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# Risk Analysis Perspectives on the Camará Dam Failure

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***Abstract.*** The June 2004 breach of Camará Dam, Brazil, is one of the few documented failures involving a Roller Compacted Concrete (RCC) Dam. The details of the failure, as well as the aftermath of the flooding, have been previously discussed, and many of the original source documents are available. From the risk analysis perspective, however, estimators continue to struggle with the mechanisms that could result in an RCC dam underperforming, and there is often a sense that RCC structures are more forgiving than those built of conventional concrete. While the case history literature suggests that a concrete dam is only as good as the underlying rock, it is important to question the idea that an RCC design is ideally suited for every geologic setting, and that the Camará Dam failure was exclusively the result of a problem with the foundation. This paper advances, in the form of a site-specific potential failure mode (PFM), an alternative conceptual model for what happened, discusses whether such a process could have been reasonably anticipated using risk analysis, and argues that a well-known concrete dam case history could have been recognized as a prototype. With the benefit of hindsight, conditional probabilities are estimated for the events that comprise the PFM, and the life loss consequences of the breach flood are compared to what might have been predicted using the Reclamation consequence estimating methodology (RCEM). The results of the evaluation are used to highlight the actions that can be taken, both during and after construction, to address the types of problems experienced at Camará Dam. The paper also provides an overview of the engineering geology and discusses opportunities to improve geologic design data collection in the interest of preventing a similar occurrence.

## I. Introduction

The June 2004 breach of Camará Dam, Brazil, is one of the few documented failures involving a Roller Compacted Concrete (RCC) Dam. The events involved a new dam built using contemporary construction techniques, and, unlike many concrete dam failures, occurred relatively recently (as opposed to 100 years ago). As such, the case history would appear to represent a logical source of information and insight on the potential susceptibilities of dams of this type. While most dam safety professionals are familiar with the widely publicized images of water pouring though a gaping hole in the left abutment, some of the more easily accessible written documents contain information that is either incomplete or anecdotal. This can be explained in a variety of ways (the failure did not occur in the United States or involve a U.S. company; original source documents are in Portuguese), but regardless of the reasons, the concern is that an incomplete understanding of what happened could lead to the wrong lessons being learned (i.e., *my concrete dam foundation does not include erodible soils so we should be okay*). The scarcity of in-depth documentation has also made it difficult to apply the case history in the risk analysis setting, and within the broader risk informed decision making context.

The original and secondary source documents generally agree on the basic facts of what happened; beyond that, inconsistencies (possibly associated with translation difficulties) begin to appear. Camará Dam was completed in 2002 with a structural height of roughly 60 m, a crest elevation of 465, and an uncontrolled (overflow) spillway sill elevation of 461. On the evening of June 17, 2004, with the reservoir approaching elevation 455 (a first fill condition), the dam breached suddenly and catastrophically. The failure occurred in the left abutment area and resulted in the reservoir emptying through a tunnel-like opening along the dam foundation contact (the remaining bridge of intact concrete did not collapse until over a week later). The failure flood resulted in several fatalities downstream, as well as property damage to homes and business.

A more detailed description of the factual events preceding the failure can be found in Kanji [1]:

- On October 19, 2002, with the reservoir elevation exceeding 431 for the first time, a seep was discovered at the crown of the left drainage gallery. The seep was considered unusual because its location was only slightly lower than the reservoir elevation. It was suspected that cracking had occurred, and recommended that the upstream face be inspected by divers (work not performed).
- Between November 2002 and January 2004, the reservoir elevation remained in the 431-435 range, and there were no reports of new seepage.
- On February 10, 2004, with the reservoir higher than El. 445 for the first time, it was determined that several of the left foundation drains were plugged, and that eroded foundation materials were present in the drainage gallery. In addition, a seep was noted along the lower left groin.
- On March 11, 2004, with the reservoir elevation above 451, additional eroded materials were observed, along with new wet spots along the left drainage gallery.
- On May 21, 2004, it was reported that the lower portion of the left drainage gallery was flooded, and that over half of the foundation drains were plugged. Of the functioning drains, three were producing significant flows. Seepage along the left groin was also reported to be increasing.
- On May 25, 2004, the flooded gallery was manually drained (method not entirely clear).
- On June 7, 2004, it was recommended to the owner that a reservoir restriction be implemented. However, the reservoir elevation continued to increase until the dam failed.
- On the evening of June 17, 2004, the dam breached. Although the breach was not witnessed by anyone, two loud bangs were heard before the initial observations of flooding.

The report by Dr. Kanji [1] provides a detailed description of the foundation geology and hypothesized failure mechanism, and is the primary source document used by the present authors. The report was carefully translated using online tools (paragraph by paragraph) and checked against the original for consistency. Another good source of information is the report prepared by the University of Paraíba Technology Center [2], which contains more detail on the construction project organization and discusses the numerical modeling performed after the failure. In terms of life loss consequences, the most useful source document found was a news article written about a month after the failure [3]. The life loss reported in this article is different than mentioned in a 2014 ASDSO conference abstract [4], though the paper associated with the abstract could not be located and the reasons for the difference (e.g., more fatalities discovered or attributed, different source documents used) is unclear. Both of the reported values are considered in the consequences discussion below.

## II. Engineering Geology Overview

Camará Dam is located in the northeastern Brazilian state of Paraíba, within the headwaters of the Mamanguape River. The dam itself is located on the Riachão River, which is a tributary of the Mamanguape River. The Mamanguape River provides the primary drainage to the region and drains eastward toward the Atlantic Ocean, located approximately 100 km from the dam site. The topography of the area is represented by the Borborema Plateau, which serves as an orographic barrier against moist, easterly winds sourced from the Atlantic Ocean. This increased moisture is coupled with steep topography near the dam site, where the topography follows a slope of approximately 10 m/km. As the river approaches the coast, the slope decreases to approximately 1 m/km. Given the steepness of the topography near the dam, the Borborema Plateau is deeply incised by the Mamanguape River in the upper reaches.

The Borborema Plateau is composed primarily of igneous and metamorphic rock and is part of a large anastomosing shear zone. This shear zone is characterized by alterations due to “high temperature, medium- to low-pressure metamorphism, partial melting of the crust, and magmatism which involves both crustal- and mantle-derived magmas” [5]. At the dam site, features related to the shear zone are evident in the bedrock materials.

Camará Dam is founded on metagranitoids typical of the Precambrian Riacho Formation. These rocks are primarily composed of garnet-muscovite-biotite-metagranite (metamorphosed granite common in mountainous areas) and migmatite (partial melting of metamorphic rock) of syenitic (coarse-grained intrusive igneous rock similar to granite, but deficient in quartz) to monzogranitic composition (biotite-granite rock, generally felsic), with a metasedimentary crustal source [6]. After the failure occurred, geologic mapping documented by Kanji delineated geologic units in the left abutment consisting of migmatite underlying metagranite, biotite shale, and granite [1].

Migmatite generally occurs within extremely deformed rocks, such as those associated with the base of eroded mountain chains, typically within Precambrian cratonic blocks. Migmatite is a banded rock, comprised of leucocratic (felsic, lighter) portions, which represent the mobilized (and more resistant) rock, and mesocratic (darker) portions, which represent more or less unmodified parent rock. In the left abutment of Camará Dam, migmatite banding is observed to occur parallel to the river in an upstream to downstream orientation. As a result of the banding between more ductile and more brittle materials, "ripples" were formed as noted in the post-failure migmatite exposures in the left abutment. Foliations within the migmatite strike upstream to downstream, and dip approximately 35 degrees toward the right abutment. Both the dip and ripple features in the migmatite would prove to be significant to interpretations made of the geology during construction.

