

**FERC Engineering Guidelines
Risk-Informed Decision Making**

Chapter R5

**Concrete Dam Analysis
Arch, Gravity and Buttress Dams**

Chapter R5

Concrete Dam Analysis - Arch, Gravity and Buttress Dams

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Chapter R5 – Concrete Dams

R5.1 Introduction

This chapter describes failure modes that are of concern for concrete dams and some analysis techniques for the evaluation of the probability of their occurrence.

The failure modes and analysis techniques discussed here are not exhaustive. This chapter should not be viewed as prescribing a specific set of failure modes to be analyzed. It is the responsibility of the owner, in collaboration with the Part 12 independent consultant and the FERC to identify, and evaluate the probability of, failure modes that are appropriate to a given dam.

R5.2 Definitions

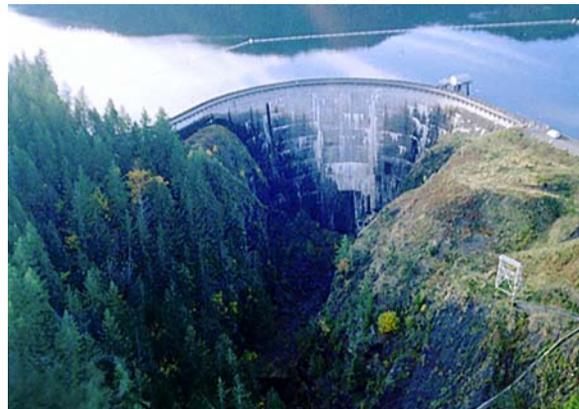
For the purposes of this chapter, concrete dams are divided into 3 categories:

1) **Gravity Dams:** Dams that resist the reservoir loading by frictional resistance with their foundations. Concrete stresses tend to be low. Extreme overtopping of these dams is typically the greatest threat to stability because of higher reservoir driving forces and increased uplift pressures on the dam base.



Gravity Dam

2) **Arch dams:** These Dams transfer load to abutments. The strength of the abutment rock is critical to their stability. Stresses in these dams tend to be moderate. Overtopping can be an issue for arch dams if erosion of the abutment rock results.



Arch Dam

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3) Slab & Buttress/Buttress Arch Dams:

These dams have a series of relatively thin slabs or arches which slope upward going downstream. Typically the angle of slope is about 45° . This slope causes the reservoir to produce a downward force approximately equal to the downstream driving force, stabilizing the dam. The slabs or arches are supported by a series of buttresses. Stresses can be high in these dams. They are susceptible to earthquake induced failure, especially from ground motions in the cross valley direction. They are also susceptible to local foundation weakness because all it takes is for one buttress to be poorly founded for failure to result. Freeze thaw damage often causes deterioration of concrete if the dam is in a cold weather environment.



Buttress Arch Dam

While these three general categories will be covered, it is important to realize that there can be mixed modes of force transfer. For example, a gravity dam can be curved in plan so that there is some force transfer to the abutments. An arch dam can bear on a thrust block, which will be governed by gravity type failure modes.

Other Definitions:

Failure: Failure is either the uncontrolled release of the reservoir, or when the dam is compromised to the point where there is no confidence in its ability to continue to function. For example, in case history R5.4.1.1 Austin Dam, the dam initially moved downstream 18" at its base and 30" inches at the crest without releasing the reservoir. While the reservoir was not released, the FERC would consider this failure. Morris Sheppard (R5.6.1.1) did not release the reservoir but was in a state of failure. There was no reason to believe that the downstream sliding on the weak shale layer in the foundation would not progress until release of the reservoir.

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Risk analysis does not reverse the burden of proof with respect to the failure of the dam. It is not up to the FERC to prove that failure in a given situation is certain, rather it is up to the licensee to prove that the probability of failure is acceptably low.

Failure Mechanism: A failure mechanism is the actual physical process required for failure; for example: monolith movement, a specific critical structural member that has to rupture, buckle, etc. For concrete dams and appurtenant structures, failure mechanisms will involve at least one of the following:

Mechanism	Case History
Overstressing	Koyna (R5.4.1.5)
Overturning	Austin (R5.4.1.1)
Sliding	Morris Sheppard (R5.6.1.1)
Erosion (External or Internal)	Camara (R5.4.1.4)
Buckling	Plum (R5.5.1.2)

In many failures, there are a combination of mechanisms involved. For example, Austin overturned and slid. The internal erosion in the abutment at Camara resulted in a rock block sliding. The buckling of Plum resulted in overstressing.

Failure Mode: A failure mode is the complete chain of events that lead to the failure of the dam. It may include concrete cracking and sliding, electrical/mechanical failures, human error, etc.

R5.3 Material Properties

Failure modes are almost always contingent upon material properties. The strength of concrete in compression, tension and shear, the friction angle of a sliding plane in rock or concrete, etc. There will always be some variation of material properties. This produces uncertainty that risk analysis is well suited to deal with. The sensitivity of failure probability to material properties is a function of the failure mechanism.

Variability of material properties invites statistical analysis. It's almost second nature for engineers to want to find a μ and a σ for a set of cylinder breaks or direct shear tests, etc. It is very important however to keep the uncertainty caused by material variation in perspective. Random variation of material properties is not as significant as discontinuous features when it comes to the probability of failure. For example, in the stability analysis of a gravity dam, the random variation of unit weight and friction angle

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has little effect on the probability of a failure mode, however a discontinuity such as a weak planar joint underneath the dam may have a huge effect.

All historic failures of concrete dams have been the result of a critical flaw in the dam or foundation that was not considered, or a loading condition that was not anticipated. Such errors cannot be corrected by statistical analysis.

Still it is important to properly incorporate the variation of material properties into a risk analysis. This section will address the different forms of uncertainty introduced by material property variation.

