The Place of Stability Calculations in Evaluating the Safety of Existing Embankment Dams

Thorough investigations of site conditions and construction records should have precedence over stability analyses for determining the safety of embankment dams.

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THE PURPOSE of evaluating the safety of an existing embankment dam is to ensure that the catastrophic loss of the reservoir will not occur. Many reports in which the safety of existing dams is evaluated relate the safety to the results of stability analyses. Yet, seismic considerations aside, stability analyses are often irrelevant and may even be misleading.

To be sure, one of the great achievements of soil mechanics has been the development of limit-equilibrium methods of stability analyses. Every soils student learns about them. Sophisticated computer programs exist for carrying them out. They have an important place in embankment dam engineering, primarily in design, but they can be misleading in evaluating dam safety if too much dependence is placed on the numerical values of factors of safety derived from them.

Purposes of Stability Analysis in Design

Before the implications of stability analyses with respect to the safety of existing dams can be considered, the ways in which such analyses are useful in design should be reviewed. The discussion herein is limited to limit-equilibrium analyses. Other techniques are required for investigating seismic behavior and liquefaction.

In practice, the slopes for embankment dams are not chosen on the basis of stability calculations; they are chosen by precedent. The initial

selection is based on the designer's judgment that takes into account foundation conditions, economics, the availability of materials, logistics and a whole series of technical and nontechnical considerations. Having tentatively selected both exterior and interior slopes, the designer carries out stability analyses to ensure that conventional factors of safety are achieved. This application of stability analyses is a legitimate step in design.

Furthermore, inasmuch as every embankment dam differs in some respect from any other, it is useful to have a basis for comparing the proposed dam with others whose performance is known. Factors of safety provided by stability analyses for the proposed design and for existing dams constitute such a basis.

Moreover, stability analyses are valuable in comparing the efficiencies of various arrangements of the zoning of the dam. The analyses can provide insights regarding the relative merits and economies of placing superior or inferior materials of different costs in different parts of the embankment.

Stability analyses assist greatly in avoiding shear failures during construction. Many dams have experienced upstream or downstream sliding failures during construction because of weak seams in the foundation. Such failures can often be predicted and avoided by careful investigation and appraisal of the foundation conditions and appropriate equilibrium analyses.

In addition, failures during construction have been known to occur as a consequence of pore pressures induced in relatively impervious zones by the addition of fill. These failures can be predicted by suitable equilibrium analyses combined with investigations of pore-pressure coefficients in the relevant materials and studies of the rates of dissipation. These failures also can be avoided by implementing a monitoring program that ensures that sufficient dissipation of excessive pressures occurs by instituting appropriate waiting periods during filling.

Finally, stability analyses are used to provide assurance that a dam will not fail under operating conditions. A large enough factor of safety is specified to guard against downstream sliding under a full reservoir and, in addition, an appropriate factor of safety is specified against an upstream failure resulting from rapid drawdown.

All these uses of stability analyses are legitimate parts of design. They require knowledge of foundation conditions and of the pertinent properties of the various materials involved. The necessary information from the field and the analyses are usually developed in successive steps of increasing refinement.

Stability Analyses of Existing Dams

The application of stability analyses in the design phase makes use of idealized or generalized soil properties, assumed known geometries and idealized surfaces of sliding. In contrast, if the factor of safety of an existing embankment dam is to be determined correctly, facts rather than idealizations are needed. Obtaining these facts is no simple task.

First, both the external and internal geometry of the dam must be ascertained, which may be difficult if reliable as-built drawings of the dam are not available. Second, the properties of the materials need to be determined, either from good records as they were actually placed or by investigation. The geometry of the surface of sliding can be determined by measurements if possible, or it can be established realistically from knowledge of the subsurface conditions. The shear strengths have to be ascertained in terms of effective-stress parameters along the surface of sliding including that portion of the surface within the foundation. The pore pressures on the actual surface of sliding (or on the potential surface of sliding if the actual one is not known) must be determined as well.

All these data are at best expensive to evaluate realistically, and in many instances may be impractical to determine. Obtaining them may necessitate exploratory work within and beneath the dam, itself often an undertaking detrimental to the safety of the dam.

Indeed, it is fair to say that if good construction records are unavailable, it may be impractical or virtually impossible to get adequate data for calculating the factor of safety reliably. However, this limitation is only one of several considerations leading to the conclusion that stability analyses may be irrelevant or mislead-

ing with respect to predicting the safety of an existing dam.

Failure of Embankment Dams

Embankment dams can fail either catastrophically or non-catastrophically. A catastrophic failure of whatever nature is defined as one that results in the uncontrolled loss of the reservoir with consequent loss of life and damage to property. It is the avoidance of catastrophic failures that justifies the authority given to regulatory bodies to require assessments of dam safety and to mandate remedial measures where safety appears to be questionable or inadequate. It is the legitimate goal of government, through regulatory bodies, to avoid future St. Francis (California), Teton (Idaho), Johnstown (Pennsylvania) or Baldwin Hills (California) catastrophes. Non-catastrophic failures, which may be expensive, annoying or embarrassing, should also be avoided. Owners of dams may be well advised to evaluate the probability of such failures and to take steps to prevent them. Yet, since their consequences fall far short of the calamities associated with the flood following a catastrophic failure, they fall outside the domains of public safety and the regulatory powers of government, and they do not fall within the scope of this study.

Catastrophic failures have one of four causes: overtopping, piping by backward erosion, liquefaction, or downstream sliding at high reservoir (possibly associated with toe failure due to piping by heave). The first three of these types of failures — overtopping, backward erosion and liquefaction — cannot be predicted by stability analyses. Hence, the legitimate application of stability analyses to catastrophic failure is restricted to downstream sliding, with or without loss of toe support, when there is enough water in a reservoir to do catastrophic damage if released.

Non-catastrophic failures can occur by downstream or upstream sliding, including sliding originating in the foundation, when there is no pool or when the pool is so small that its release is inconsequential. In addition, failures can occur by rapid drawdown, but such failures are not in themselves catastrophic even if the reservoir contains a high pool.