While the underlying migmatite appears to be present consistently from the top to the bottom of the left abutment, the overlying metagranite, shale, and granite do not share the same consistency due to the presence of key structural features. This made the complex geology in the abutment even more difficult to decipher during construction and pre-construction investigations.

As is common in a sharply incised mountainous landscape, stress relief fractures are present parallel to the ground surface in both abutments of the dam, extending in the upstream to downstream direction. These stress relief fractures were especially prominent in the left abutment, where the overlying metagranite was noted to be heavily altered during post-failure investigations. In drawings developed by Kanji following the failure [1], the fracture planes and foliations within approximately 5 m of the ground surface were noted to contain residual soils and saprolitic materials. The degree of alteration was noted to decrease with depth, with most of the altered rock occurring within approximately 10 m of the ground surface. However, these fracture planes and foliations did not appear to be continuous from the top to the bottom of the left abutment along the dam centerline. Post-failure photos show that, while the contact between the metagranite materials and underlying migmatite materials dips at approximately 35 degrees, the contact between the migmatite and the overlying biotite shale and granite has a significantly steeper dip, which is attributed to a potential fault [1]. Refer to Figure 1 for geologic contacts.

Unaltered granite in contact with the underlying shale appears to have been the material that was exposed at the ground surface during construction of the dam. While the deeper metagranite contained numerous alterations which contributed to the formation of residual soils and opportunities for seepage paths through the rock, the granite exposed at the ground surface appeared to be relatively intact and fracture free in construction photographs. This complex geology became increasing prone to misinterpretation when, during construction, a soil-filled shear zone was encountered near the base of the abutment.

Construction photos indicate that the rock surfaces for the dam footprint along the abutments and main section were stripped and cleaned. While the extent of foundation treatment is unknown, it can be inferred that some effort was made to shape or level uneven rock surfaces given the timeframe that the dam was constructed (2000 through 2002). When a soil-filled shear zone was encountered near the base of the abutment, efforts were made to further investigate and treat this zone. However, those investigations were limited to three drill holes and did not cover the extent of the shear zone. The drill holes were limited to the lower left abutment, proximate to the location of the visible shear, and were shallow in depth. The drill holes seemed to indicate that the shear zone was thinning as it extended into the abutment, and an interpretation was made that the shear zone likely pinched out at some distance beyond the furthest left drill hole. [1] Refer to Figure 2 for photos during construction and after breach.

With the presence of residual soils in fracture planes and along foliations in the upper 5 to 10 m of the foundation, the rippled geometry of the contact between the migmatite and the metagranite became especially significant to how the foundation of the left abutment would be interpreted during construction. Post failure investigations would support the idea that this shear zone, sourced in the heavily altered metagranite, actually tracked much further up the left abutment, following the 35-degree foliation of the migmatite and overlying metagranite contact. Given the rippled geometry of the migmatite contact, the shear zone was observed to repeatedly narrow and thicken, in a sausage-link or "almond" [1] pattern.

Given the interpretation during construction that the shear zone pinched out, the selected remediation was to excavate and replace the soil infilling material, with care taken to not disturb the large granite surface slab; this ultimately prevented observation of the heavily altered metagranite below. The remediation further included shotcrete applied to the surface for slope protection, grout injections and concrete fill placed in the lower extent of the shear zone, and upstream protection to the interface between the RCC and foundation. During construction of the remedial feature, additional soil was removed and replaced with concrete. The final remediation product featured a concrete key constructed for passive stability. It was reportedly noted that removal of the unstable wedge of rock above the soil shear zone would be ideal; however, this option was not pursued due to construction safety considerations and impact to the construction schedule.

 

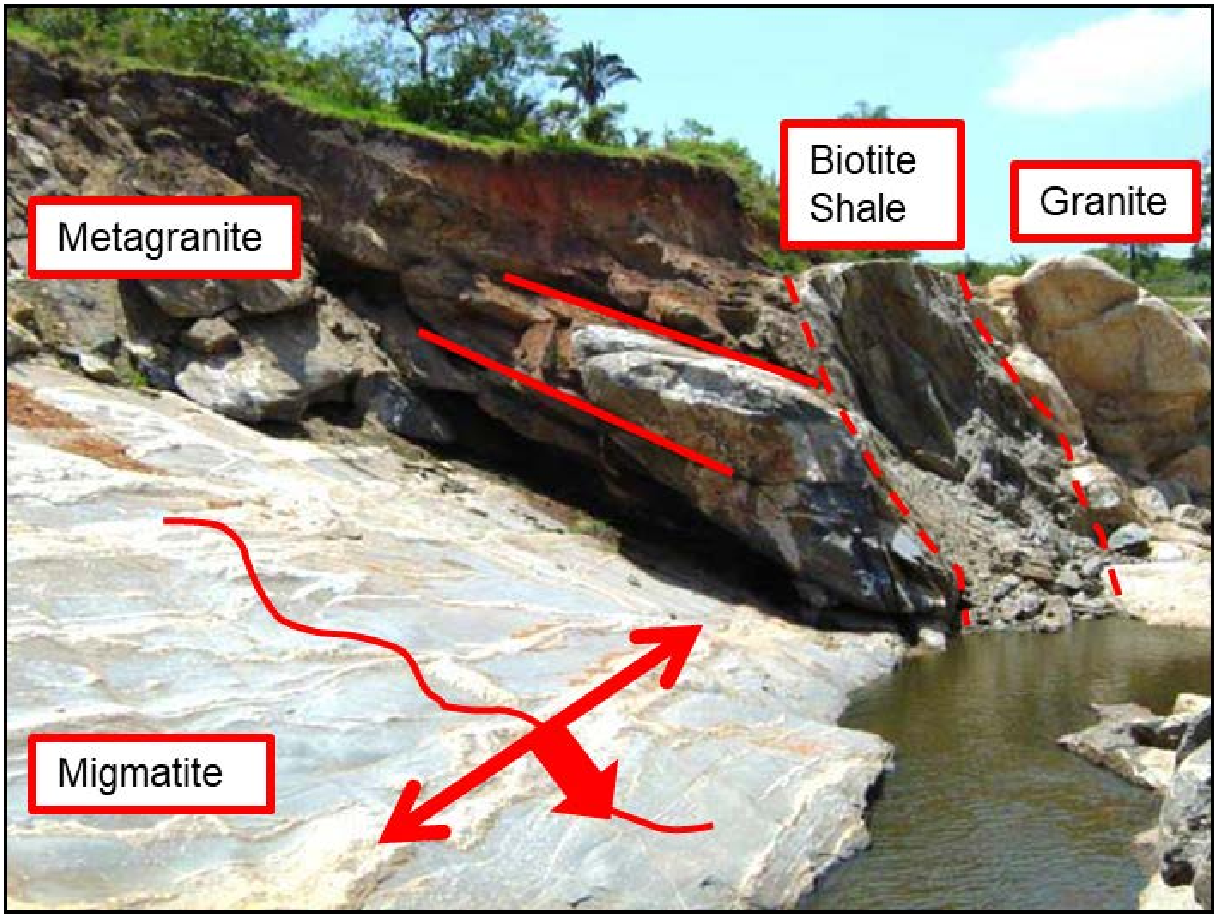


Figure 1. Photo adapted from Kanji [1]. This photo illustrates the foliation of the migmatite, which strikes upstream to downstream and dips approximately thirty-five degrees. The light and dark alterations of the migmatite (representing ductile/brittle materials) causes "ripples" in the rock. Red lines illustrate the stress relief fractures in overlying, heavily altered rock. Note the unconformity with granite exposed at the surface of the abutment.