R5.3.1 Parallel Failure Mechanisms

Consider for example the case depicted in Fig. R5.3.1 below in which the failure mode of frictional sliding is being considered. There will always be some random variation of the coefficient of friction, and what has historically been done is to downgrade $Tan(\phi)$ to account for this variation, insuring that the shear resistance of the failure plane is at least as great as what is calculated. This technique will necessarily under estimate the strength of the failure plane and result in an overestimate of failure probability.

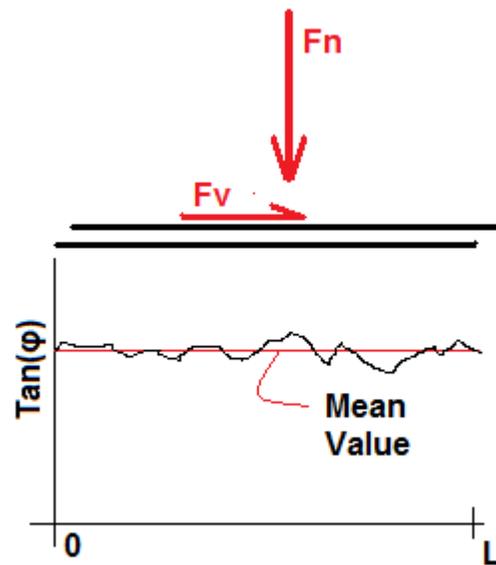


Fig. R5.3.1

The true shear strength of the failure plane is given by:

$$\int_0^L \sigma_N Tan(\phi) dx = \frac{F_n}{L} \int_0^L Tan(\phi) dx = F_n * \overline{Tan(\phi)}$$

Integrating the shear resistance over the failure plane results in F_n times the mean value of $Tan(\phi)$. This phenomenon occurs because the failure mode being considered is a parallel failure mode, one in which weaker shear strengths are offset by stronger ones. In such cases, the variation of the parameter becomes irrelevant and the total system strength is contingent upon the mean value only.

R5.3.2 Brittle Material Behavior.

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Concrete is a brittle material in tension, and since the ultimate shear strength is typically controlled by principal axis tension, it is brittle in shear also. In reinforced concrete, the brittle behavior of concrete in tension is cancelled by the ductility of steel reinforcement. In dams however, we are often dealing with unreinforced or very lightly reinforced concrete. In such cases, crack propagation is governed by fracture mechanics, a sub discipline of regular continuum mechanics that deals specifically with the stress state in the vicinity of a crack tip.

The closed form solution for an elliptical hole in an elastic plate subject to uniaxial tension is helpful in understanding the stress at a crack tip. (See Fig R5.3.2). The increase in stress at the perimeter of the hole is given by:

$$\sigma_{MAX} = \sigma * \left(1 + 2 * \left(\frac{A}{B} \right) \right)$$

Where A is the ellipse axis perpendicular to the stress field and B is the ellipse axis parallel to the stress field.

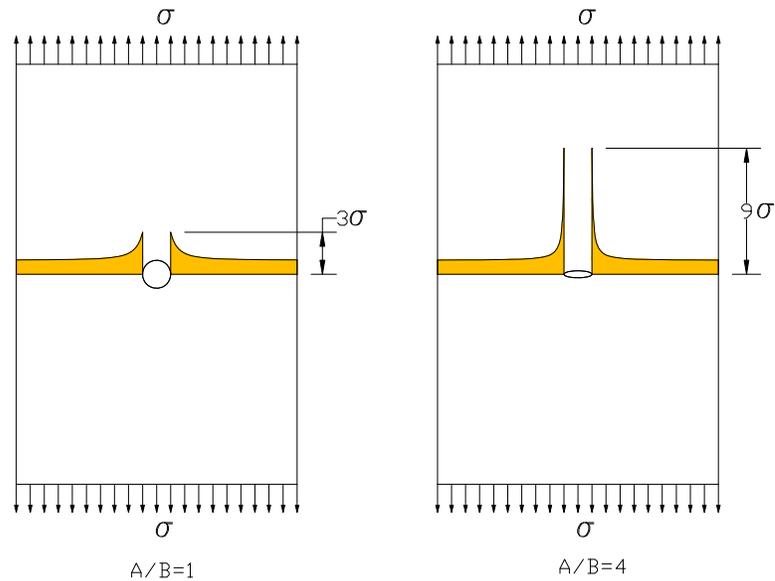


Fig. R5.3.2

As can be seen, as the A/B ratio becomes large, the ellipse begins to resemble a crack. At large A/B ratios, the stress amplification becomes enormous. In elasto plastic materials such as mild steel, a zone of plasticity forms around the crack tip and the crack propagates only with difficulty. In brittle materials however, the crack propagates. If the tension field is kept constant, crack propagation progresses at sonic velocities. Because the stress amplification at the crack tip is so great, the crack can continue to propagate through areas where the tension field is significantly less than the tension required to initiate cracking. The crack will propagate not only to where the tension field is less than the tensile strength of the concrete, but it will continue until it gets to a region of no tension. For this reason, once a crack begins to propagate, the tensile strength of concrete becomes irrelevant.

A crack in a shear field behaves in a similar manor to a crack in a tension field because the crack is propagated by principal axis tension. The direction of propagation is altered

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however as the crack attempts to turn to align itself perpendicular to the major principal axis.

R5.3.3 Series Failure Mechanisms.

Now consider the case depicted in Fig. R5.1.3.3. Here the failure mechanism is shear failure of an unreinforced or under reinforced corbel supporting the upstream slab of a slab and buttress dam. Note that the upstream corner of the corbel is highly stressed in tension. If tension exceeds the concrete strength, a crack will initiate and begin to propagate in a path approximately perpendicular to the axis of principal tension. While this is considered a shear failure mechanism, the controlling parameter will be tensile strength.

Unlike the parallel failure mode described in R5.3.1, there is no way for weak areas to transfer load to stronger areas. This is a series failure mode. What controls the failure mechanism is the weakest link, not the average strength. If tensile strength variation allows for the possibility of the concrete tensile strength to be exceeded, the probability of failure can become very high.

