoided, and the accuracy of rapid drawdown stability analyses can be very considerably improved.

Corps of Engineers Method

The Corps of Engineers method (Corps of Engineers, 1970) is a two-stage analysis procedure. The first stage, which analyzes conditions before drawdown, is used to determine values of consolidation pressure along the slip surface. These consolidation pressures are used to determine undrained shear strength values for the second stage analysis, with the reservoir drawn down.

The undrained shear strength values for the second stage are determined from the total stress IC-U strength envelope, as shown in Fig. 1. At low stresses, where the total stress envelope lies above the effective stress drained strength envelope, the drained strength envelope is used to determine strengths for the second stage analysis.

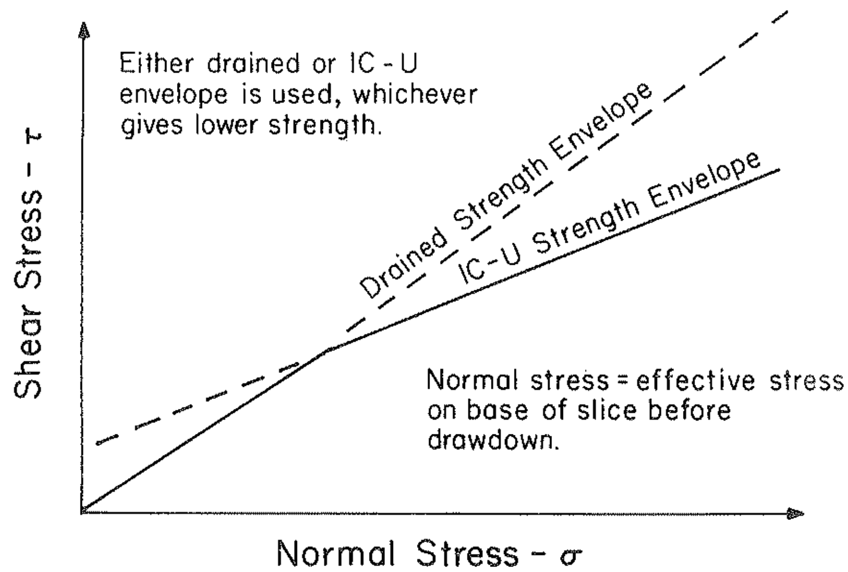

Fig. 1 - Strength Envelope Used in Corps of Engineers Method

The justification for using the drained strength envelope at low stresses is to avoid relying on shear strength that results from negative pore pressures, which could not be mobilized should these pore pressures disappear as a result of cavitation or drainage. The use of the drained envelope is overly conservative, however,

because the effective normal stress used to determine the shear strength is the effective stress before drawdown, which is lower than the effective stress after drawdown, even when it is assumed that the pore pressures after drawdown are hydrostatic. As a result the Corps of Engineers method is unnecessarily conservative.

The Corps of Engineers method avoids the problems of estimating changes in pore pressure during drawdown by using undrained strengths to analyze stability after drawdown. By using undrained strengths, this procedure accounts for the tendency of the material to dilate or compress during shear.

While the use of the total stress IC-U envelope is qualitatively correct, it cannot be justified in detail. The circles of stress used to develop the total stress envelope combine the effective stress during consolidation with the deviator stress at failure, and do not represent a state of stress in the soil at any given time. Furthermore, the way that the stresses on the slip surface are related to the IC-U envelope is not consistent with the way the envelope is plotted. Although the errors resulting from these inconsistencies are not extremely large in most cases, the use of an inconsistent and illogical method of relating shear strength to consolidation pressure is undesirable. As discussed in subsequent sections, better procedures for relating shear strength to consolidation pressure have been developed by Lowe and Karafiath (1960a).

Lowe and Karafiath's Method

Like the Corps of Engineers method, Lowe and Karafiath's method uses a two-stage analysis procedure, and uses undrained strengths for analysis of stability during drawdown.

As shown in Fig, 2, Lowe and Karafiath's method uses envelopes that are more logical than the total stress IC-U envelope for relating undrained shear strengths to consolidation pressures. The horizontal axis is the effective stress on the failure plane during consolidation (σ'_{fc}), and the vertical axis is the shear stress on the failure plane at failure (τ_{ff}).

Lowe and Karafiath showed that the undrained strength of soils, when plotted as τ_{ff} vs σ'_{fc}, varies with the effective principal stress ratio during consolidation, $K_c = \sigma'_{1c}/\sigma'_{3c}$. The value of this ratio can vary from unity (for consolidation under isotropic stresses) to K_f, the value of σ'_1/σ'_3 at failure. As the value of K_c increases, the undrained strength increases. For any material there is a family of undrained strength envelopes corresponding to different values of K_c varying from 1.00 to K_f.