Rapid drawdown has led to significant damage in a number of instances, but there appears to be no record of catastrophic loss of a reservoir resulting from this mode of failure. Therefore, it is not included as a cause of catastrophic failure. However, the potential for catastrophic failure exists if a rapid drawdown slide could block outlet works and if spillway capacity would be inadequate to prevent overtopping in the event of such blockage. Under these circumstances the potential for a rapid drawdown requires assessment.

Critical Periods for Sliding

If an embankment dam were to fail under conditions that could be appropriately defined by a limit-equilibrium analysis, it would do so at one of three critical periods. The first of these is during construction. As the embankment rises, the factor of safety against a slope failure, and particularly against foundation failure, decreases. Such a slide would not be catastrophic unless the pool had been allowed to rise against the embankment as it was being placed. Under these circumstances, whatever pool had been accumulated might escape and cause flooding.

The second critical period is the first filling of the reservoir. If the dam survives the initial filling and if there is no blowup at the toe, the dam can be considered safe (in effect, proof-tested) against failure by piping due to heave.

The third critical period is achievement of maximum pore pressure under a full reservoir. If the dam has not failed when this condition has been reached, its safety against downstream slope failure has been demonstrated. Under many circumstances, including the presence of relatively thin cores or ample well-drained downstream shells, pore-pressure maxima follow so rapidly after the first filling that the survival of the first filling can be considered to be a demonstration of the ultimate safety of the dam under full-reservoir conditions. However, if the impervious section of the dam is thick and impermeable enough to create a time lag between the rise of the reservoir and the rise of piezometric levels in the core or supporting downstream zones, pore-pressure equilibrium may not occur for several years after the reservoir is first filled and the critical period may be delayed. So-called

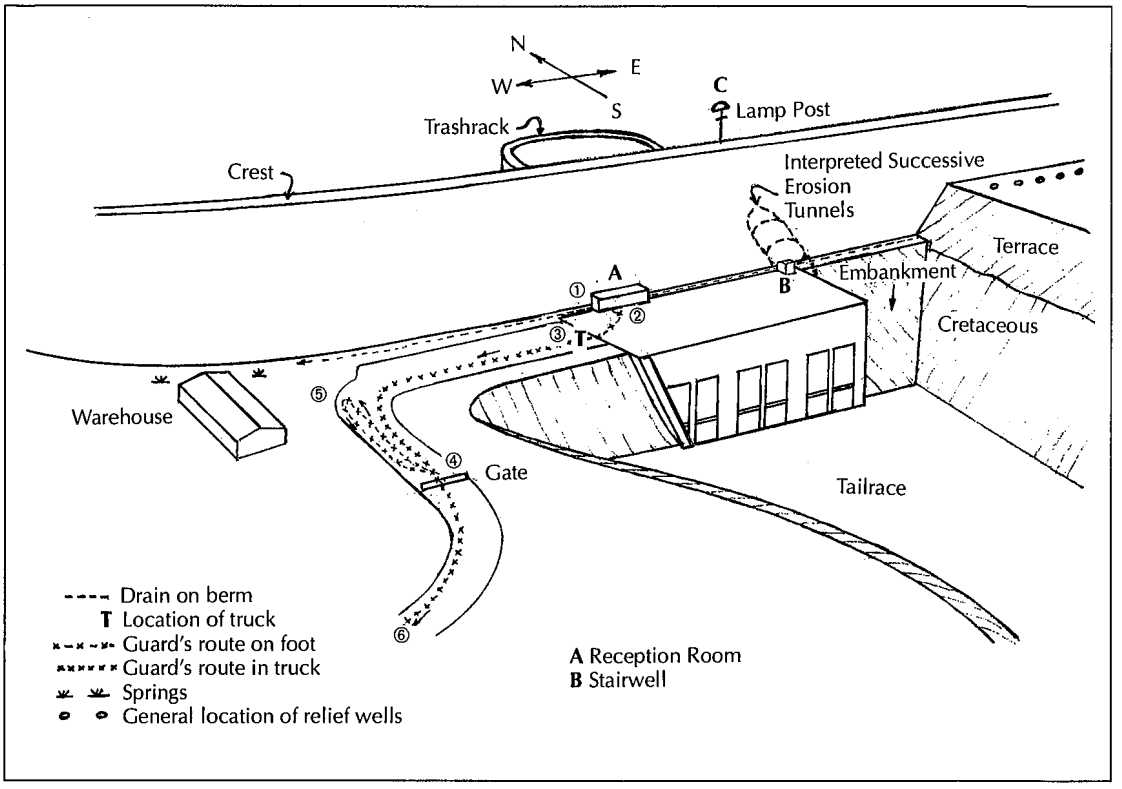

FIGURE 1. Principal features of the Walter Bouldin Dam as related to its failure.

homogeneous dams, especially in regions of high intensity of rainfall, are potentially vulnerable, and at least one failure, a small dam near Ponce, Puerto Rico, has occurred under these conditions.

In principle, it remains generally correct to postulate that if a dam survives the first filling and the corresponding pore-pressure increases under the filled condition, its safety against failure of the downstream slope has been demonstrated. There is one possible exception to this statement.

If the factor of safety is so close to unity that cyclic loading by the pool causes strain softening and critical loss of strength, the factor of safety may decrease. Aside from this special case, surviving the first filling and corresponding pore-pressure increase ensures that the dam is safe against catastrophic failure by any mode to which an equilibrium analysis is applicable.

Failure of Walter Bouldin Dam

Notable for its absence from the listed causes of catastrophic failure is upstream sliding leading to overtopping. Yet, the official cause of failure of Walter Bouldin Dam in Alabama, as set forth in reports of the Federal Power Commission, is an upstream slide that occurred without preceding drawdown and that breached the crest of the dam.[1,2,3] The failure resulted in loss of the reservoir within a few hours.