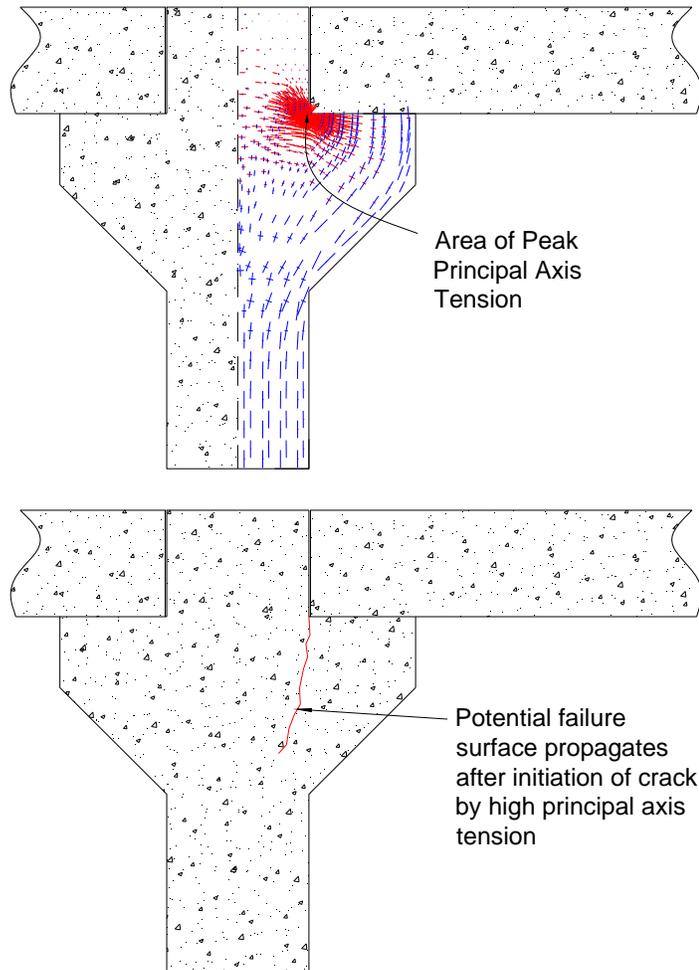


Fig. R5.3.3

R5.3.4 Obtaining Data for Analysis

We are typically faced with the situation where we have no more than a handful of tests on which to base a selection of a material parameter. There is scatter in the data that we do have. Suppose for example you have 6 core samples that cross a potential failure

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plane, $\text{Tan}(\phi)$ was obtained for each of them using direct shear testing. Suppose further that the tested values are as shown in the table below.

Sample	$\text{Tan}(\phi)$
1	.72
2	.68
3	.7
4	.59
5	.63
6	.68

R5.3.4.1 Parallel Failure Mechanisms

If the failure mechanism is parallel, all that matters is the mean value, which in this case is .667. This value could be used in analysis, but there is a probability that the mean of the 6 samples is not the mean of the friction angle of the entire failure surface. The mean of the sample set could be an overestimate of the mean for the failure surface if the 6 samples just happened to have been taken from locations of higher strength.

If one assumes that $\text{Tan}(\phi)$ is normally distributed, the PDF of the mean of a sample set of size N is also normally distributed with a standard deviation as shown below.

$$\sigma_{mean} = \frac{\sigma}{\sqrt{N}}$$

The standard deviation of the total distribution can be approximated by the standard deviation of the sample set, which in the case of our 6 data points is 0.043843. Thus the standard deviation of mean of a sample set is:

$$\frac{.043843}{\sqrt{6}} = .0179$$

The PDF for the sample mean is then a normal distribution with a mean of .667 and a standard deviation of .0179. The probability that the real mean is

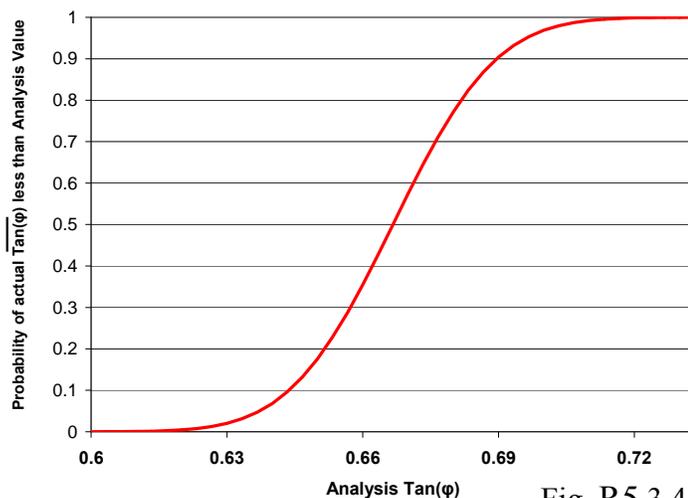


Fig. R5.3.4

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less than the sample mean is 0.5. The probability that the real mean is less than the sample mean $-\sigma_{\text{mean}}$ is .16, etc.

One can construct a cumulative distribution curve based on the above. (See Fig R5.3.4). Note that the probability of the real mean of $\text{Tan}(\phi)$ being .6 or less is less than 10^{-4} even though one of our six samples had a value of .59. Because the normal distribution ranges from $-\infty$ to $+\infty$, the PDF for the sample mean admits finite probabilities of unrealistically low values of $\text{Tan}(\phi)$, even negative values. However these probabilities are infinitesimal. For example, the probability of $\text{Tan}(\phi) \leq 0.5$ is 6×10^{-21} . Inclusion of these unrealistic values of $\text{Tan}(\phi)$ in the calculation of failure probability has no effect since their probabilities are so small.

Stability analysis of the failure mechanism will result in a critical friction angle at which sliding initiates for a given load condition. The probability of that critical $\text{Tan}(\phi)$ being greater than the mean of the failure surface can be read directly off a cumulative probability distribution curve similar to that shown in Fig. R5.3.4. Thus the probability of failure under the given load condition can be determined.