The manner of representing undrained strength suggested by Lowe and Karafiath, which is shown in Fig. 2, is logical and accurate. The testing required to establish these strength envelopes, however, is very difficult. Test specimens must be consolidated under anisotropic stresses, with various values of K_c. This process requires considerable time and careful attention because the stresses must be increased slowly to avoid the possibility that the specimens will fail during consolidation. The larger the value of K_c, the more difficult the test procedure, and the greater the likelihood that the specimen will fail during consolidation. Few laboratories are experienced in performing these difficult tests, and their use adds considerably to the cost of employing the Lowe and Karafiath method for analyzing rapid drawdown stability.

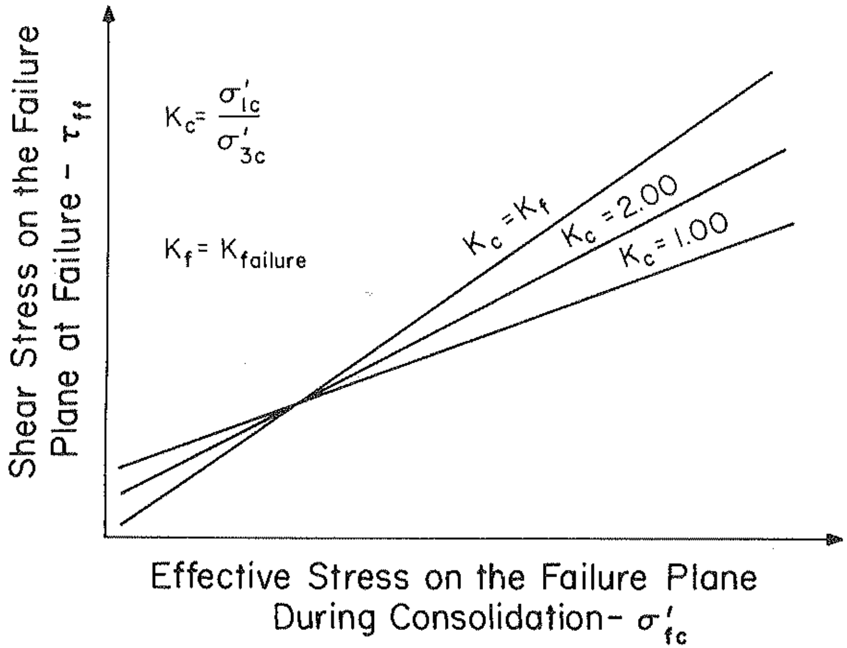

Fig. 2 - Strength Envelope Used in Lowe and Karafiath's Method

Unlike the Corps of Engineers method, the Lowe and Karafiath method uses the undrained strength envelopes at low stresses, where the undrained strength envelopes lie above the drained strength envelope. The method thus does not allow for the fact that it may not be possible to mobilize the full undrained strength in circumstances where it is limited by cavitation or drainage.

EVALUATING STRENGTH FOR RAPID DRAWDOWN ANALYSES

As noted previously, the method of representing undrained strengths proposed by Lowe and Karafiath is logical and accurate, but establishing the variation of undrained strength with consolidation pressures involves difficult and expensive laboratory tests.

Because of the difficulties involved in performing AC-U triaxial tests, a number of investigators have suggested procedures for estimating the results of AC-U tests from the results of IC-U tests. The benefits of being able to do this are considerable: IC-U tests are the easiest to perform of any consolidated-undrained triaxial tests. The specimens cannot fail during consolidation because they are not subjected to shear stresses. The consolidation stresses can be applied in a single increment, and little operator attention is required.

Accordingly, methods of estimating AC-U strengths from the results of IC-U tests have been suggested by Taylor (1948), Lowe and Karafiath (1960b), and Noorany and Seed (1965). These methods were studied by Wong et al. (1983), with the following conclusions:

- the method suggested by Taylor (1948) results in reasonable estimates of the AC-U strengths, but is cumbersome because it requires graphical construction and involves processing pore pressure data for many stages during the test.

- the method suggested by Lowe and Karafiath (1960b) results in unreasonable estimates of AC-U strength envelopes for some values of K_c.

- the method suggested by Seed and Noorany (1965) results in reasonable AC-U strength envelopes.

The studies performed by Wong, et al. (1983) showed that it is also possible to estimate AC-U strength envelopes from IC-U strength test results by simple linear interpolation. The envelope corresponding to $K_c = 1.00$ in Fig. 2 is the IC-U envelope, and is established by plotting values of τ_{ff} vs σ'_{fc}. The envelope corresponding to $K_c = K_f$ is simply the effective stress strength envelope, established by plotting values of τ_{ff} vs σ'_{ff}. All of the required stresses (τ_{ff}, σ'_{fc}, and σ'_{ff}) can be computed from the results of an IC-U triaxial test with pore pressure measurements. Envelopes for values of K_c intermediate between 1.00 and K_f can be established by interpolating linearly between the envelopes for K_c = 1.00 and $K_c = K_f$. This procedure makes the use of AC-U strengths much easier and more practical, because it eliminates the necessity of performing difficult AC-U tests. Accordingly, it is recommended that AC-U envelopes of the type shown in Fig. 2 be established by linear interpolation, using the results of IC-U tests.