It is important to the profession and to the public that the failure of Walter Bouldin Dam be correctly explained, as it would otherwise be necessary to include upstream sliding as a cause of catastrophic failure. In this case, the failure was clearly the result of subsurface erosion. The official reports and other sources can be consulted for details about the dam and of the investigations after the failure.[4] The investigations disclosed shortcomings in both design and construction. However, these shortcomings were not responsible for the failure. The investigators did not ignore the possibility of piping, but they concentrated their attention with respect to piping on zones where seepage had been noted and extensive observational and control measures had been

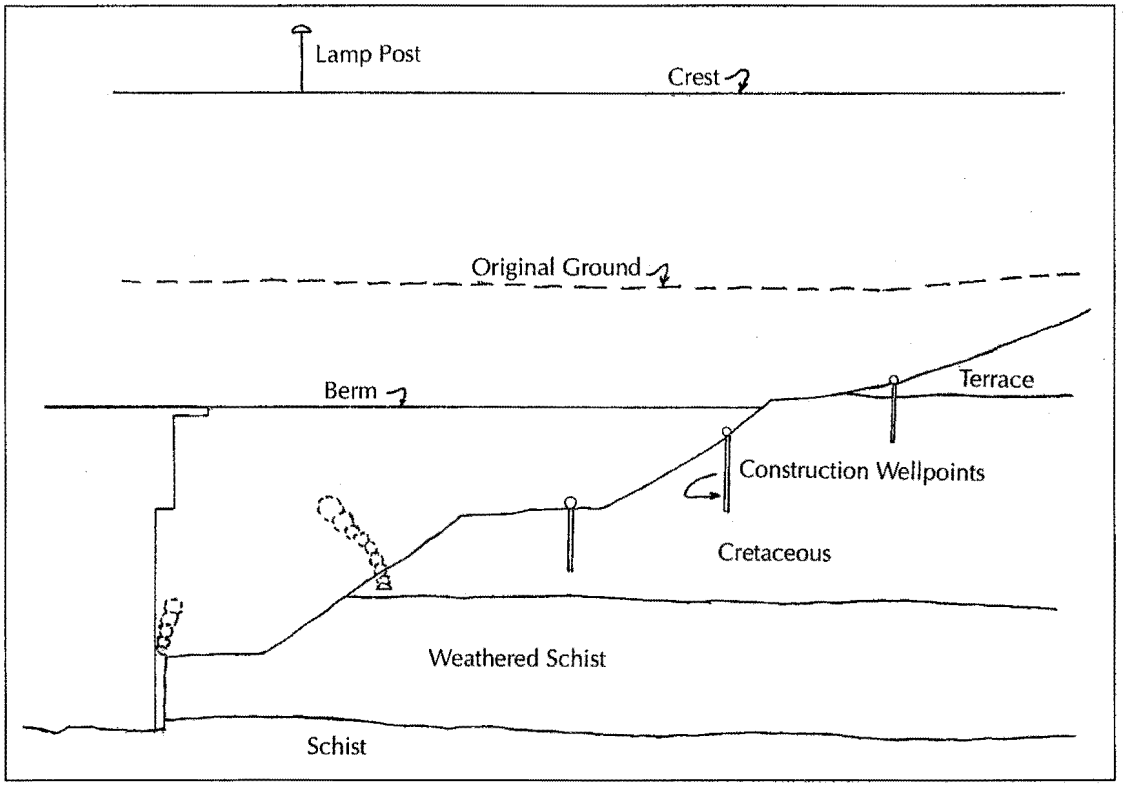

FIGURE 2. Longitudinal section through the east end of the Walter Bouldin power house and the junction with east wing dam.

established. They discounted the likelihood of piping at a lower elevation where it probably occurred.

The essential features of the project included an earth dam about 165 feet high across the deepest part of a valley, flanked on the west and east by wing dams founded on Pleistocene terrace deposits at a higher level. A power house was embedded in the downstream slope of the highest part of the dam; the roof of the power house was at the same elevation as a berm on the downstream slope. The power house's foundation extended into Precambrian schist bedrock overlain by Cretaceous sediments consisting largely of slightly cohesive sands and silts with layers of stiff clay. Beneath the wing dams these sediments were overlain in turn by the terrace deposits. The relationships are shown diagrammatically in Figure 1. Figure 2 depicts a longitudinal section of the area through the east end of the power house and its junction with the east wing dam; it shows the excavation made through the Cretaceous soils into the schist to reach suitable foundation support for the power house raft.

There was one eyewitness to the events leading up to the failure: Mr. Sanford, the night guard. He was interrogated many times in the course of the ensuing investigation and recounted a remarkably consistent series of recollections. As a non-technical person, he had no hypotheses about the causes of failure and apparently had no reason to report other than what he experienced.

The chronology of Sanford's activities on the cloudy, moonless night of the failure can be traced with reference to Figure 1, a simplified sketch of the power house and dam as seen from downstream. He went on duty about 9:45 p.m., made his first routine inspection starting at his office in the reception room (A) at the northwest corner of the roof of the power house, and returned about midnight without having observed anything unusual. After reading the Sunday paper for some time in the reception room, he glanced out a west window

FIGURE 3. Development of erosion tunnel through Teton Dam. (USBR photo.)

(1) and noticed that the paved gutter along the north edge of the roof was running nearly full of muddy water. He was unable to see the drain or embankment directly north of the reception area, because there were no windows on that side of the building. Recognizing that something was wrong, he called his supervisor. The time was 1:10 a.m. Before he completed his brief call, water was coming beneath the door into the room. At first, it was one or two inches deep, but it quickly became deeper.

By this time he had decided to leave. As he did so, he looked around from (2) and saw that muddy water was now flowing over the power house roof, apparently gushing from the northeast corner near the back stairwell (B). He waded through water, now 4 to 6 inches deep, to his truck (T) and drove from (3) to the gate (4). At (4) he got out of the truck, opened the gate, and looked back for a few minutes. He observed that the light (C), which was located 26 feet east of the power house on the crest of the dam, was illuminated, but he could not observe many other details of the dam because of the intervening switchyard and other objects. He thought he saw a cavity behind the stairwell and mist rising behind it, possibly from beyond the crest of the dam.

He then noticed that the water on the road and on the powerhouse roof was getting shallower, so he decided to walk back for a closer look. He stopped (5) in front of the warehouse and while standing there heard rocks begin to fall on the roof and on some of the equipment it supported. There was then an electric flash and the lights went out, including the one at (C). The clock in the control room was later found to have stopped at 1:33 a.m., about 23 minutes after Sanford's phone call to his supervisor. Shortly thereafter, other plant personnel arrived; by that time they could see, with the aid of their automobile lights, that a gap existed in the crest of the dam and that water was pouring through.