R5.3.4.2 Series Failure Mechanisms

Suppose now that the failure mechanism is series. For example, consider the corbel shear example in R5.3.3. Suppose further that you have no tensile test data, and only the design records that state that the specified concrete strength was 4000 psi. Assuming that the concrete was mixed to insure that the 4000 psi was equaled or exceeded, and that no test fell below 3500 psi, (ACI 318 requirements), it is reasonable to assume that the mean compressive strength at 28 days was 4700 psi with a standard deviation 500 psi. The probability density function (PDF) based on these assumptions is shown below (Fig R5.3.5).

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Suppose the dam is 50 years old. It could be argued that the concrete has gained strength over and above the 28 day strength specified. This is certainly a reasonable assumption, but without any testing, it is difficult to say how much strength gain has occurred. Rather than rush out to obtain field data, it is prudent to see what conclusions can be drawn with respect to failure probabilities based on the data present. It may be that the failure mode can be ruled out based on conservative strength parameters. It may also be possible that the failure mode has a high probability even if very optimistic strength parameters are assumed.

Using Jerome Raphael's relationship between compressive strength and tensile strength,

$$f_t = 1.7 * (f_c')^{2/3}$$

one can calculate the expected tensile strength using 4700 for f_c' . The resulting value for f_t is 477 psi. However, because the compressive strength is variable, described by the PDF shown in Fig. R5.3.5, the tensile strength will also be variable.

One can generate a PDF of tensile strength based on the assumption of normally distributed compressive strength. From statistical theory, if a variable f_c' has a PDF $P(f_c')$, and f_t is a function of f_c' , $f_t = F(f_c')$ such that $f_c' = G(f_t)$, where $G = F$ inverse, then the PDF of f_t is as follows:

$$PDF(f_t) = P(G(f_t)) * \left(\frac{dG}{df_t} \right)$$

In our case, the PDF of f_c' is, $n(4700,500)$. Our G function is the inverse of the Raphael equation:

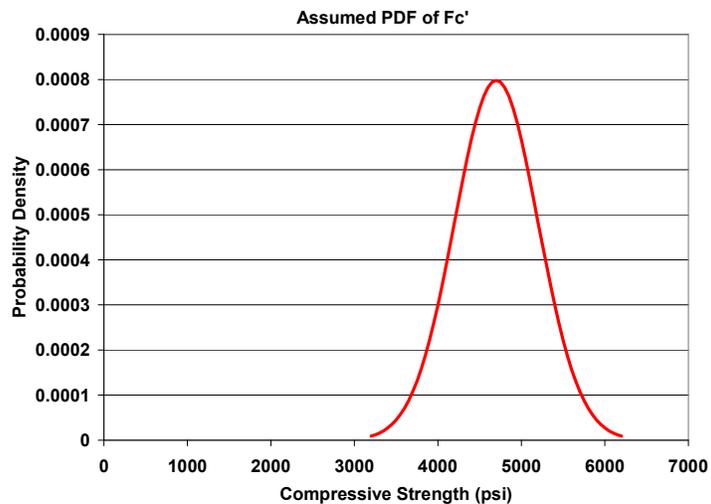


Fig. R5.3.5

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$$f'_c = \left(\frac{f_t}{1.7} \right)^{1.5}$$

The derivative of this G function with respect to f_t is:

$$\frac{dG}{df_t} = \left(\frac{1}{1.7} \right)^{1.5} * 1.5 * \sqrt{f_t}$$

The PDF for f_t is then:

$$PDF(f_t) = \left(\frac{1}{500 * \sqrt{2 * \pi}} \right) * \exp \left(- \left(\left(\frac{f_t}{1.7} \right)^{1.5} - 4700 \right)^2 \right)$$

This PDF is now no longer normal in f_t . It is depicted in Fig R5.3.6. The mean of this PDF is 475 psi.

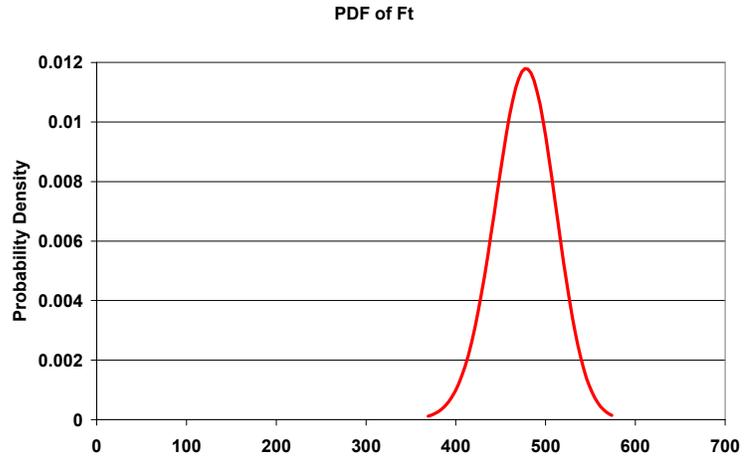


Fig. R51.3.6

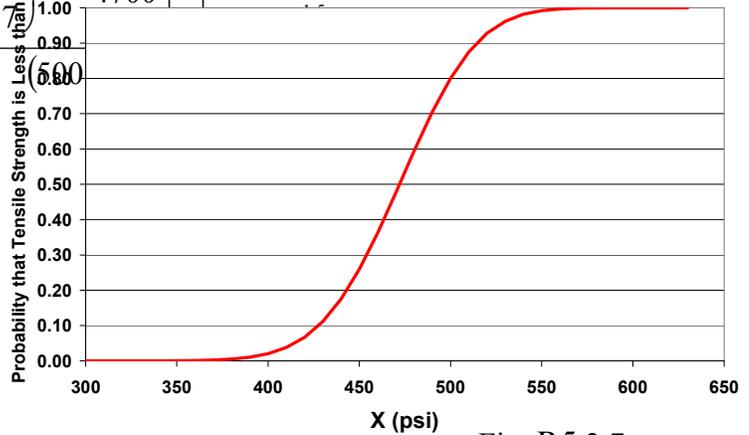


Fig. R5.3.7

This PDF can be integrated to yield a cumulative probability curve, the probability that the tensile strength is less than X. (See Fig. R5.3.7)

Consider a dam 50 ft in height with buttresses constructed in 10 ft. lifts. It is reasonable to assume that the concrete properties will not vary much within an individual placement. Figure R5.3.8 shows the variation of peak principal axis tension at the reentrant corner of the corbel as a function of dam height. Note the maximum stresses that occur in the first, second and third lifts are 425 psi, 409 psi and 328 psi respectively.