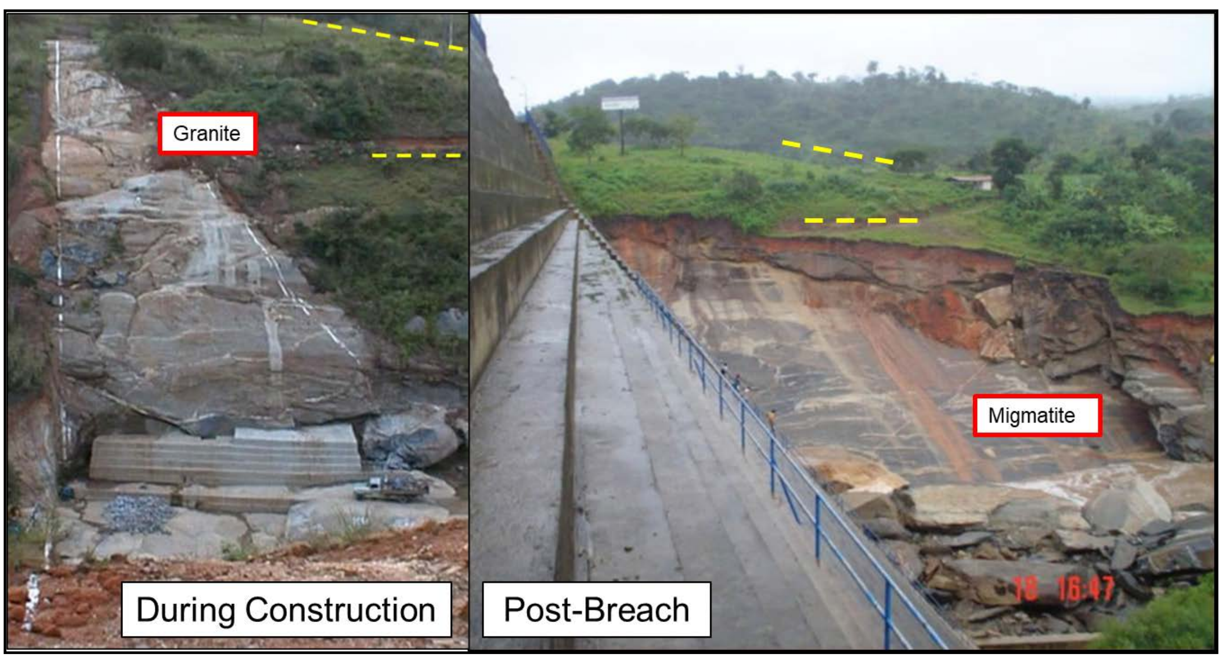


Figure 2. The granite exposed along the left abutment appeared to be relatively intact and fracture free (left photo). the underlying migmatite is exposed following the breach (right photo). Note the yellow lines for scale between photos. Left photo adapted from [2]. Right photo by Silas Porto (dated 18 June 2004), adapted for this paper.

 

## III. PREVAILING THEORY OF FAILURE

Of the references available to date, the report by Kanji [1] provides the most complete description of the failure process. Although the present authors offer a slightly different interpretation of the same basic facts (see below), the description provided by Dr. Kanji is well thought-out and insightful. It can be summarized as follows:

1. As the reservoir fills, seepage begins to occur along the base of the large granite surface slab.
2. As seepage intensifies, the residual soils at the base of the slab begin to move into the foundation drains; some of the drains become plugged while others reach their flow capacity.
3. As a result of increasing uplift pressures (due to plugged and capacity-limited drains), as well as the low shear strength of the residual soil, the dam-foundation system becomes marginally unstable.
4. Once the critical reservoir elevation has been exceeded, the granite slab begins to slide out and break up (first loud bang), with the upstream portion still remaining attached to the concrete.
5. With the downstream portion of the seepage path effectively removed, seepage gradients beneath the remaining portion of the granite slab increase, resulting in concentrated flows and the internal erosion of the underlying soils.
6. Having been undermined by the loss of the residual soil, the upstream slab collapses into the void (second loud bang), taking some of the concrete with it.
7. As reservoir water begins to flow through the opening, additional concrete and rock is scoured out, resulting in an enlarged opening and the uncontrolled release of the reservoir.

The failure process described above is fairly complex, and takes into account all of the key observations made prior to the breach. However, secondary reference documents have tended to highlight specific aspects of the process. For example, the case-history brief in FERC RIDM Chapter R5 [7] states that “a hole was piped beneath the dam”, suggesting that this was essentially an internal erosion failure. Other authors [8] have described the failure as a case of foundation wedge instability, putting it in the same category of events as the Malpasset disaster (the classic example of a foundation wedge failure). These interpretations would seem to be diametrically opposed, but the real problem hinges on what they have in common: each is focused solely on the foundation. In this framing, the only available RCC dam case history becomes a generic lesson in foundation engineering, with the concrete dam (and the RCC concept in general) being largely exonerated.

## IV. ALTERNATIVE CONCEPTUAL MODEL

As noted, the Kanji model provides a complete and evidence-based conceptualization of the failure process. However, there are three specific areas in which improvements could be made. The first concerns the large granite surface slab, which Kanji himself describes as unusually free of fractures [1]. This quality, which he credits with the absence of downstream pressure relief, would presumably also have made the slab stronger in tension. As such, it is not clear why the slab would break up following incremental movement (leaving the upstream portion intact) rather than stay in one piece before the concrete that adhered to it (which was likely much weaker in tension) failed. The second critique is that the final phase of the internal erosion process, which results in the undermining and collapse of the upstream slab, has to take place between the first and second “bangs”. Although the timing of these observations is not clear, presumably they would have been separated by no more than an hour given that the dam failed early in the evening [3] but would have been visible in the late afternoon (obviating the need for audible cues). This would be an extremely short period of time for the internal erosion of a clayey residual soil to develop and progress over a distance of roughly one hundred feet. The third critique has less to do with the conceptual model developed by Dr. Kanji than with how it has been interpreted by others. The prevailing theory of failure lacks a central unifying element, which makes it easy to focus on specific details while missing the overall point: that this was a concrete dam failure involving the catastrophic breach of an RCC structure.

The authors would like to propose an alternative conceptual model of what happened at Camará, one that is based on essentially the same pre-failure observations (but a slightly different interpretation thereof). With respect to the prevailing theory outline of the previous section, the primary differences are in steps 4 and 5. However, the entire failure sequence has been rewritten to provide additional detail (the physical observations associated with the various steps of the process are shown in italicized text below). An important premise of the alternative model is that even before high uplift pressures began to develop, the stresses within the concrete above the foundation contact were close to the minimum strengths, such that the added force of the slab pushing up was sufficient to produce cracking. While the authors of this paper are not structural engineers, and have not delved into the post-failure FEM analyses [2], this would appear plausible based on the inexperience of the prime contractor with RCC construction (the dam was originally specified as an embankment [1]), the presence of the drainage gallery (stress concentrations), and the relatively low compressive strength, compared to a conventional concrete, of the RCC mix used at Camará (only about 1000 psi, with an "allowable" value of half that [2]).