NEW METHOD FOR EVALUATING STABILITY DURING RAPID DRAWDOWN

At the current state of the art of analyzing stability during rapid drawdown, it is possible to combine the best features of the methods that have been developed, and to devise a new method that is accurate and easy to use:

- Like the Corps of Engineers method and Lowe and Karafiath's method, the new method uses undrained shear strengths to avoid the complications and inaccuracies involved in estimating undrained pore pressure during drawdown.

- Like Lowe and Karafiath's method, it uses the more accurate τ_{ff} vs σ'_{fc} envelope, with AC-U strengths varying with the value of K_c.

- Following the findings of Wong et al. (1983), the AC-U strengths are estimated by linear interpolation using the results of IC-U tests.

- To avoid using undrained strengths higher than drained strengths, which cannot be mobilized if cavitation or drainage occurs, drained strengths are used wherever they are smaller than the undrained strengths. This procedure is different from the one involved in the Corps of Engineers' method. In the Corps of Engineers' Method the effective stress envelope is used with the effective stress before drawdown to determine the shear strength. In the new procedure the effective stress after drawdown is used, which is more logical. Use of the effective stress after drawdown requires a 3-stage analysis. The first two stages are the same as the Lowe and Karafiath method. In the third stage the drained strength is used for sections of the slip surface where the drained strength is smaller than the undrained strength.

The new procedure is believed to combine the best features of the methods that have been used previously for analysis of rapid drawdown stability. In addition, it incorporates two new features: (1) It simplifies the task of estimating AC-U strengths by using linear interpolation, and (2) it uses a more accurate and less conservative method of accounting for possible reduction of undrained strengths at low pressures due to cavitation or drainage.

The specific steps involved in the method are these:

(1) Determine, for each soil in the cross-section, whether drainage will occur during drawdown. The most logical means of making this determination is to estimate the value of the dimensionless time factor T, given by the following expression:

$$T = c_v \, t / D^2 \qquad (1)$$

in which c_v = coefficient of consolidation, t = time for drawdown, and D = length of drainage path. If the calculated value of T is greater than or equal to 3.0, the dissipation of excess pore pressures during drawdown will be 90 percent or more, and it is reasonable to treat the material as fully drained.

Values of c_v can be calculated using data for the time rate of consolidation during triaxial tests. Approximate values for various types of compacted soils are listed in Table 1.

Table 1. Approximate Values of c_v for Various Soils

Type of Soil	Values of c_v
Coarse sand	>10,000 ft^2/day
Fine sand	100 to 10,000 ft^2/day
Silty sand	10 to 1000 ft^2/day
Silt	0.5 to 100 ft^2/day
Compacted clay	0.05 to 5 ft^2/day
Soft clay	< 0.2 ft^2/day

If the value of T calculated using equation (1) is less than 3.0, the undrained strength should be considered. The following steps assure that the undrained strength used in the stability analyses will not be larger than the drained strength. It is thus conservative to assume that the material is undrained in those cases where there is doubt whether or not complete drainage will occur.

(2) Establish the strength envelopes required for the analyses. For materials that drain during drawdown, only the drained strength envelope is required. For materials that are undrained during drawdown, both the drained envelope (τ_{ff} vs σ'_{ff}) and the undrained envelope (τ_{ff} vs σ'_{fc}) are required. The required envelopes for values of K_c between 1.0 and K_f can be determined by linear interpolation, as discussed previously.

(3) For a selected slip surface, calculate the factor of safety after drawdown by a 3-step procedure:

(i) Perform an analysis of stability before drawdown using drained strength parameters for all materials, to determine the effective normal stress (σ'_{fc}) on the base of each slice, and the value of K_c for each slice. Values of K_c are determined in the manner suggested by Lowe and Karafiath, based on the assumption of no reorientation of principal stresses during drawdown. These values are used to determine undrained strength values for the materials that do not drain freely during drawdown.

(ii) Perform an analysis of stability after drawdown using undrained strengths for materials that do not drain freely, and drained strengths for materials that do drain during drawdown. To this stage the method is the same as Lowe and Karafiath's method, except for the use of linear interpolation to estimate AC-U strength values.

(iii) Determine if the drained strength for any slice is lower than the undrained strength used in step (ii). The drained strength is determined using the total stresses on the bases of the slices from step (ii) and pore pressures for the drained condition. If for any slice the drained strength is lower than the undrained strength, perform another analysis using drained rather than undrained strengths for slices where the drained strengths are lower.

(4) Repeat the 3-step procedure for other slip surfaces to locate the one with the lowest factor of safety following drawdown.