Now the probability of corbel failure must be estimated. The probability that lift 1 has a tensile strength of less than the 425 psi shown in figure R5.3.8 can be read off Figure R5.3.7.

$$Pr(F_t < 425) = 0.089$$

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Also:

$$\Pr(F_t < 409) = 0.036$$

$$\Pr(F_t < 327) = 0.00004$$

The probability that the corbel will **not** fail then is given by:

$$(1 - 0.089) * (1 - 0.036) * (1 - 0.00004) = 0.88$$

Suppose there are 20 buttresses. The probability that the dam will **not** fail is then:

$$(0.88)^{20} = 0.078$$

This implies that failure is almost certain. What is important to note is that failure is almost certain event though the peak tensile stress (425 psi) is 52 psi below the strength predicted by the Raphael equation.

This was a series failure mechanism for 2 reasons. First, the process of brittle fracture is inherently series. This fact required us to not pay attention to the mean, but to consider the variation of tensile strength which allows a finite probability of failure even when stress is significantly below the mean value of strength. Second, in a slab and buttress dam, all it takes is one slab to fail to release the reservoir. This fact required us to raise the 88% probability of the successful performance of 1 buttress to the 20th power.

R5.3.5 Characteristics of Material Parameters.

The table below attempts to group some commonly used parameters with their associated failure mechanisms and assign them the characteristic of series or parallel. The table is not an exhaustive list of failure mechanisms or material parameters, and there may be particular exceptions to the characterizations listed. For example, "Tensile Failure of Rebar" could be a series failure

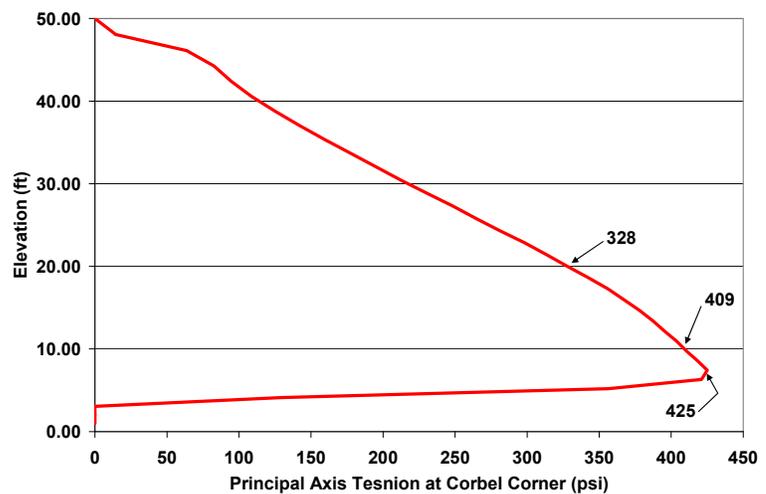


Fig. R5.3.8

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mechanism if the steel is brittle.

Failure Mechanism	Parameter	Characteristic
Sliding on a Rock or Concrete Failure Plane	Peak $\tan(\phi)$	Parallel
Sliding on any Failure Plane	Residual $\tan(\phi)$	Parallel
Sliding/Floating/Overturning	Unit Wt	Parallel
Sliding on a Rock or Concrete Failure Plane	Cohesion	Series
Tensile Cracking (Rock or Concrete)	F_t	Series
Tensile Failure of Rebar	F_y	Parallel

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R5.4 Gravity Dams

R5.4.1 Historic Failures and Near Failures

R5.4.1.1 Austin (Bayless) Dam:

43 ft high dam, Austin, Pennsylvania - failed in 1911

Austin (Bayless) Dam is a straight gravity dam made of cyclopean masonry completed on December 1, 1909 located 1.5 miles upstream from Austin. The concrete dam was a major engineering feat for its time and impounded 275 million gallons of water (about 850 acre-feet).

Bedrock underlying the dam site consists of horizontally layered sedimentary rocks: thin layers of shales between narrow layers of sandstone. The dam's foundation was described in Anonymous (1911) as being constructed on a 2-foot-thick layer of sandstone. The surface rock was well washed and grouted.

A 4-foot-deep by 4-foot-wide keyway extended into the bedrock and the abutments were also cut 20 feet into bedrock (Freiman and Schlager 1995b, p. 207). Compacted earthfill was placed along the heel to reduce seepage under the dam. Twisted 1.25-inch-diameter steel rods, 25 feet long, spaced 2.75 feet apart, 6-feet from the upstream face were grouted 5 to 8 feet into the foundation to reinforce the dam. Numerous design recommendations were ignored. The dam reservoir was filled within about six weeks of completion.