Sanford's account is fully compatible with the progressive collapsing of the roof of an erosion tunnel that had penetrated through the dam at the east end of the power house, as suggested schematically in Figure 1. When the elevation of the tunnel reached that of the berm, water was able to flow over the power house roof. As erosion progressed, one of the successive collapses partially blocked the tunnel and the flow decreased, prompting Sanford to return as far as the warehouse. The final collapse, accompanied by falling riprap, breached

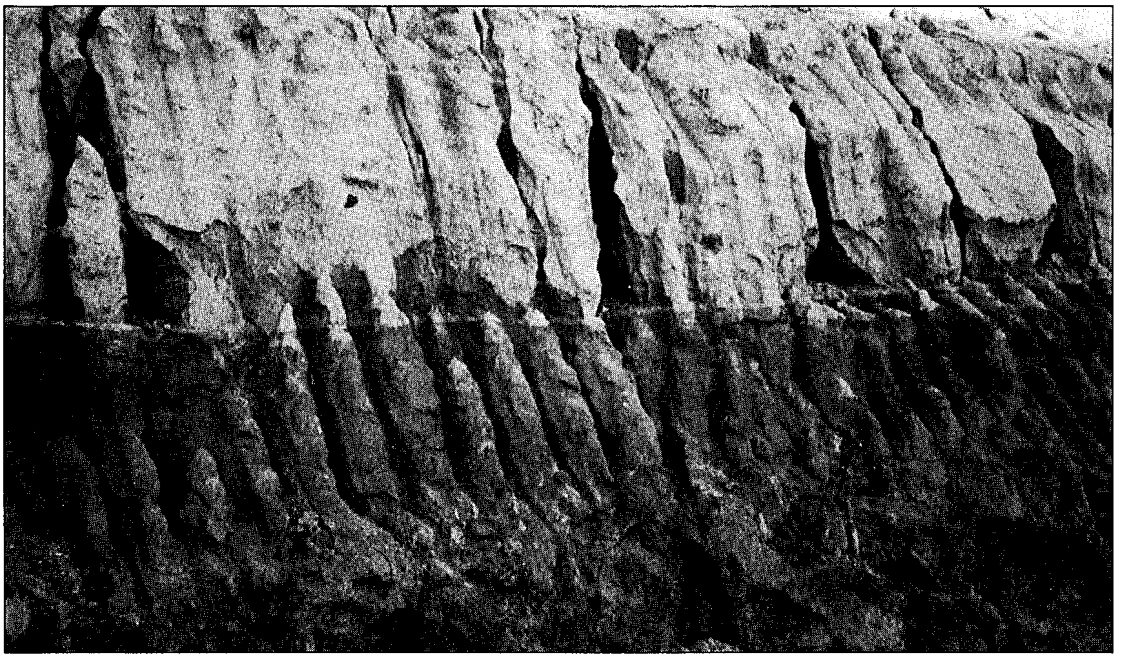

FIGURE 4. Erosion of cut slope in Cretaceous soils during Bouldin Dam reconstruction.

the crest. The similarity of the events to the development of the erosion tunnel through Teton Dam is evident (see Figure 3).

The official reports concluded that the failure started as an upstream slide extensive enough to breach the crest to a level below the reservoir surface, whereupon the water flowed through the gap and initiated the erosion. This scenario is incompatible with Sanford's account. Foremost among the facts that cannot be explained by the upstream slide hypothesis is that the light at the crest of the dam, located where the gut ultimately developed, remained illuminated for more than 20 minutes after muddy water began to flow over the roof of the power house. Had overtopping occurred through a slide-produced gap, the lamp post or its power supply cables would have been among the first casualties, and the events described by Sanford would have taken place in the dark. Furthermore, the temporary decrease in the flow that he observed when he reached the gate is characteristic of blockage caused by collapse of material overlying a tunnel, whereas the flow through an open channel would only have increased with time. It is also noteworthy that the horizontal thickness of the dam at the water line was some 64 feet. To allow the reservoir to escape, a slide would have had to extend upstream at least this distance. The extent of such a slide parallel to the axis of the dam would have had to be of comparable magnitude, several times greater than the 26 feet from the end of the power house to the lamp post at (C). Thus, the crest light would have disappeared with the first earth movement. Yet, it continued to function for at least twenty minutes after the dam was releasing water.

From the time that Sanford first detected trouble, his description fits the classic mechanism of the upward development of an erosion tunnel. The scenario would not be complete, however, unless a set of physical conditions existed that would permit the initial undetected development of such a tunnel. The official reports correctly noted that seepage was expected by the designers and had occurred extensively at the downstream toes of the east and west wing dams where they rested on permeable terrace deposits. Relief wells had been installed, along with other remedial and observation works, to control the seepage. Indeed, the record is replete with references to the attention given to the prevention of piping at and above the Tertiary-Cretaceous interface. The investigators, however, discounted the

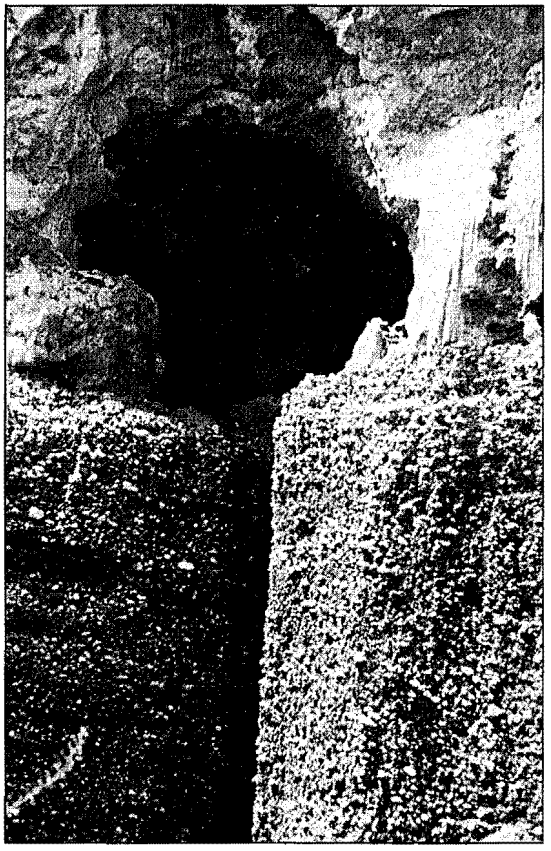

FIGURE 5. Erosion tunnel at the contact of two layers of Cretaceous soils in excavation at the Bouldin damsite. Note the vertical joint in the cohesive gravelly material.

possibility of piping in the underlying Cretaceous materials, although these materials contained many slightly cohesive, highly erodible sands and silts (see Figures 4 and 5).