The computations involved in the method are quite lengthy, and a computer program is a practical necessity to do all of the computations necessary to search for critical slip surfaces. The analyses discussed in the following sections were performed with the computer program UTEXAS3, which incorporates the new method for rapid drawdown stability analyses.

EXAMPLES

The methods of analysis described previously have been used to analyze three dams, as discussed in this section. The first two of these dams, Pilarcitos and Walter Bouldin, suffered slides due to rapid drawdown. The third is a hypothetical dam for a pumped storage project. The dam is considered to be subject to large drawdowns as a normal operating condition for the pumped storage project. Accordingly, reliable estimates of the stability during drawdown are especially important for this embankment, and the required factor of safety is higher than for embankments for which rapid drawdown is a relatively unlikely occurrence.

Pilarcitos Dam

Pilarcitos Dam , shown in Fig. 3, is a homogeneous rolled earthfill embankment. The crest of the dam is about 78 feet above the upstream toe. The upstream slope is inclined at 2.5 on 1 for the lower 58 ft. (to El. 678), and at 3 on 1 from this point to the crest (El. 698). When the water level was lowered from El. 692 to El. 657 between October 7 and November 19, 1969, a slide occurred. During the two

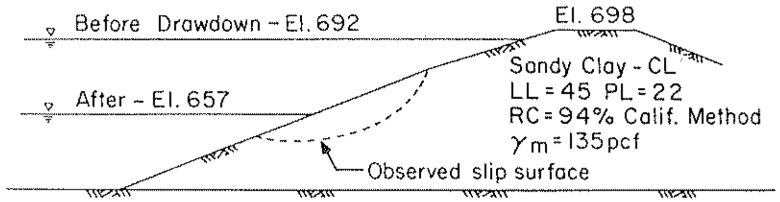

Fig. 3 - Pillarcitos Dam Cross-Section

weeks prior to the failure, the drawdown rate was nearly constant at about 1.7 feet per day. The approximate location of the failure surface found in an exploratory trench is shown in Fig. 3.

W. A. Wahler & Associates of Palo Alto, California was contracted by the San Francisco Water Department to perform field investigations, laboratory tests and analyses, and to recommend remedial measures. The information regarding Pillarcitos Dam presented in this paper was obtained from the Wahler and Associates report (W. A. Wahler and Associates, 1970).

The material comprising the upstream slope is classified as a sandy clay (CL) with 60 to 70 percent passing the number 200 sieve, and a permeability of about 4×10^{-8} cm/sec. The liquid limit is 45 percent and the plastic limit is 23 percent. The average field compaction was approximately 94 percent of the laboratory maximum density obtained using 20,000 ft-lb/ft^3 compactive effort (the California Method).

The bilinear failure envelope used in the Corps of Engineers procedure is shown in the upper part of Fig. 4, and the envelopes used in Lowe and Karafiath's and the three-stage procedure are shown in the lower part of Fig. 4.

As shown in Fig. 5, the factor of safety calculated by the Corps of Engineers' procedure was 0.82. The factors of safety calculated by Lowe and Karafiath's procedure and the new procedure are both 1.05. The critical circles are very nearly the same for all three methods, as can be seen in Fig. 5.

Because it uses drained strengths where they are lower than undrained strengths, the new method always results in factors of safety that are the same as or lower than the factors of safety calculated by the Lowe and Karafiath procedure. In the case of Pillarcitos Dam, there were no slices where the drained strengths were lower than the undrained strength, and the factors of safety calculated using Lowe and Karafiath's method and the new method were therefore the same.

Walter Bouldin Dam

Walter Bouldin Dam is a rolled earthfill embankment. In the section that failed, the dam is approximately 60 feet high, resting on 80 feet of clayey sand and gravel; the underlying bedrock is schist. As shown in Fig. 6, the lower portion of the embankment is a clayey sandy gravel that was not involved in the slide. Overlying the gravel are a layer of cretaceous clay, a zone of micaceous silt, and a clayey silty sand layer that blankets the slope. The upper portion of the upstream slope is blanketed with riprap. The upstream slope is 2 on 1 above Elev. 245 and 2.5 on 1 below Elev. 245.