On January 17, 1910, a combination of heavy rain and snowmelt due to warm temperatures caused the reservoir to sharply raise and flow over the spillway. An undetermined thickness of ice still covered the reservoir. Heavy seepage near the toe of the dam with water bubbling up from the ground as far as 50 feet downstream and cracks on the downstream face of the dam were observed. The dam developed six prominent vertical cracks. The dam had been built without construction joints, so the cracks divided the dam into seven separate segments. A portion of the eastern earthen embankment slid down eight feet, allowing water to bypass the dam. To prevent a catastrophe, Bayless used dynamite to blast two notches in the



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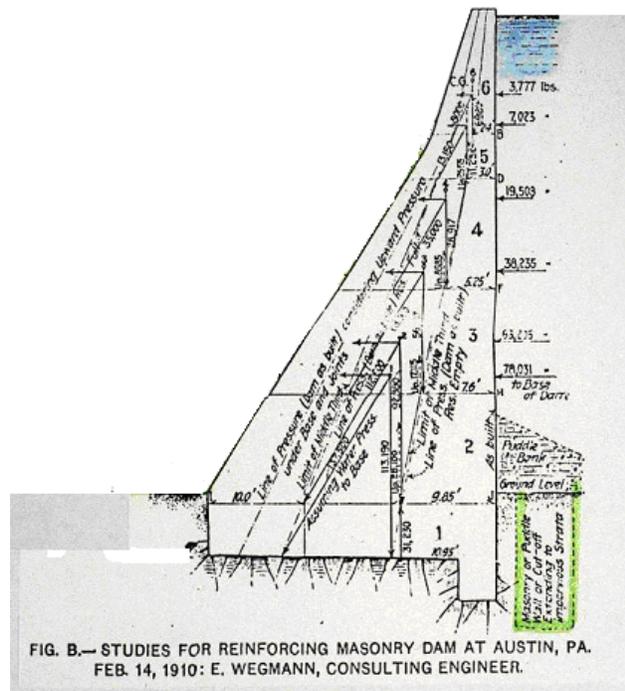
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dam crest to lower the reservoir. The blasting had been necessary because the dam had no spillway gates to allow controlled water release (Greene and Christ 1998, p. 9). The combination of these efforts emptied the reservoir in around sixteen hours and major devastation was circumvented. The massive structure was pushed downstream an amazing 18 inches at its base and 31 inches at its crest. Superficial repairs were made to the dam after the water level dropped, and then the water level was allowed to rise to normal pool again despite leakage under the dam of about 600 gallons per minute (Freiman and Schlager 1995b, p. 208).

Rainfall in September 1911 was unusually heavy. Water began flowing over the spillway for the first time in about 20 months. The dam failed on the afternoon of Saturday, September 30, 1911. Post-failure analysis clearly showed that the plane of sliding developed in the shale underlying the sandstone, which represented the weakest zone in the foundation (Greene, 1997). In essence, the upper sandstone layer slid downstream along with the concrete blocks comprising the dam. This was a classic foundation failure in which the dam failed by sliding.

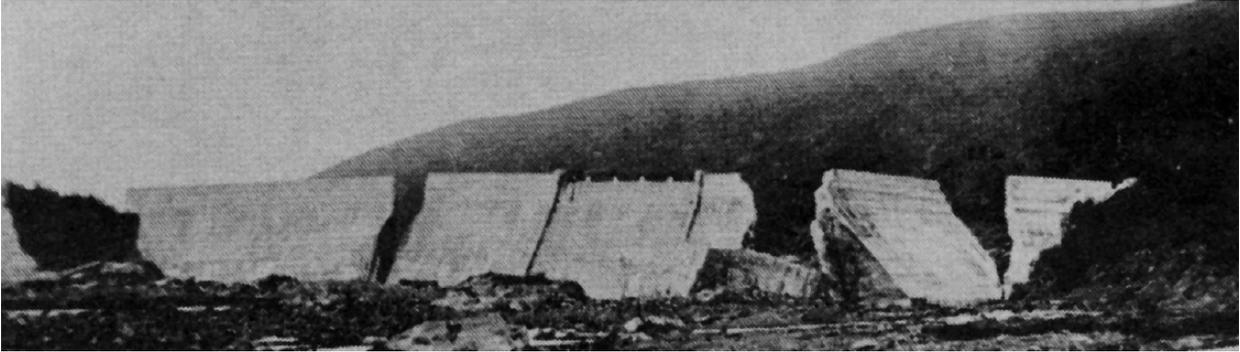
The paper mill whistle blew but warning went unheeded due to previous false alarms. The water smashed into the town of Austin in about 11 minutes engulfing the town of 2,300 inhabitants and killing 78 people.

Back analysis suggests that sliding occurred on a weak shale layer within the foundation (Anderson et al, 1998). A friction angle of 48 degrees was required to prevent sliding if there is no cracking at the heel. Limit equilibrium analysis with reservoir at the crest of the dam and full uplift compute 26 lb/in² vertical tension at the heel with the resultant force well outside the kern of the base and almost to the toe of the dam. As a result, reservoir water could enter an upstream crack. Taking this into account, cracked-base analysis indicates cracks would extend through the dam to the toe. Using an estimated actual friction angle of probably between 25 and 31 degrees produces a sliding factor of safety less than 1.0.



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Failed Austin Dam

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R5.4.1.2 Bouzey Dam:

75 ft high dam, Epinal, France - Failed in 1895

Bouzey Dam was a masonry gravity dam constructed across the L'Aviere River. Similar to Austin Dam, the dam was founded on horizontally interbedded sandstone and lenticular clay seams, with no drainage provisions in the dam or foundation. The original design height for Bouzey Dam in 1876 was 65 feet, but was changed to 74.5 feet in 1880. The masonry and mortar construction reportedly had a density of 125 lb/ft³. A 6.6-foot-wide cutoff wall built along the upstream heel of the dam varied from 14- to 33-feet deep. Lime with dirty sand of poor quality was used for the mortar and it is alleged that the preparation of the mortar was done carelessly.

Bouzey Dam was completed at the end of 1880 and began filling in late 1881. As the reservoir filled, the dam began to leak with seepages over 1000 gpm. As seepages reached 3600 gpm with the reservoir at over 60 feet in March 1884, the center 440 feet of the dam moved downstream about a foot, shearing the key and causing a 305 foot long crack along the heel without vertical settlement. The rock was crushed and dislocated to a depth of 6 to 10 feet under the dam, with lenticular clay deposits several mm thick and openings allowed seepage. After the partial failure, the dam was strengthened from 1888 to 1889 by adding a downstream buttress, covering and sealing the upstream crack and dislocated heel with masonry, and covering the heel with clay to reduce seepage. Vertical cracks were grouted and drains were installed in an attempt to prevent further sliding. Piezometers showed uplift pressures of 33 feet of head, but measurements were ultimately abandoned because it was thought that the dam was performing well.