The foundation raft of the power house was cast inside formwork in an excavation extending a few feet into somewhat weathered schist underlying the Cretaceous beds. An approximate section through the edge of the raft and adjacent materials is shown in Figure 2. The geometry of the backfilled zone would have favored seepage along its boundaries. Seepage could also have developed readily through the joints and more pervious zones in the Cretaceous deposits. Indeed, the excavation for the power house and intake structure required dewatering by well points; three tiers were installed, the lower two of which were entirely in the Cretaceous beds. Thus, avenues for seepage were undoubtedly present. Backward erosion could have gone unobserved below tailwater level for a long time until finally an erosion tunnel reached the reservoir and permitted concentrated destructive flows to cause rapid enlargement and failure.

The course of events before the tunnel reached the level of the power house roof is speculative, but the existence of conditions favorable to the development of a tunnel by backward erosion is not. Neither are the events observed after the tunnel reached the level of the power house roof. The conclusion seems inescapable that failure actually occurred by piping, and that any supposed shortcomings in the construction of the embankment, even if they existed, were irrelevant.

The official reports postulate an upstream slide without the destabilizing influence of a drawdown. This explanation in itself is logically questionable. An earlier shallow drawdown slide in the steep upstream slope had occurred, however, and had been repaired by dumping crushed stone and rockfill in the affected area. The investigators reasoned that the strength of the clayey materials under the dumped fill had gradually deteriorated as their moisture content increased, and that at the time of failure the strength had reduced to the extent that the slide was reactivated. This hypothesis, like that of any upstream slide, is incompatible with the events recounted by Sanford and with the extent of a slide that would have been required to lower the top of the dam below reservoir level.

In short, the failure of Walter Bouldin Dam occurred because of piping by backward erosion. As no other example of catastrophic failure and loss of reservoir has been attributed to an upstream slide, it may be concluded that a dam that has successfully survived construction will not experience a catastrophic upstream slope failure. Any analysis that indicates otherwise must be erroneous. Further, a stability analysis to investigate the safety of the upstream slope under full reservoir is irrelevant. If an upstream slope does not fail during construction, its factor of safety must exceed unity under the more favorable condition of reservoir loading. Rapid drawdown may induce a failure, but such a failure is shallow and has never been known to cut back into

the embankment far enough to permit overtopping and cause the catastrophic loss of a reservoir.

There is a remote possibility that a dam consisting of fine-grained soils possessing shear strength due to capillarity, or containing stiff cohesive materials susceptible to swelling, may lose strength when submerged if the thickness of stable upstream shell material is inadequate. The upstream slopes of dams containing such materials are usually fairly flat and failure surfaces would tend to be shallow, with consequences similar to those of drawdown failures. Again, no catastrophic failure of this type is known.

If Walter Bouldin Dam is eliminated from the category of failure by upstream sliding, then it may be concluded that once a reservoir has been filled and the associated pore-pressure increases have been achieved, the factor of safety is at least equal to unity with respect to limit-equilibrium conditions, and that any calculation showing a factor of safety less than unity must be in error.

Downstream Slope Failures

The factor of safety of a dam that survives its first filling and the associated increases in pore pressure will increase with time, unless this factor of safety is so close to unity that cyclic loading produced by fluctuations in the level of the pool causes strain softening and a critical loss of strength. If, however, the factor of safety is indeed so close to unity, downstream slope failure will be preceded by progressive and increasing increments of movement at successive full pool levels. Any calculation showing a factor of safety appreciably different from unity under these conditions must be erroneous. Stability calculations are thus irrelevant in assessing the safety of such a structure; observations of movement must take their place. If successive periods of full reservoir are accompanied by decreasing increments of movements, the stability of the structure is increasing. If the contrary occurs, the stability may be decreasing. Stability calculations may be useful in judging the influence of various remedial measures, but the computed magnitudes of the factor of safety are meaningless. An outstanding example of the irrelevance of stability calculations under conditions of decreasing increments of movement is Gardiner Dam on the South Saskatchewan River in Canada. The case of this dam illustrates the limitations of equilibrium stability analyses.

Behavior of Gardiner Dam

Conception, design and construction of Gardiner Dam took place during the quarter century in which understanding of shear strength was undergoing its most radical revisions, and at each step in the evolution of the design the geotechnical studies reflected the new frontiers of knowledge. The following history is greatly abbreviated, perhaps beyond tolerable limits, but since the project has been exceptionally well documented, the interested reader can readily learn the details.[5,6,7]

At the site the South Saskatchewan River flows in a valley cut into the Cretaceous Bearpaw formation, of which the main shale member at the site, the Snakebite, is of high plasticity and contains bentonite or bentonitic zones with liquid limits ranging up to about 300. The depth of the shale bedrock valley at the site is about 75 m, but the bottom 30 m are filled with alluvium. The valley is bordered by wide zones of slump or landslide topography giving testimony to the propensity for stability problems during excavation and fill placement (see Figure 6).

In 1943, the Prairie Farm Rehabilitation Administration of Canada (PFRA) began studies for an irrigation project involving a dam across the river. Total stress stability analyses were the rule at the time, and the initial design was based on two sets of undrained peak-strength parameters: $c = 15$ to 20 psi, $\phi = 10°$; and $c = 20$ psi, $\phi = 0°$. Circular surfaces of sliding were assumed, and a factor of safety, $FS = 2.7$, was adopted for the end-of-construction condition.