The slide occurred on February 10, 1975 during an extremely rapid drawdown of 32 feet in 5.5 hours that occurred as a result of a piping failure in another part of the dam. The slide extended for a length of about 32 feet along the slope. The slip surface extended about 10 feet below the surface of the slope near the edges of the slide where it was excavated, and possibly reached a greater depth near the center of the slide mass where it was not excavated. A detailed study of the failure was carried out by Stephen L. Whiteside at Stanford University. The information

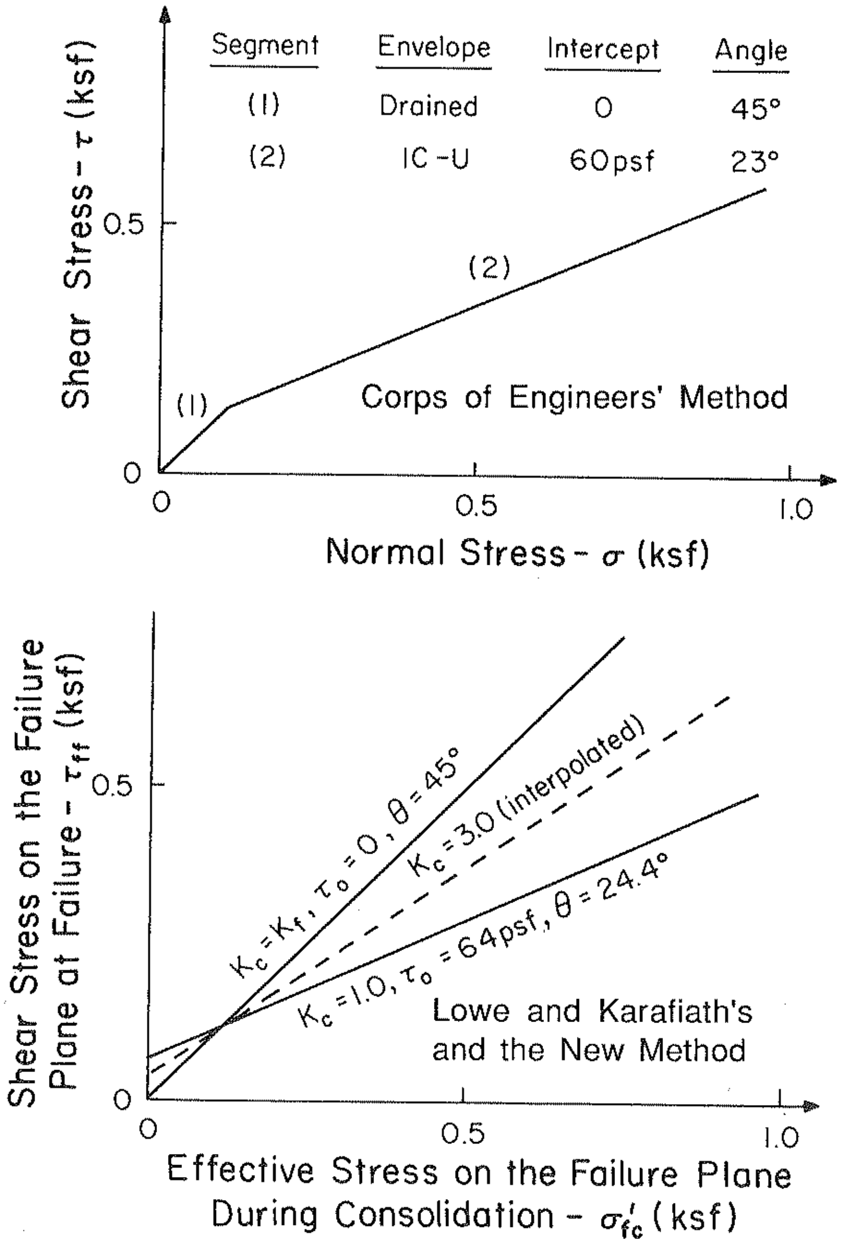

Fig. 4 - Strength Envelope for Pillarcitos Dam

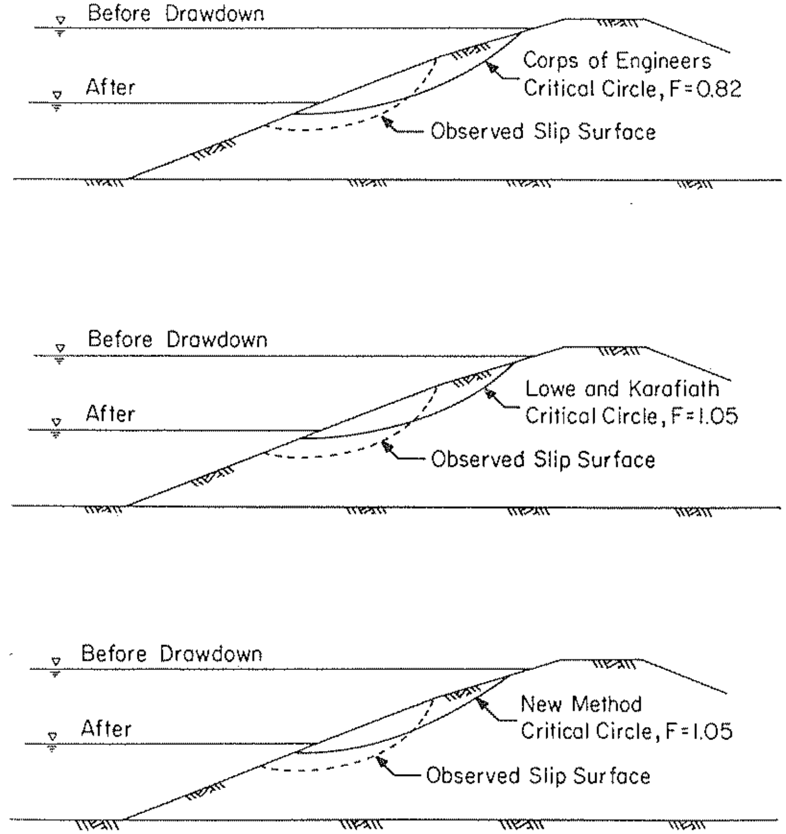

Fig. 5 - Results of Stability Calculations for Pillarcitos Dam

Zone	Material	Unified Class.	LL	PL	RC-Mod. Proctor	γ_m(pcf)
1	Riprap					
2	Clayey Silty Sand	SM-SC	39	31	86	128
3	Micaceous Silt	ML	43	33	84	123
4	Cretaceous Clay	CH	52	28	88	124
5	Clayey Sandy Gravel					

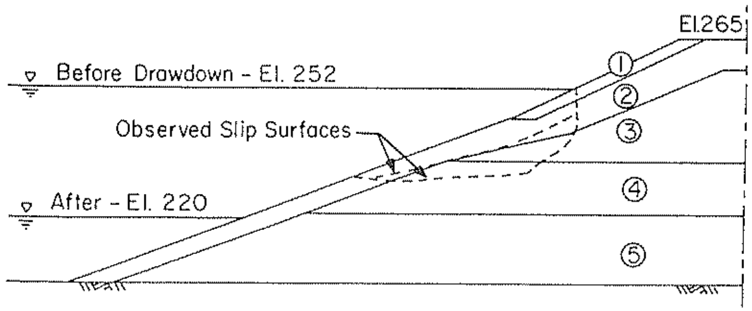

Fig. 6 - Walter Bouldin Dam Cross-Section

regarding Walter Bouldin Dam presented in this paper was obtained from the Whiteside report (Whiteside, 1976).

Strength parameters for the materials involved in the slide are listed in Table 2. The parameters for the drained strength envelope used in the Corps of Engineers' Method are (by definition) the same as the parameters for the $K_c = K_f$ envelope used in Lowe and Karafiath's and the new method.