1. As the reservoir fills, the single-line grout curtain is defeated and seepage begins to occur along the base of the large granite surface slab.
2. Uplift forces begin to develop along the base of the slab, which begins to push up on the dam, resulting in some initial cracking (*first evidence of seepage in the left abutment gallery*).
3. As the reservoir rises above El. 435, seepage intensifies and the residual soils at the base of the slab begin to be eroded into the foundation drains (*direct observation*) as well as toward additional exits downstream of the dam.
4. As the residual soils continue to be eroded, some of the foundation drains become plugged (*direct observation*), and the capacity of the remaining drains is soon exceeded (*observation of high flows*), resulting in a further increase in uplift pressures along the base of the granite slab.
5. Once the residual soils have been essentially removed both upstream and downstream of the foundation drains, reservoir-like pressures begin to develop along the base of the slab, which begins to heave, creating new downstream seepage paths (*observations of increasing seepage along the left groin*) and resulting in additional cracking (*observations of new wet spots in gallery*).
6. As the reservoir reaches the critical elevation for stability, the uplifted portion of the granite slab abruptly rotates or twists into the space below it, resulting in the tensile failure of the already-weakened overlying concrete (*first "bang"*) and concentrated flows through the left abutment.
7. As a result of the surging water pressures, the detached sliver of concrete and rock is swept out, breaking up into smaller pieces as it moves downstream (*second "bang", post-failure photos*).
8. As reservoir water begins to flow through the opening, additional concrete and rock is scoured out, resulting in an enlarged opening and the uncontrolled release of the reservoir (*post-failure photos*).

In this conceptualization, the large granite surface slab does not begin to break up until it is being floated downstream, which is more plausible than it breaking up under the kind of small-strain conditions that would allow the upstream portion to remain intact and in place. Furthermore, the internal erosion process that results in the slab being completely undermined has months to develop (perhaps episodically), which would be more typical of what has been observed from case histories.[1] With respect to the central unifying element, it is neither internal erosion (to call this an internal erosion failure is a stretch, though the process did play a role) nor wedge instability (the fact that the dam remained largely intact after the removal of the slab suggests that the stability of the foundation was being compromised by something other than the resultant force of the dam). Rather, what brings all the elements of the alternative conceptual model together is the concept of uplift, to which gravity dams (and concrete dams in general) have historically been susceptible. If the concrete had not cracked as a result of pressures building up below the base of the granite slab, the dam may not have failed.

Based on the available evidence, the authors of this paper would propose that the Camará event be categorized as an *uplift failure*. This category would include scenarios that involve a structure being floated off its foundation, of which there are numerous examples [10], as well those in which uplift-related stresses internal to the concrete precipitate the breach (see Section IX, below, for another example). In both types of scenarios, it is the response of the dam, rather than the properties of the foundation, that determine the final outcome. The foundation conditions at Camará were certainly important (and incorrectly interpreted), and it could be argued that the same dam would have survived under a different set of foundation conditions. However, it could just as easily be argued that a different dam would have survived under the same set of foundation conditions. For example, with larger diameter drains that were capable of meeting the flow demand (and not susceptible to plugging), the pressures under the slab could potentially have been controlled. Similarly, a modern rockfill embankment would likely have been capable of expanding into the opening created by the settlement of the slab while maintaining a high degree of vertical stress-transfer, increasing the shear resistance of the slab to the point that it was prevented from sliding out.

[1] Even for the proverbially rapid Teton Dam failure, which involved highly erodible embankment soils, it took days or weeks for the internal erosion to develop and progress [9].

## V. ESTIMATED FAILURE PROBABILITY (PRE-FAILURE STATE OF KNOWLEDGE)

Risk analysis is a tool used by dam safety practitioners to help prioritize action within an inventory of dams [11]. It is not used to predict whether a dam will fail, but rather to identify and frame the key risk factors in a manner that highlights their potential significance. For a new dam, risk analyses would typically be performed during the design stage (with changes made to the design as appropriate) and after the completion of construction (in order to evaluate the impact of observed conditions). In general, the risks associated with a new dam would be expected to fall comfortably below any applicable threshold or guideline values [12].

According to Kanji [1], the legal responsibility for the safety of Camará Dam lay with the owner (inferred, based on other references, to be the State of Paraíba or some corporation thereof [2]-[3]). It is not known what kind of design standards the owner was required to meet, or whether deterministic performance criteria were applied. The use of dam safety risk analysis was certainly not as widespread in 2004 as it is today, and there is no evidence that the owner had either the technical capability to perform a dam safety risk analysis or that they operated in a regulatory framework that required or encouraged such a process. Whether a risk analysis performed after the construction of Camará Dam would have been beneficial is thus not a relevant question. Rather, the purpose of the next three sections is to explore the question of whether an agency with a mature dam safety program, if confronted with similar circumstances at the present time, would take action to reduce the probability of failure based on the results of a risk assessment. Since the present authors are affiliated with the Bureau of Reclamation, and well acquainted with the methodologies used by that agency, the premise of the exercise will be that Reclamation has just constructed a dam at a similar site.

To apply the process using the post-failure state of knowledge would be somewhat pointless, since the failure mode would be known and the probability of each component event logically equal to 1.0. Instead, it will be assumed that the risk analysis is being performed about a year and a half after the completion of the dam (either for the purpose of verifying the results of an earlier design-stage risk analysis or in response to unusual performance). With respect to the timeline provided in the Introduction, this would be somewhere in the February 2004 timeframe (when the first evidence of drain plugging and high flows was becoming available). By then, as-built documentation would have been organized to the extent that it could be used in a risk analysis, and there would have been enough of a record of performance observations to both develop a conceptual model and estimate the probability of failure.

The first and most important step of a risk analysis is to develop, for each failure process of concern, a common narrative [11]. This is done by discussing, thinking through, and developing a detailed description known as a potential failure mode (PFM). The PFM must capture the key steps of the process and provide a causal link between the normal and failed conditions of the structure. In this case, the hypothetical risk team can be thought of as consisting of several structural engineers (including the facilitator), an engineering geologist, and a geotechnical engineer. Based on personal experience, knowledge of previous case histories, and overall understanding of the project, the risk team comes up with the following description for the critical uplift PFM (see Figure 3 for sketch):

*As the reservoir fills, the single-line grout curtain is defeated, and seepage begins to occur along the base of the large granite slab overlying the left abutment shear zone. Due to the size of the slab and the absence of fractures open enough to allow drainage near its downstream end, higher than anticipated uplift pressures develop, resulting in an upward force being imparted to the dam and in the formation of cracks along the drainage gallery. Although the drains are initially effective in controlling the uplift pressure, they eventually become plugged with soils eroded from the shear zone. As the reservoir continues to fill, hydraulic jacking of the smooth, low-friction, slope-parallel, granite slab occurs to the extent that it is prevented from sliding downstream by the resistance of the dam concrete alone. As the tensile and shear stresses begin to exceed the strength of the concrete, it cracks through between the base of the dam and the gallery, resulting in concentrated flows through the area. Intervention fails, the damaged concrete and rock along the left abutment contact are progressively washed out, and an uncontrolled release of the reservoir occurs.*