The upper portion of the dam had been left unchanged with a thickness of 18 for about the upper 35 feet, and upon refilling the reservoir to over 60 feet, the dam failed in 1895. Eyewitnesses said that 66 feet of the crest in the middle collapsed and then 33 feet of a 594 foot length followed. Stability calculations indicate that cracking was likely at the elevation where the shear failure occurred, and once cracked through, the upper portion of the dam was subject to uplift and became unstable. (Anderson et al, 1998).

The flood wave swept through the city of Bouzey, just 1000 feet downstream, and caused more than 100 deaths. This was the first time that uplift was recognized as a contributor to dam failure.

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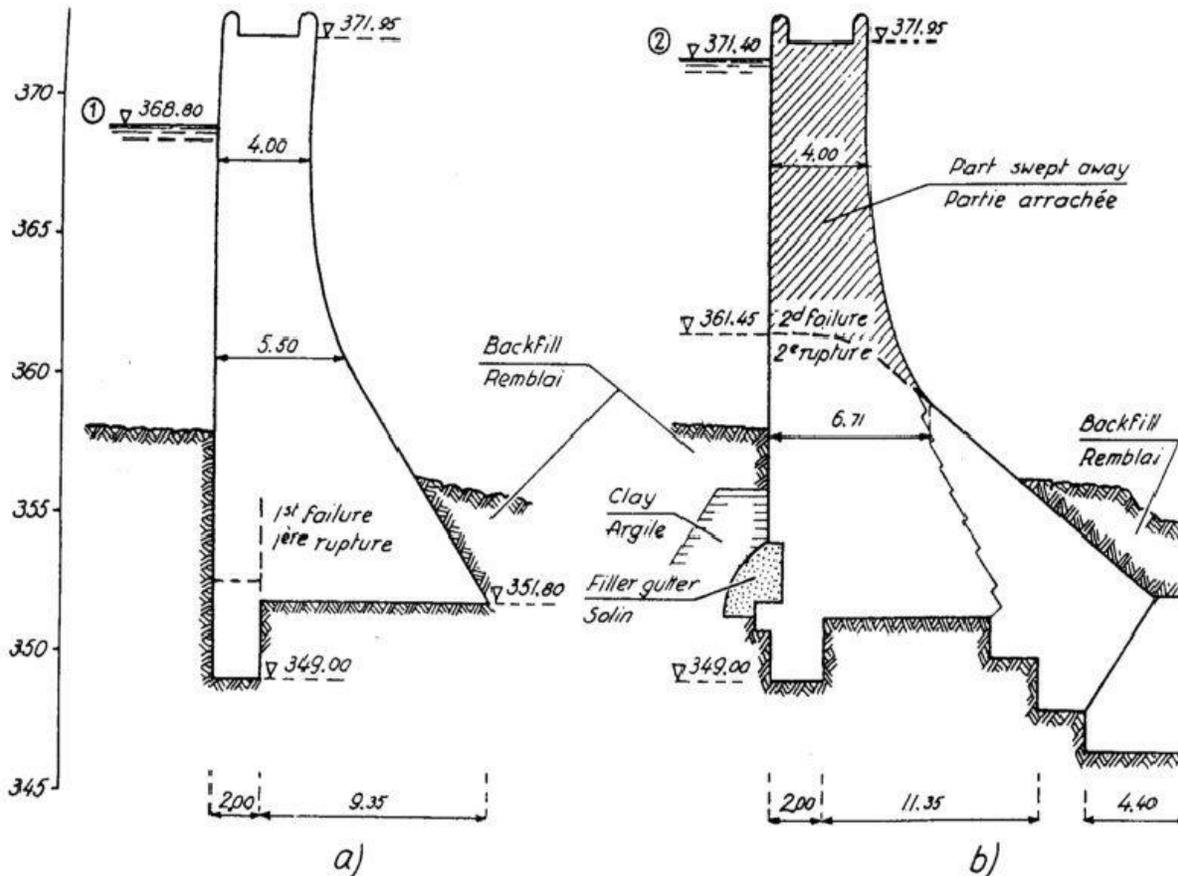
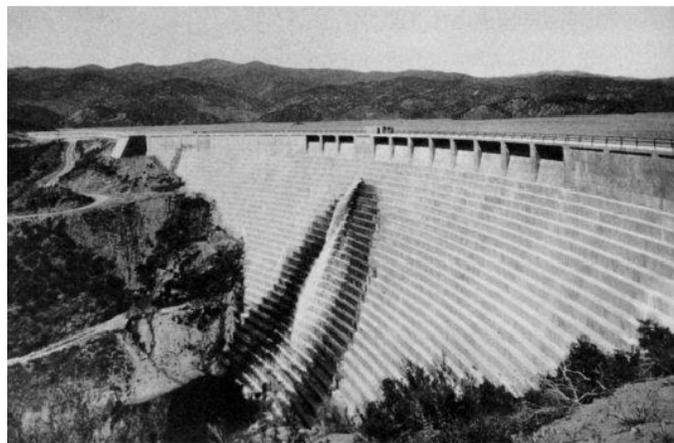


Figure showing part of Bouzey dam that was swept away.