Table 2 - Strength Parameters for Walter Bouldin Dam

Material	Corps of Engineers Method		Lowe and Karafiath and New Methods	
	Drained	IC-U	$K_c = K_f$	$K_c = 1.0$
Clayey Silty Sand	c' = 240 psf ϕ' = 32.7 deg	c = 650 psf ϕ = 13 deg	τ_0 = 240 psf θ = 32.7 deg	τ_0 = 750 psf θ = 15 deg
Micaceous Silt	c' = 220 psf ϕ' = 22.5 deg	c = 450 psf ϕ = 11 deg	τ_0 = 220 psf θ = 22.5 deg	τ_0 = 480 psf θ = 13 deg
Cretaceous Clay	c' = 180 psf ϕ' = 19 deg	c = 230 psf ϕ = 13 deg	τ_0 = 180 psf θ = 19 deg	τ_0 = 280 psf θ = 15.5 deg

The riprap was assumed to be drained, with γ_{sat} = 140 pcf, γ_{dry} = 125 pcf, and ϕ' = 40 deg.

As shown in Fig. 7, the factor of safety calculated by the Corps of Engineers' procedure was 0.93. The factor of safety calculated by Lowe and Karafiath's procedure was 1.09, and the value calculated using the new procedure was 1.04. The critical circles for all three methods were nearly the same, as may be seen in Fig. 7.

Pumped Storage Project Dam

The dam for this example is shown in Fig. 8. The embankment has a wide, densely compacted, silty clay core. The lower portion of the upstream slope is a random zone with the same strength properties as the core. The upper portion of the upstream slope and all of the downstream slope is a free-draining rockfill.