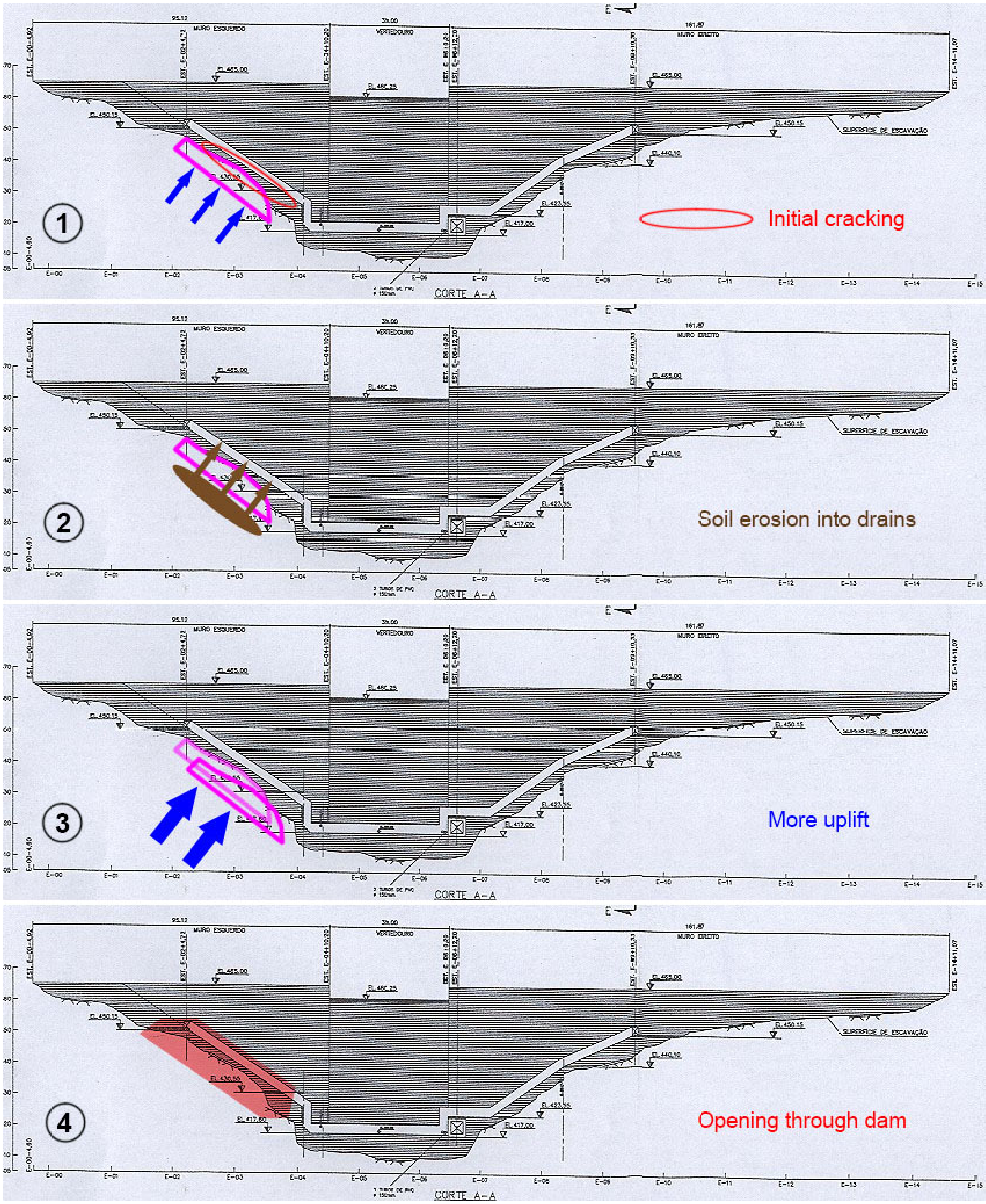


Figure 3. Conceptual sketch of the failure process. Adapted from [2].

The next step would be to discretize the PFM into a series of steps or *events* for which the conditional probabilities can be reasonably estimated (the appropriate number events will vary depending on the experience of the team and the type of PFM being considered). Note that while the events should collectively provide an equivalent causal link between the normal and failed conditions, their titles need not capture in verbatim the wording of the associated PFM description. The PFM was decomposed into the following seven events:

1. Critical reservoir elevation is exceeded (taken as a given based on the first fill conditions)
2. Seepage begins to occur along the base of the large granite surface slab
3. Internal erosion of the shear zone materials at the base of the slab initiates by scour
4. The capacity of the foundation drain system is exceeded
5. Increasing uplift pressures result in the hydraulic jacking or lifting of the granite slab
6. The strength of the concrete is exceeded by the downstream pull of the granite slab, resulting in its separation and the formation of a series of voids along the left abutment contact
7. Concentrated flows result in the enlargement of the voids and an uncontrolled release of the reservoir

The conditional probability estimates selected by the team, along with the key factors upon which they are based, are given below (conditional probability distributions would be used to help characterize the uncertainty, but only the average or best-estimate values are shown below). For each event, the factors supporting a higher probability estimate are marked as "more likely", and those supporting a lower estimate as "less likely". Note that the conditional probability estimates are developed under the assumption that the preceding events have already occurred (e.g., the probability of the drain capacity being exceeded may not be equal to 1.0, but the subsequent estimates do assume that the drain capacity has been exceeded).

2. Seepage has been observed downstream (more likely); High flows have been observed in the foundation drains, which extend to at least the base of the granite slab (more likely); Single-line grout curtains are often ineffective (more likely); The observed seepage could be coming from somewhere other than the base of the slab, e.g., from an even deeper set of fractures with less impact on dam stability (less likely). **Best estimate probability = 0.8**.
3. At the present reservoir elevation, there is about 30 m of differential head, which could result in localized seepage velocities high enough to initiate internal erosion (more likely); Sediments have been observed in the galleries (more likely); The shear zone soils have some plasticity, which could increase their erosion resistance (less likely); The shear zone soils could be fairly compact as a result of being compressed by the granite slab (less likely). **Best estimate probability = 0.5**.
4. Some of the foundation drains that previously operated have stopped working, likely as a result of plugging (more likely); The system was designed under the assumption that the grout curtain would be relatively effective (more likely); Water is spouting from the remaining foundation drains, suggesting that their sizing and number may not be adequate for the amount of seepage that is occurring (more likely); The fact that flows continue to increase at some drains suggests that their capacity has not yet been exceeded (less likely). **Best estimate probability = 0.8**.
5. The granite slab is unusually large and fracture free, with no obvious way of dissipating pressures other than via the foundation drains, which are at this point ineffective (more likely); The jacking mechanism has been observed in spillways, where only a fraction of the surface flow needs to get below the concrete to lift it out (more likely); If the slab is really functioning as one large unit, then it would tend to have greater inertia and greater resistance to jacking (less likely); There could be other unknown seepage exits that are helping to relieve uplift pressures (less likely). **Best estimate probability = 0.5**.
6. The concrete is already cracked, and new cracking continues to appear (more likely); The drainage gallery could be further concentrating stresses at a critical location (more likely); There is no evidence that the existing cracking is continuous upstream to downstream (less likely); Like conventional concrete, RCC has non-negligible tensile strength (less likely); Although large, the granite slab is much smaller than the dam, and the forces imparted by it may not change the internal stress distribution in a truly catastrophic manner (less likely). **Best estimate probability = 0.1**.
7. RCC is somewhat more erodible than conventional concrete (more likely); The differential head at the initial opening would be at least 30 m, which could result in high flow velocities (more likely); Based on the irregular shape of the initial opening, flows would likely be turbulent, which could increase their potential to strip away rock and concrete (more likely); Since the left abutment slab and underlying soil

materials have already been removed, and since concrete is generally considered erosion resistant, it is plausible that both the dam and foundation could withstand additional damage until the reservoir elevation drops, resulting in a self-limiting "controlled" release that does not exceed the safe channel capacity (less likely). **Best estimate probability = 0.5**.