The early geotechnical studies were carried out by Robert Peterson with equipment representing the latest Harvard designs. Subsequently, Arthur Casagrande, engaged as a consultant, turned the emphasis to a study of the slumped slopes in the vicinity supplemented by laboratory tests on a few select samples, by a test drift in which surfaces of sliding in the shale could be observed, by instrumentation to detect movements and by installation of

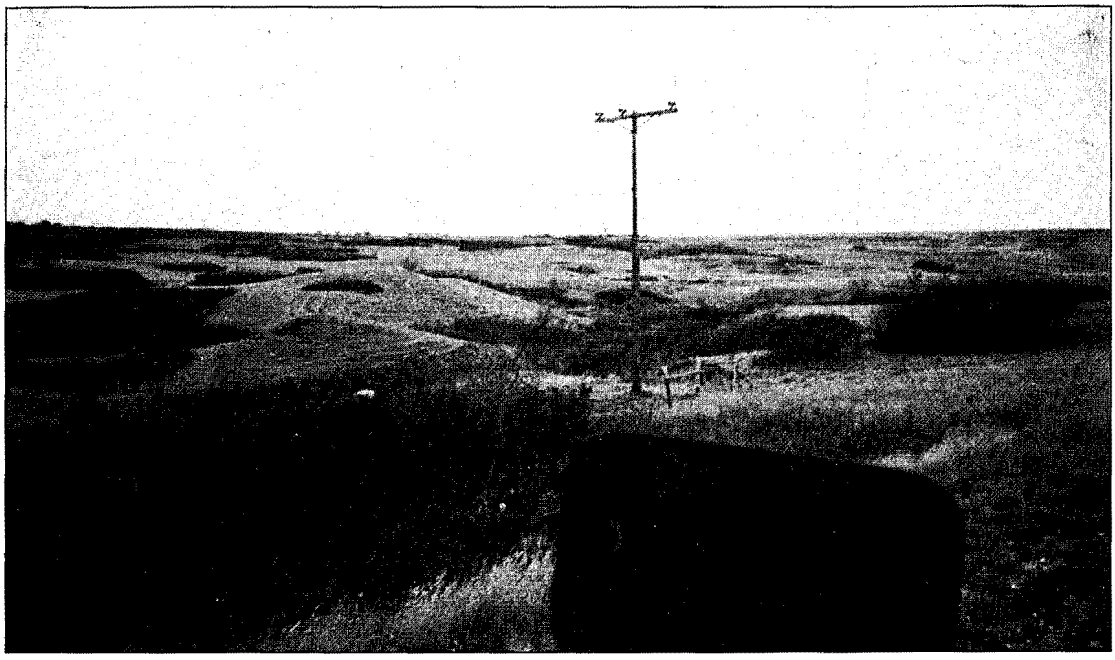

FIGURE 6. Landslide topography downstream of Gardiner Dam.

piezometers. Backfiguring the undrained strengths of the shales from the observed natural slumped slopes, on the assumption of circular surfaces of sliding, led to the conclusion that the most appropriate values for design were c = 3 psi and ϕ = 3°. A factor of safety of 1.1 was considered adequate for the end of construction because the interpretation was believed to be conservative. Even so, the design slopes required flattening.

During the early stages of construction from 1959 to 1961, however, minor excavations reactivated slips at calculated factors of safety greater than unity, according to analyses based on the design parameters, and a re-evaluation was undertaken. By now, effective stress analyses had become the norm. When the reactivated slips were backfigured, with composite surfaces of sliding and with piezometric levels assumed to be near the upper surface of the shale, shear parameters $c' = 0$, $\phi' = 8.5°$ to 11.5° were found. A reanalysis of the slumped slopes as they existed before construction activities, carried out under the same premises, indicated $c' = 0$, $\phi' = 5°$ to 7°. The design was then revised on the basis of $c' = 0$, $\phi' = 9°$ in the foundation, and the slopes were flattened accordingly.

A minimum value $FS = 1.4$ was postulated for short surfaces of sliding; $FS = 1.2$ was considered sufficient for long surfaces emerging near the toe at locations where stabilizing fill could be added, and $FS = 1.3$ was required where such stabilization would not be practical because of physical constraints. Construction proceeded on this basis through 1964. Although up to 0.3 m of foundation displacement toward the river channel was noted during stage construction on the west side of the river, the deformations were considered acceptable because there was no visible indication of upstream cracks in the embankment or of overthrusting at the toe. As piezometric data accumulated, the calculations were refined by incorporation of observed values of the pore-pressure ratio, r_u.

However, when the river-section embankment was raised from a height of 26 m to 47 m in 1965, horizontal displacements of as much as 0.8 m took place in the downstream direction. In 1966, when the dam was raised only 11 m from this height to nearly its full height at the center line, further movements of about 0.3 m occurred. The slopes, which had already been flattened after each redesign, were flattened once more. The final slopes were extended to a distance of about 1,200 m downstream to

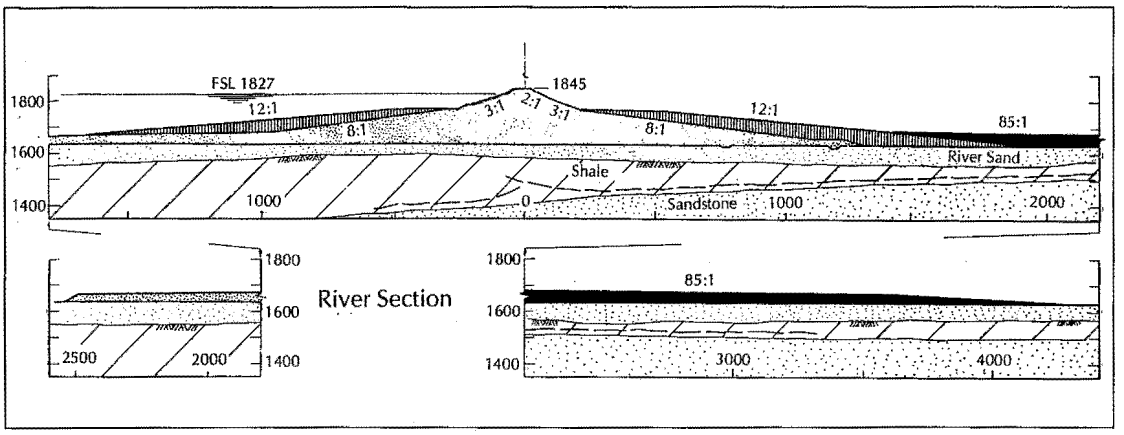

FIGURE 7. Successive stages in flattening of slopes of Gardiner Dam (after PFRA 1980).

prevent an overthrust from developing from the ancient shear zone located some 50 m below the valley floor. Three of the successive stages in the flattening of the slopes are shown in Figure 7. Although the increment of fill near the crest had produced a disproportionately large increment of movement, the rate slowed rapidly, and the embankment was completed to its designed crest elevation. In all, a maximum displacement of over 2 m occurred in the region about 150 m downstream of the center line. The displacement was associated with an upstream-downstream compression of the downstream shell, as the displacement at the toe of the embankment 1,200 m downstream of the center line was less than 1 cm during the same time interval.