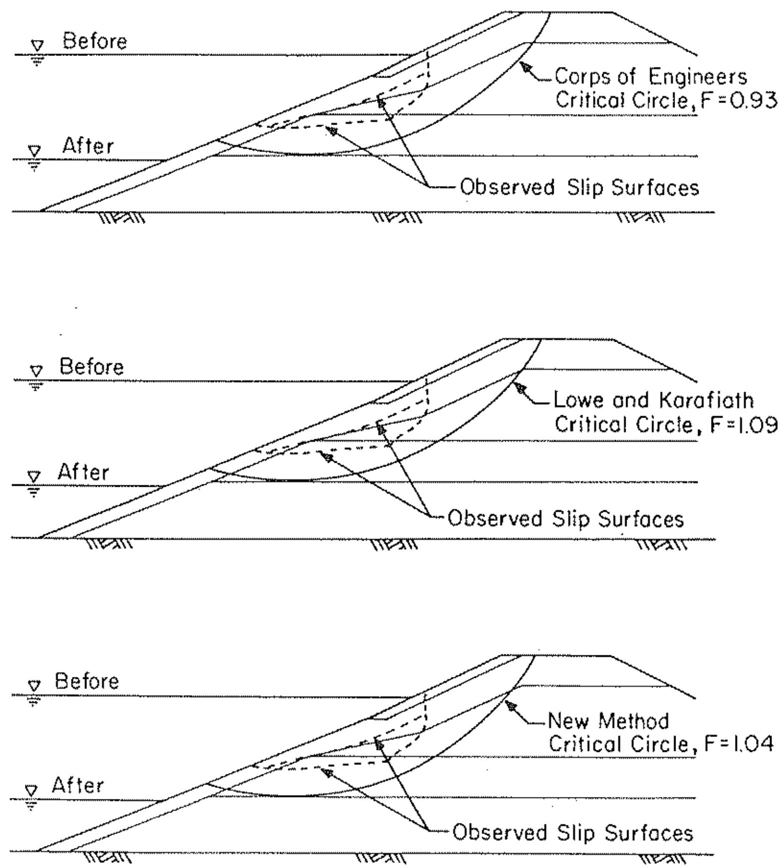

Fig. 7 - Results of Stability Calculations for Walter Bouldin Dam

Zone	Material	γ_m(pcf)
1	Compacted Rockfill	142
2	Silty Clay Core	140
3	Silty Clay Random Zone	140

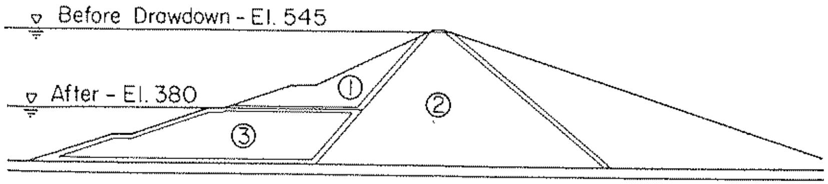

Fig. 8 - Hypothetical Pumped Storage Project Dam Cross-Section

Strength parameters for the materials in the dam are listed in Table 3. The significant (2000 psf) cohesion intercept for the IC-U envelope is indicative of a highly dilatant material. The drained strength of this material is lower than the undrained strength in the range of stresses below the point where the two envelopes cross, at an effective consolidation stress of approximately 5000 psf. This corresponds approximately to a depth of 40 feet for a drained slope and 80 feet for a submerged slope.

The rockfill, filter and drainage zones are free draining materials, and all have the same properties. The dry and saturated unit weights are 128 and 142 pcf, respectively. Dry unit weights were used for portions of these materials above reservoir level, and saturated unit weights were used for those portions below reservoir level.

Table 3 - Strength Parameters for Hypothetical Pumped Storage Project Dam

Material	Corps of Engineers' Method		Lowe & Karafiath and New Methods	
	Drained	IC-U (R)	$K_c = K_f$	$K_c = 1.0$
Compacted Rockfill and Filters	$c' = 0$ $\phi' = 37$ deg	N.A.	$\tau_0 = 0$ $\theta = 37$ deg	N.A.
Silty Clay Core	$c' = 0$ $\phi' = 36$ deg	$c = 2000$ psf $\phi = 18$ deg	$\tau_0 = 0$ $\theta = 36$ deg	$\tau_0 = 2250$ psf $\theta = 20$ deg
Silty Clay Random Zone	$c' = 0$ $\phi' = 36$ deg	$c = 2000$ psf $\phi = 18$ deg	$\tau_0 = 0$ $\theta = 36$ deg	$\tau_0 = 2250$ psf $\theta = 20$ deg

The stability of the embankment was analyzed for a condition of drawdown from elevation 545 (5 feet below the crest of the dam) to elevation 380, which is the elevation of the bench at mid-height of the embankment. The drawdown results in complete dewatering of the upstream rockfill shell and a portion of the filter layer beneath the shell.

As shown in Fig. 9, the factor of safety calculated by the Corps of Engineers' procedure was 1.37. The factor of safety calculated by Lowe and Karafiath's procedure was 1.58, and the value calculated using the new procedure was 1.56. The critical circles for all three methods were nearly the same, as may be seen in Fig. 9.