In accordance with the intersection formula of elementary probability theory [11], the annualized failure probability (AFP) was calculated by multiplying the above conditional probability estimates by the load probability (taken as 1.0). The **estimated AFP of 8.0E-3**, or roughly one in a hundred, is compared to Reclamation's public protection guidelines in Section VII (below), which outlines the case for dam safety action. For the purposes of this section, two things should be noted when considering the estimated AFP. The first is that if the dam ended up failing four months later (e.g., because action was not taken in time), that would not imply that the estimated AFP had been "wrong" as a result of not being closer to 1.0. The AFP best-estimate developed here is based on an interpretation of the information that would have been available circa February 2004, and not on the complete post-failure state of knowledge (hence the apparent inconsistencies between the PFM description and the "alternative conceptual model" of Section IV). The second is that the *Internal erosion initiates* event could be omitted from the PFM with little change to the subsequent conditional probability estimates or the overall AFP. Under such a formulation, it would still be possible to consider internal erosion (e.g., as a more likely factor for the capacity of the drains being exceeded). However, the fact that it does not need to be explicitly considered as a part of a working conceptual model is consistent with the authors' view that this was not an internal erosion failure.

## VI. Estimated Life-Loss Consequences (Post-Failure State of Knowledge)

Prior to portraying the risks on a standard Reclamation fN chart (discussed in greater detail below), the life loss consequences of the breach flood would be estimated [13]. Reclamation's approach to estimating the consequences of dam failure, referred to as the Reclamation consequence estimating methodology (RCEM), is rooted in the empirical interpretation of dam failure and flood event case histories. Typically, the required background information, including breach inundation mapping and population at risk (PAR) estimates for various breach scenarios, would be prepared by consequence specialists. In order to apply the RCEM methodology to Camará Dam, the hypothetical risk team would have needed to have this documentation available to them before the risk analysis. However, since it likely does not exist, the authors have relied on post-failure information to estimate the required parameters.

The PAR consists of those individuals who would typically be present within the inundation area at the time of breach. The size of the inundation area is in turn a function of the size of the reservoir and the peak discharge during breach, In most available references, the reservoir volume is identified as 17 million cubic meters (14,000 acre-feet) [3], but this information is of little use without inundation mapping. However, the PAR can be very roughly estimated using the available information on the population impacted (but not necessarily harmed) by the flooding. According to a news article posted by Brazzil.com, the flooding following the breach of Camará Dam left just under 800 families homeless in the town of Alagoa Grande (which is the primary consequence center), located approximately 10 linear miles, or approximately 25 river miles, downstream of the dam [3]. For the estimation of PAR, the authors assumed an average household size of 4 people[2], and arrived at a PAR estimate of 3,200 people.

In the RCEM procedure, fatality rates are applied to the population at risk as a function of warning time and flood severity. To get a sense of what kind of warning would be provided, a standard RCEM analysis would consider numerous factors, such as time of day (or season) factors, flood wave arrival time, and information or assumptions for the warning dissemination (timing, effectiveness). Documentation and reporting from after the failure provide clues to how these parameters could have been estimated and characterized by a hypothetical risk team. From the Brazzil.com article, warning of the dam breach reached the residents of Alagoa Grande by radio and by telephone prior to the arrival of the flood wave. The flooding occurred in the evening, with fog present along the river. Thus, while adequate warning was likely issued to most of the residents of Alagoa Grande, its effectiveness is not clear. The available information does not provide an indication to the average time between downstream notification of the flooding and the arrival of the flood wave. Additionally, references mention that the warning may not have been well heeded due to resident resiliency to fluctuating river stage.

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[2] Some households may have been larger, but there are also examples of individuals living alone with their ageing parents [3]. It is also important to keep in mind that the PAR was concentrated in towns rather than in rural areas (where a larger average household size may apply).

 

The RCEM procedure continues with an estimation of the flood severity at the location of interest, which is defined quantitatively in terms of DV (product of flood depth and velocity). At a minimum, this step in the RCEM procedure requires an estimate of the peak discharge (Q) and the flood width (w) at the location of interest. To estimate the peak discharge at the location of Alagoa Grande, the authors applied the Froelich (1995) [14] equation to estimate the peak discharge at the location of Camará Dam, and applied an attenuation relationship borrowed from an unrelated inundation study that involved similar topography and distance to the PAR as for Camará Dam. While a well-defined scour channel downstream of the dam was evident in Google Earth imagery, the extent of the flooding was not as obvious in Alagoa Grande. To estimate the flood width, the authors relied on the information (combined with imagery showing the typical housing density) that approximately 250 homes were completely destroyed by the flooding [3].

While the assumptions and calculations for DV are not discussed in this paper, the flood severity analysis coupled with the warning time considerations prompted an RCEM "suggested" fatality rate range of 0.0005 to 0.02. Applied to the estimated PAR of approximately 3,200, the estimated life loss per RCEM ranged from 2 to 64 people. In reality, the actual life loss ranged from 5 (in most reports including [3]) to 20 (highest reported estimate [7]), within the estimated range. Although the life loss resulting from this event may seem "low" for a concrete dam failure (e.g., because conventional concrete is brittle and because many such breaches have been sudden), it is the authors' view that the numbers are consistent with the severity of the flooding that apparently occurred in Alagoa Grande. Additional factors, such as the availability of high ground adjacent to town and awareness of the potential for flooding along the Mamanguape River, likely contributed to the number of fatalities falling closer to the low end of the estimated range and aided in the survival of many residents.

## VII. Dam Safety Case for Action (Pre-Failure State of Knowledge)

The next step in the risk assessment process would be to combine the failure probability and life loss estimates into a single PFM data point on the standard Reclamation fN chart [12]. The chart is used to determine whether the AFP and Annualized Life Loss are plotting, with respect to the guideline values, in an area of increasing (or decreasing) justification to reduce or better understand the risks. The Annualized Life Loss is a normalized measure of life loss risk that takes into account the occurrence probability of the associated PFM; for an individual PFM with only one breach scenario, it would be calculated as the product of the AFP and estimated life loss. Under the current Reclamation public protection guidelines [12], the threshold value for Annualized Life Loss risk is 1/1000 (1E-3), and the threshold value for AFP is 1/10,000 (1E-4).

The fN chart risk portrayal corresponding to the failure probability and life loss estimates developed by the hypothetical risk team is shown in Figure 4. The total risk marker, which is associated directly with the uplift PFM (since no other PFMs are considered for this exercise), plots approximately two orders of magnitude above both the AFP and Annualized Life Loss guidelines. While an estimated AFP on the order of 1E-2 would in itself be a cause for concern (since the average dam failure probability *before* the passage of dam safety legislation in the 1970s was only about 1E-4 [12]), the risk estimates would not be used as the sole basis for any recommended actions. Instead, a case for action would have been built by bringing in additional arguments, such as:

- Both the design of the dam (which relies on the apparently ineffective grout curtain to prevent dangerous uplift forces from developing) and recent performance observations (increasing seepage and concrete cracking) suggest that there may be a failure in progress.
- Even with advanced warning, parts of the downstream population could be difficult to notify and evacuate, and some amount of life loss would almost certainly result from a failure.
- Based on the elevated plotting position of the total risk marker as well as the strength of the available evidence, there is little potential for the interpretation of risk to change given new information. In other words, additional investigations or analyses are not needed to support the conclusion that some kind of corrective action is needed.
- If action is not taken immediately, there may not be an opportunity to act later. Given the present time of year (February), rainfall will only intensify, and based on the limited capacity of the outlet works, it may not be possible to effectively draw down the reservoir a few months later.