The reservoir was raised for the first time, although not to full pool, in 1967. The raising was marked by a substantial increment of downstream displacement. Each annual rise of the reservoir has been accompanied by an additional increment, although a trend for the magnitude of the increments to decrease is evident if the different peak annual reservoir levels are taken into account. However, in view of the large movements already experienced during construction, the reservoir-induced movements prompted further evaluation of the structure's safety. Many factors were taken into account, including the numerous features of the design that were introduced to cope with the large movements that had been anticipated:

- a cross-section capable of accommodating substantial deformations without rupture of the core;
- the remoteness of the possibility of cracking and leakage through the core;
- the great depth of the preexisting shear surface below the river bed; and,
- the extensive installations of slope indicators, piezometers and other means of field observations provided to permit close surveillance.

Indeed, this and subsequent evaluations by the PFRA and its ongoing Boards of Consultants, which have enhanced confidence in the inherent safety of the dam, have placed principal dependence on the results of the observational program and the favorable trends that it has disclosed.

In his 1964 Rankine Lecture, Skempton introduced the concept of residual strength on surfaces along which large displacements had occurred.[8] It had by then become apparent that the surface of sliding constituting the seat of principal movement was an ancient shear zone in the hard shale near the base of the Snakebite member. The zone extended from an area beneath the upstream shell of the dam to at least 1,200 m downstream of the crest. According to Skempton's concepts, it should certainly have been at residual strength.

Reversing direct-shear tests on samples from the shear zone indicated $c_r' = 0$, $\phi_r' = 2.7°$ to $3.3°$; rotational-shear tests indicated $c_r' = 0$, $\phi_r' = 3.5°$ to $4.5°$. Back analyses using measured piezometric levels and composite surfaces of

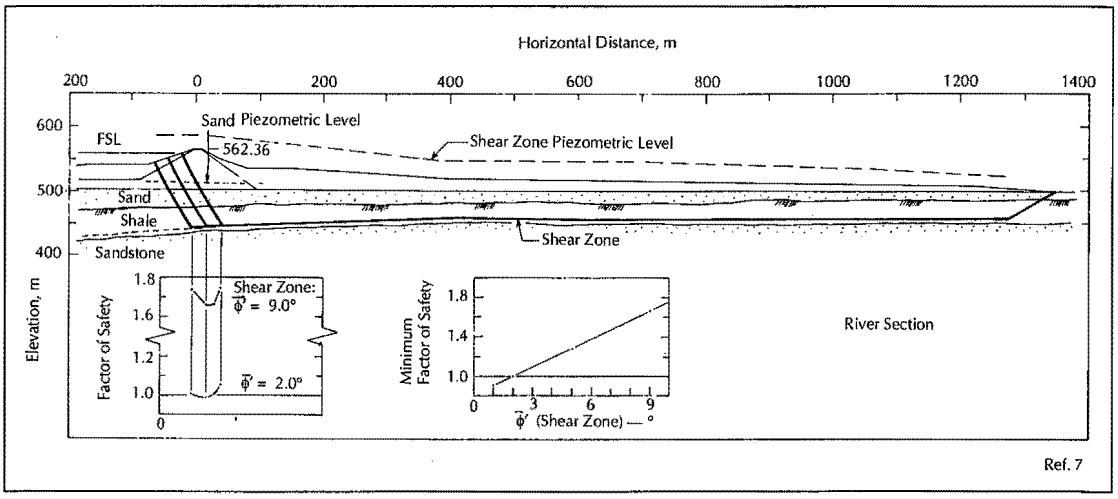

FIGURE 8. Results of stability calculation for the river section of the Gardiner Dam on the basis of the best information available in 1979.

sliding corresponded to $c_r' = 0$, ϕ_r' ranging from 2° to 7°. (In reality, three different embankments make up Gardiner Dam — all experiencing a similar history. Part of the range in the shear parameters may be ascribed to differences in the geometry and geology of the sections.) Although the shear-strength parameters from the tests and back calculations are in general agreement and leave little doubt that the residual strength is the pertinent parameter, the range in back-calculated values of ϕ demonstrates that the margin of uncertainty in such stability calculations is still appreciable.

One of the most recently published examples of such a calculation for the river section is illustrated in Figure 8.[7] It represents a parametric study in which the factor of safety was calculated for various residual friction angles from 2° to 9°. The composite surface of sliding included the observed shear zone in the foundation, an active segment (taken at 3 different positions) along which peak strength was assumed, and a passive segment at the downstream end at a location corresponding to observation. Measured piezometric levels along the shear zone and in the embankment were used. The results show that for $FS = 1.0$ a residual friction angle of only 2° is needed, but even at this small value a maximum shearing displacement of over 2 m occurred during construction. The calculations illustrated are for full pool, but similar calculations for low pool, corresponding to conditions at the end of winter, show a negligible influence of pool elevation on the factor of safety. Inasmuch as each application of the reservoir load has actually produced a distinct increment of displacement amounting to several millimeters, it is apparent that even these calculations are not yet at a stage where they are definitive predictors of behavior.