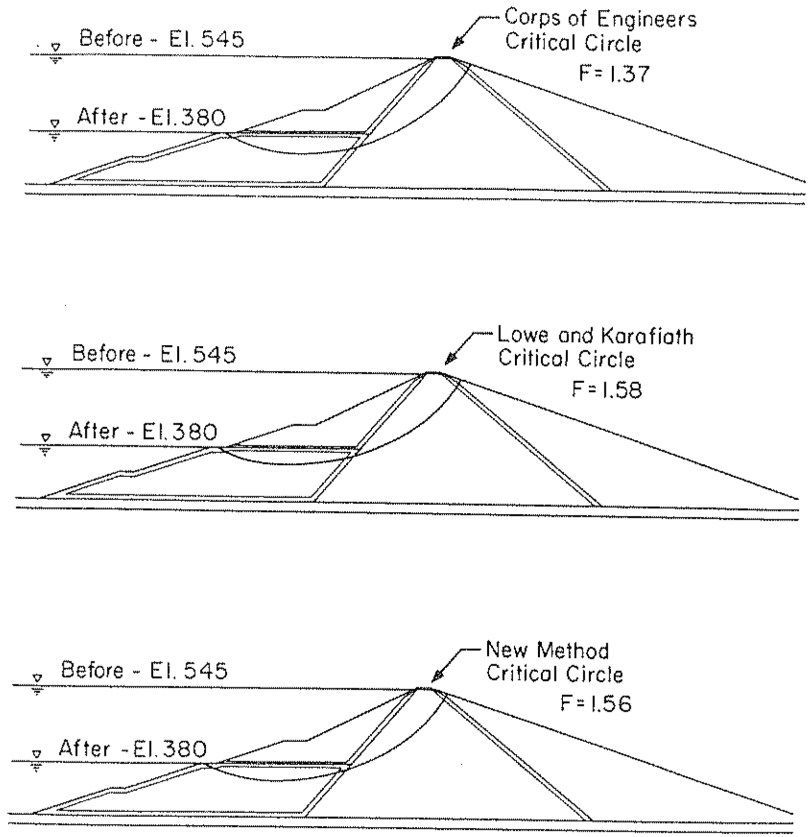

Fig. 9 - Results of Stability Computations for the Hypothetical Pumped Storage Project Dam

Conclusions

The new method for analyzing slope stability during rapid drawdown described in the previous pages combines what are believed to be the best features of the methods that have been used previously.

It avoids the problems associated with estimating pore pressures in undrained materials for the after-drawdown condition by using undrained strengths, and correctly reflects the strengths of materials that tend to dilate during shear. It uses the most accurate representation of undrained strengths (values of τ_{ff} that vary with the values of σ'_{fc} and K_c), but it avoids the necessity for difficult AC-U strength testing by using linear interpolation of strength values from IC-U tests.

By using drained strength values where these are smaller than undrained strengths, the method avoids reliance on strengths due to negative pore pressures, which cannot be mobilized if cavitation or drainage occurs. The procedure for eliminating strengths due to negative pore pressures is more accurate, and less conservative, than the procedure used in the Corps of Engineers method.

The new method is somewhat more conservative than Lowe and Karafiath's method, and provides a reliable estimate of stability following drawdown that is believed to be as accurate as possible at the current state of the art.

For the three examples described, the factors of safety calculated using the new method are 15 percent to 30 percent higher than values calculated using the Corps of Engineers method, and zero to 5 percent lower than values calculated using Lowe and Karafiath's method.

Although the values of safety factor calculated using the new method are only slightly smaller than values calculated using Lowe and Karafiath's method for the examples considered, it is important that the new method is conservative because it does not rely on strengths resulting from negative pore pressures, whereas Lowe and Karafiath's method does rely on these strengths.

Thus, while the new method offers the same reliability with respect to strengths due to negative pore pressures as the Corps of Engineers method, it uses a more accurate evaluation of strength, and results in values of safety factor only slightly lower than Lowe and Karafiath's method. The method thus is reliably conservative with regard to undrained strengths, but does not involve excessively conservative approximations.

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Response of Soft Soil Sites during Earthquakes

*I.M. Idriss**

INTRODUCTION

As noted in the announcement, my initial plan for this Symposium was to present and discuss three projects where evaluating the liquefaction potential was of critical importance to each project. The first evaluation was done in 1967-68, the second was done in 1973-74 and the third evaluation was completed in 1986-87. Professor H. B. Seed participated as a consultant in all three projects, and as usual made significant contributions both to the evaluation process and to the successful implementation of the project. My intent was to illustrate the evolution of the procedures for evaluating liquefaction potential and to discuss the mitigative measures recommended and undertaken in each instance.

Each project was completed at a time when refinements and/or changes in the evaluation procedure were taking place. The first was done when the procedure was in its formative years with heavy reliance on ground response analyses and on laboratory cyclic test data. The second project was done as the importance of SPT blow count was being highlighted by Professor Seed. The third project was initiated as refinements regarding the effects of grain size on liquefaction potential were being finalized.

Two very significant events intervened and caused me to modify my original plan. First, I was unable to get permission to publish the material from one project. The second, and the more important, was the occurrence of the Loma Prieta earthquake on 17 October 1989.

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