In taking action to reduce the risk of failure, the logical first step would have been to lower the reservoir and keep it as low as possible. Other interim actions, such as 24-hour monitoring, would also have been considered, and conversations with emergency response staff would likely have been initiated. Once the situation at the dam had been stabilized (to the extent that this was possible under the projected inflows), long-term risk reduction measures would have been evaluated, and a permanent modification designed. Some of the modification alternatives that would potentially have been considered are discussed below.

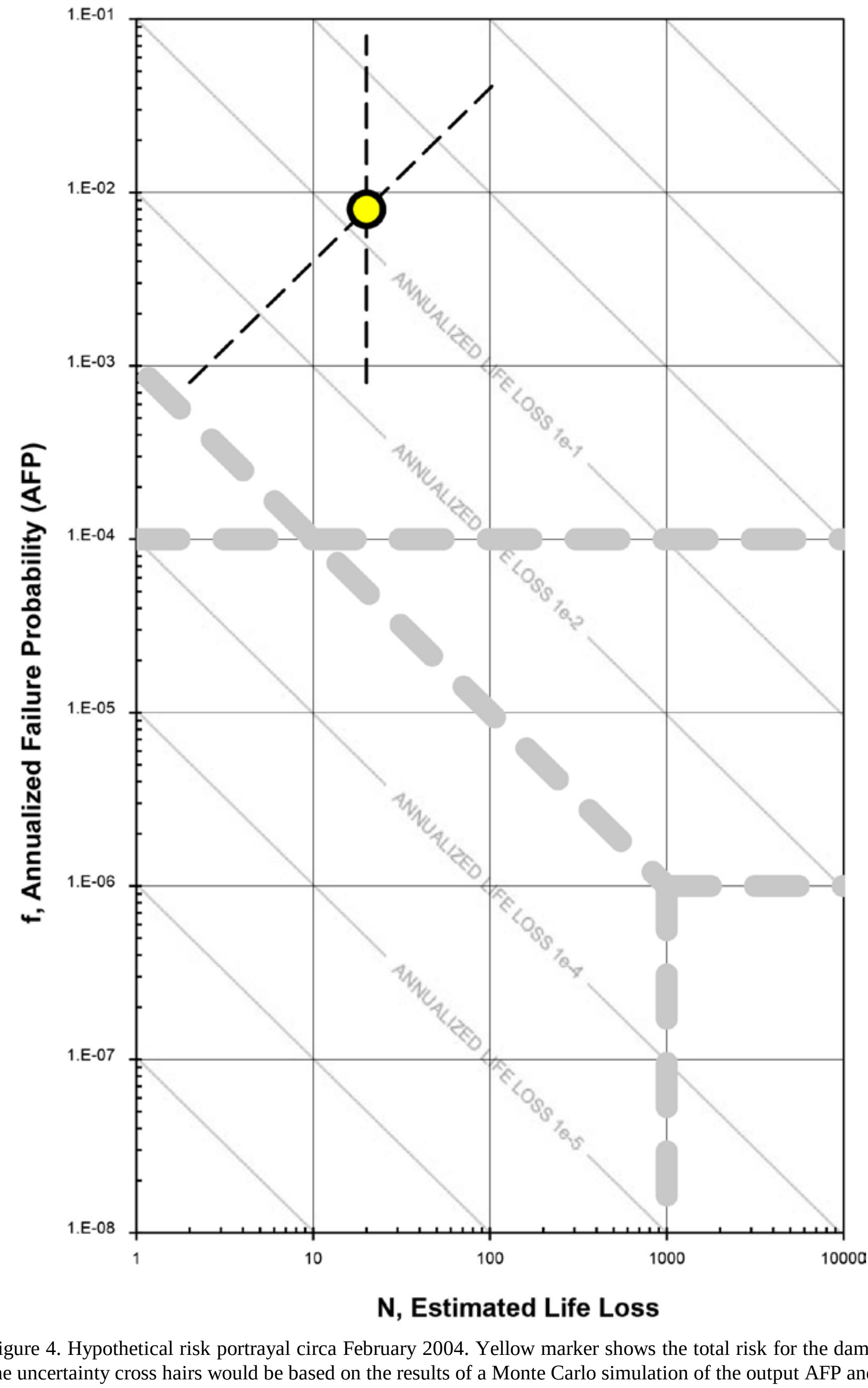


Figure 4. Hypothetical risk portrayal circa February 2004. Yellow marker shows the total risk for the dam (with respect to the PFM evaluated). The size of the uncertainty cross hairs would be based on the results of a Monte Carlo simulation of the output AFP and Annualized Life Loss distributions.

## VIII. Remedial Measures and Supplementary Data Collection (Pre-Failure State of Knowledge)

The following discussion is meant to highlight the actions that could be taken, both during and after initial construction, to address the types of problems experienced at Camará Dam. Characterizing potential remedial measures without being biased by the facts of the eventual failure is unrealistic, but the following modification alternatives could potentially have been considered by a team tasked with assessing the safety of Camará Dam, given specific pre-failure observations. For most of the remedial measures listed below, supplementary data collection would likely be required before selecting a preferred alternative (or combination of measures) or proceeding with design.

- Sub-surface treatment (such as chemical grouting) of the shear zone with installation of post-tensioned anchors to prevent displacement of the granite slab. This option assumes that the rock slab has been successfully identified by the team as a vulnerability. Depending on how the results of the pre-construction investigations are interpreted, this could be the case; however, it should be noted that the actual geometry of the shear zone was not inferred in any of the pre-failure investigations. The development of this remedial measure would likely require additional field exploration activities that would be formulated by projecting all reasonable orientations of foliations, fractures, and faults to identify and properly characterize the spatial extent and properties of rock discontinuities. Such activities would be required to both identify and characterize the flaw and inform the design of the anchor system.
- Installation of large diameter drains along the left abutment contact with improved instrumentation and an early warning system. This option would likely be readily available without the pursuit of additional field investigations and would likely be identified as feasible with the pre-failure information, given the evidence of seepage in the left abutment gallery and the observation of soils being eroded into the foundation drains, causing the drains to plug and no longer function as designed. In conjunction with the installation of large diameter foundation drains, additional instrumentation and monitoring would be identified to validate the effectiveness of the drains in alleviating uplift pressures. Instrumentation and monitoring would be established with thresholds for performance, so action could be taken if an identified performance parameter is exceeded.
- Construction of a post-tensioned gravity overlay with additional drainage features. Similar to the improved drainage, this option would likely be identified with the pre-failure information as a plausible means of counteracting the excessive uplift. This remedial measure would likely require investigations and additional instrumentation to characterize the uplift pressure force acting on the dam and the foundation properties related to anchor design. This would include additional field exploration activities that would be formulated by projecting all reasonable orientations of foliations, fractures, and faults to identify and properly characterize the spatial extent and properties of rock discontinuities. These investigations would be especially relevant to the portion of the slab where uplift was thought to be promoting instability.
- Partial dam replacement with improved left abutment grout curtain or cutoff wall. Given the knowledge available to the team prior to failure, the team would likely have identified seepage and uplift as a concern. This alternative would be used to reduce the amount of seepage bypassing the existing single line grout curtain and limit the seepage available to erode the soils in the shear zone and generate uplift. Depending on the severity of the cracking observed by the time this measure was being looked at, partial dam replacement may also have been considered (and was actually adopted as the post-failure reconstruction method). The development of this remedial measure would require additional field exploration activities to properly characterize the spatial extent and properties of the rock discontinuities. If an expanded grout curtain or cutoff wall was pursued, redundancy and provisions would be required to address locations where voids below the slab may already exist.
- Long-term reservoir restriction. With the estimated risk of failure relatively high, a reservoir restriction could have been established for a reservoir elevation that coincided with improved performance or based on other considerations. This option could also have been pursued as an interim risk reduction measure, while the team evaluated the need for additional subsurface data, instrumentation and monitoring, and considered other long-term alternatives. If the cost of the technically viable design alternatives exceeded the overall benefit of the project, a long-term reservoir restriction would have become a competitive option.