Considering the talent that was brought to bear on this project over a period of more than forty years; the degree of care and sophistication in soil testing; the data obtained from some 83 slope indicators, over 500 piezometers and an even greater number of surface reference points; and the care and continuity with which the observations have been carried out, there arises an inescapable conclusion. Forty years of research have reached the point where a limit-equilibrium stability analysis can indeed demonstrate that the calculated factor of safety of the structure is equal to the known value of approximately unity. Even so, the analysis is still deficient inasmuch as it does not yet realistically account for the influence of the water level in the reservoir. The analysis is still deficient in spite of the outstanding resources of the PFRA and the unusual effort that has been expended as compared to the investigations possible in connection with the usual dam-safety assessments.

The unusual geometry and physical proper-

ties of the mass undergoing movement in Gardiner Dam impose inherent limitations on limit-equilibrium analyses. The limitations have been recognized, and finite-element studies have been carried out with sophisticated refinements.[9] By assigning what appeared to be reasonable values to the physical properties of the various materials involved, and by adjusting these values as required to achieve agreement between predicted and observed behavior, a model was developed that could reproduce deformations similar to those in the field, including the pattern of response to cyclic reservoir operation. Prediction over many load cycles, however, was not satisfactory. Although the study provided valuable insight into the behavior of the dam, it is clear, nevertheless, that no such finite-element study, without the calibration afforded by extensive observational data, can yet be depended on to indicate the degree of safety of this or probably any other existing dam.

Conclusions

The foregoing discussion leads to the conclusion that stability analyses are unreliable bases for assessing the stability of an existing dam with respect to catastrophic failure. The conclusion strictly applies only to static conditions; insight regarding the behavior in an earthquake may be gained by analysis, although not generally by limit-equilibrium analyses.

If the pool has been filled and pore-pressure equilibrium has been reached, the results of stability analyses may be assessed as follows:

1. If the calculated factor of safety is less than unity, it must be erroneous.
2. If the calculated factor of safety is greater than unity, the results merely indicate the obvious. The calculation is unnecessary to show that the dam is standing. Furthermore, no definitive conclusion about the degree of safety can be drawn from the numerical value of a computed factor of safety: hence, satisfying some prescribed criterion for this value is not in itself a suitable indicator of safety.
3. If progressive movements are occurring, a calculation is irrelevant because the factor of safety is obviously close to unity. The actual safety can be assessed only on the basis of monitoring the movements and associated events. The procedure is exemplified by the studies at Gardiner Dam, where the crucial observations were those indicating decreasing increments of movement under successive comparable reservoir fillings. Nevertheless, if the calculated factor of safety is approximately unity, limit-equilibrium calculations may be useful in judging the effectiveness of various alternatives for increasing the safety. This use of equilibrium analysis is justifiable, and its effectiveness has been demonstrated not only with respect to dams, but with respect to many natural slopes. It should be clear, however, that the absolute value of the factor of safety resulting from any of the calculations is of no significance.

Stability analyses are tools for the guidance of the investigator. They have their limitations with respect to evaluating the stability of existing dams. It is not meant that they should never be performed. However, the numerical values for the factor of safety should carry little if any weight in judging the actual safety of the structure with respect to catastophic failure.

The great danger in placing too much emphasis on stability calculations is that they may be regarded as a substitute for the much more difficult and expensive field investigations and historical research needed to establish the real character of the structure in question. Some dam owners may prefer the relatively small expenditure for a perfunctory stability study in contrast to costly and time-consuming field studies. Of greater importance, because of the greater potential danger, some regulatory bodies may take more comfort in orderly stability calculations based on unsupported or unverifiable assumptions than in qualitative judgments based on experience and careful investigation. Yet, the former may have little or no relation to the real safety of the dam, whereas the latter are essential in assessing the likelihood or possibility of a catastrophic failure.

This discussion quite possibly conveys a negative impression about our ability to deter-

mine whether or not an existing embankment dam is demonstrably safe against a catastrophic failure. This impression is not the intention of this study, nor would it be a correct summarization. On the contrary, a sound engineering appraisal can almost always be reached, but such an appraisal will often require painstaking field investigations; searches for construction records and, if they are found, their assessment and interpretation; inspection of all visible features; location and interpretation of maintenance records; and in many instances installation and monitoring of instruments and devices capable of disclosing not only present behavior but also future trends. The latter requirements may sometimes preclude an immediate evaluation of the safety of the dam, and they may sometimes prove to be expensive. On the contrary, they may also sometimes establish the safety quickly and beyond reasonable doubt. However, factors of safety derived from stability analyses, even when based on what appear to be the most reasonable of assumptions, are dangerous substitutes for the thorough investigations that are needed to reach sound, even if qualitative, judgments.

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REFERENCES

1. "Report on Investigation by Atlanta Regional Office Staff of Walter Bouldin Dam (Project No. 2146), Failure of February 10, 1975," September 1975.

2. "Investigation of Failure, Walter Bouldin Dam and Safety of Other Dams of the Alabama Power Company," Federal Power Commission, Bureau of Power, February 1976.

3. "Report to the Federal Energy Regulatory Commission, Walter Bouldin Dam Failure and Reconstruction," Office of Electric Power Regulation, FERC, Washington, DC, September 1978.

4. Sowers, G.F., "Reflections on the Design of Earth Structures," Contribution to the Fifth Nabor Carrillo Lecture by Raul J. Marsal, Mexico, 1980, pp. 133-141.

5. Peterson, R., Jaspar, J.L., Rivard, P.J., and Iverson, N.L., "Limitations of Laboratory Shear Strength in Evaluating Stability of Highly Plastic Clays," *Proc. Research Conf. on Shear Strength of Cohesive Soils, ASCE*, Boulder, CO, 1960, pp. 765-791.

6. "The Design and Construction of Gardiner Dam and Associated Works," PFRA, Canadian Gov't Publ. Ctr., Hull, 1980, 382 pp.

7. Jaspar, J.L., and Peters, N., "Foundation Performance of Gardiner Dam," *Can. Geot. Journal.*, Vol. 16, No. 4, 1979, pp. 758-788.

8. Skempton, A.W., "Long-Term Stability of Clay Slopes," *Géotechnique*, Vol. 14, No. 2, 1964, pp. 77-101.

9. Morgenstern, N.R., and Simmons, J.V., "Analysis of the Movements of Gardiner Dam," *Proc. 4th Int. Conf. on Numerical Methods in Geomechanics*, Edmonton, Vol. 3, 1982, pp. 1003-1027.