As noted in the bullet list above, structural remedial measures would potentially require a significant amount of additional information depending on the quality and extent of the pre-construction documentation (which is unknown to the authors of this paper) and on how it was interpreted. As engineers and subsurface scientists, we can collectively understand the reality that if all structures were designed to the worst-case geological conditions, investigation costs would be prohibitively high. This is recognized in the observational method for geotechnics, proposed by Terzaghi and documented by Peck in 1969 [15]. This method proposes that as new information becomes available, such as with new investigations or the uncovering of non-represented materials, the design and construction schedule accommodate a change to address the deviation. This type of flexibility is especially important given that the foundation is the area where even the most adverse scenario can still be a conceivable scenario.

## IX. CASE HISTORY PRECEDENTS

It is the view of the authors that a knowledge of potential failure modes and relevant dam safety case histories is critical for the development of subsurface data for risk assessments. As Ralph Peck stated, "We would do well to recall and examine the attributes necessary for the successful practice of subsurface engineering. There are at least three: knowledge of precedents, familiarity with soil mechanics, and a working knowledge of geology. Of these, familiarity with precedents is by far the most important." [16] When thinking about Camará Dam, the case history of St. Francis Dam stands out as particularly relevant. References to St. Francis Dam in this section are sourced from Dr. David Rogers' paper "The 1928 St. Francis Dam Failure and Its Impact on American Civil Engineering" [17].

The geology of both dams shares certain similarities. In both cases, the left abutment contained a similarly dipping metamorphic rock where the ground surface topography roughly followed the dip of the rock foliation. In the case of St. Francis, the materials overlying the dipping metamorphic rock were comprised of an ancient landslide. This differs from the materials overlying the dipping metamorphic rock at Camará Dam; however, the behavior of the overlying materials, with the development of high uplift pressures, is somewhat similar. At St. Francis, the failure to recognize the ancient landslide structure in the left abutment is identified as a key design deficiency that led to failure of the dam [17]. The landslide was incorporated into the schist of the left abutment, and roughly followed the existing dip of the foliations in the left abutment rock. At Camará, the left abutment featured metamorphic rock (migmatite) overlain by a heavily altered metagranite. The heavily altered metagranite overlying the migmatite may have allowed seepage to enter the abutment and initiate uplift similar to that of the ancient landslide at St. Francis, given the potential for development of preferred seepage paths through both types of materials.

A few key similarities also exist in the design and construction of Camará and St. Francis Dams. Whereas the arch of St. Francis Dam was constructed without any transverse joints, Camará Dam's RCC construction similarly lacked engineered joints in the concrete. Without control joints, hydrostatic pressures developing in the abutment led to the cracking of the structures at the weakest locations. St. Francis completely lacked a cement grout curtain in the foundation, where Camará was constructed with a single line grout curtain that lacked closure, as evidenced by the amount of shallow seepage through the left abutment. In the case of the drainage features provided, St. Francis lacked uplift relief wells on the sloping abutment sections of the dam, compared to potentially undersized foundation drains on the sloping abutments at Camará (as evidenced by the working drains flowing at their capacity prior to failure). In these comparisons, the design deficiencies of St. Francis are reflected in the ineffectiveness of some of the corresponding design features at Camará Dam.

As the failure of each dam became inevitable, cracking due to initial uplift permitted the full hydrostatic pressure of the reservoir to enter the cracks and begin to destabilize the concrete structure. In both structures, the initial uplift and cracking of the concrete resulted in increased seepage through the abutments, though the mechanisms vary somewhat. In the case of St. Francis, strong seepage flows began to scour and undercut the schist at the base of the abutment; at Camará, seepage intensification resulted in the residual soils at the base of the slab eroding into the foundation drains (as well as toward additional exits downstream of the dam). As the residual soils eroded, some of the foundation drains were plugged and the capacity of the remaining drains was soon exceeded. At St. Francis, the increase in seepage and resultant scour of the schist allowed for mobilization of the ancient landslide, which moved into the valley taking a section of the concrete dam with it. At Camará, with the removal of residual soils, uplift pressures began to increase catastrophically, and the overlying granite slab began to heave, creating new downstream seepage paths and additional cracking. As the reservoir reached the critical elevation for stability, the uplifted portion of the slab likely rotated abruptly or twisted into the void, resulting in the tensile failure of the already-weakened overlying concrete and concentrated flows through the left abutment.

As noted above, dam sites are often assessed based on the most probable conditions, and foundation geology is often the key condition with the potential for unfavorable deviations from those conditions. As such, a thorough understanding of design precedents and case histories allows one to use their imagination and hypothesize the *reasonably possible*[3] conditions that could adversely affect dam safety. Dr. David Rogers provides the following as a lesson learned from St. Francis, which echoes the authors' emphasis on the importance of understanding and learning from case histories:

> We will not identify those geologic features or structures which we are not specifically looking for. We have to have in mind what we are seeking, realizing that we will seldom be able to recognize those features with which we've had little prior experience [17].

## X. Closing Remarks

In the case of Camará Dam, the geologic feature that would lead to the failure of the structure was not directly observable until after the failure had occurred. However, had the geologic investigations projected all reasonable orientations of foliations, fractures, and faults to identify and properly characterize the spatial extent and properties of the rock discontinuities, the critical shear zone may have been more completely characterized. Without a complete understanding of the abutment geology and of the susceptibility of the chosen design to those conditions, remedial measures during construction were unlikely to be effective. Not unexpectedly, the failure was a direct result of uncontrolled uplift, a mechanism that has led to many other concrete dam failures. In the weeks prior to breach, the signs of a failure in progress had become apparent to the extent that immediate action to save the dam was recommended to the owner. However, by then it was raining and the pool was rising, and it is not clear if an eleventh-hour drawdown would have been effective or possible.

Risk analysis is a process that combines performance observations, analysis results, and design history with engineering judgment in an effort to better understand the key sources of risk at a dam. The process relies on the availability of subject matter experts, but it does not take a degree in statistics to understand how it works or to be an informed participant. A risk analysis performed in early 2004 would likely have provided enough time for deliberation and action and would have created the necessary platform for implementing remedial measures, including interim action, to reduce the risk of failure. With the PFM correctly identified, there likely would have been several viable structural risk-reduction options. Each of these options would likely have prevented the loss of life and would have been less expensive than the post-failure reconstruction that occurred. In general, an appreciation of potential failure modes and dam safety precedents is critical for the development of subsurface data for risk assessments. The use of case histories, such as that of St. Francis Dam, might have enabled the designers of Camará to recognize that RCC structures are not infallible, and to envision the "reasonably possible" conditions that could affect the safety of their design.

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