R5.4.1.3 St. Francis Dam:

205 foot high dam, San Francisquito Canyon, CA - failed in 1928

St. Francis Dam was a curved concrete gravity dam constructed approximately 45 miles north of Los Angeles California. The dam was 16 feet thick at the crest, and 175 feet thick at the base. The crest length of the main dam was about 700 feet. The dam had no contraction joints or inspection gallery. The foundation was not pressure grouted, and drainage was installed only under the center section. The foundation was comprised of two kinds



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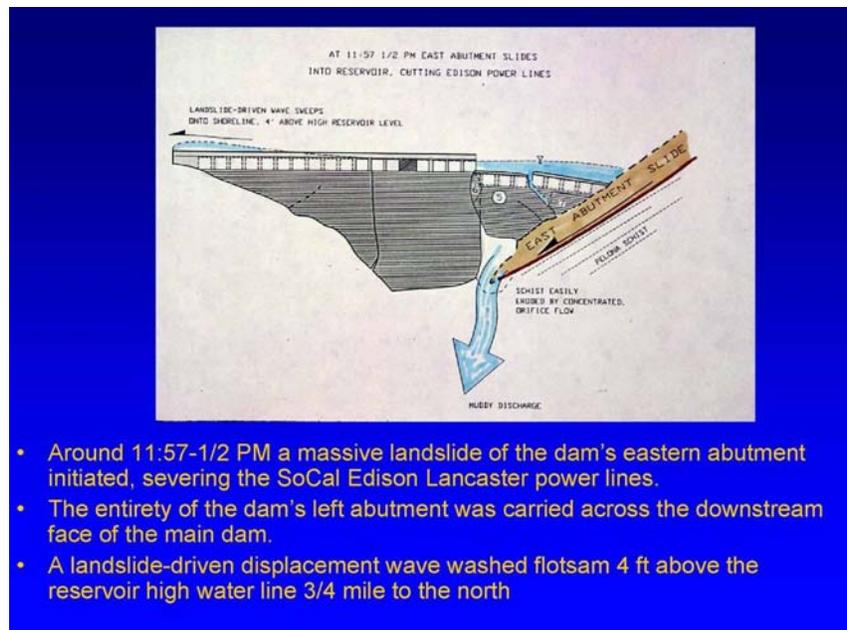
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of rock. The canyon floor and the left abutment was relatively uniform Pelona Schist (mica schist) known locally as “greywacke” shale. The schistosity planes were essentially parallel to the canyon wall, dipping toward the canyon at about 35 degrees. The contact between the two formations was a fault which, at the dam site, had a strike approximately parallel with the stream and dipped into the right abutment at about 35 degrees. The dam was placed across the fault with full knowledge of its existence. Mulholland chose the site for the dam because of a favorable topography with the narrowing canyon. However, the reason for the narrowing canyon was the ancient paleo-mega-landslide on the left abutment.

Large tension cracks were noted in the schist on the left abutment two days before the failure. The morning of the failure, muddy water was reported to be leaking from the right abutment, but when examined in detail, the flow was found to be clear, picking up sediment only as it ran down the abutment. Another leak on the left abutment was similarly dismissed as normal leakage. Several hours before failure the reservoir gage recorded a sudden 3.6 inch drop in the reservoir level.

Several people drove by the dam just minutes before failure. One person reported crossing a 12-inch-high scarp across the roadway upstream of the dam. The dam failed suddenly at 11:58 p.m. on March 12, 1928. Within 70 minutes, the entire 38,000 acre-foot reservoir was drained. A 110 foot high flood wave devastated the river channel for 54 miles to the Pacific Ocean. It has been estimated that 470 lives were lost (Anderson et al, 1998).

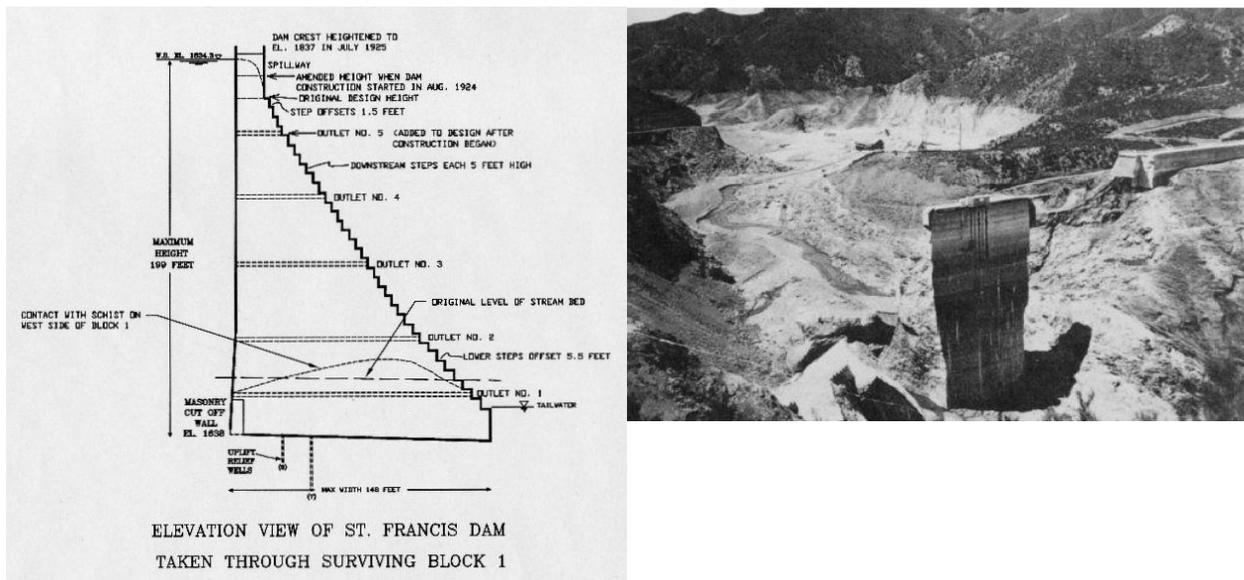
Reanalysis of the disaster indicated that failure was initiated by sliding along weak foliation planes in the left abutment on a remnant of the old paleo-landslide due to the reactivation of 500,000 cubic yards of slide material. The slide mass had been saturated to a depth of 200 feet due to the left abutment’s high porosity. The keyblocks beneath the left slope appear to have been



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hydraulically lifted and broke the dam, initiating a rapid chain reaction failure similar to Malpasset dam.



R5.4.1.4 Camara Dam:

160 foot high dam, Paraiba, Brazil –Failed in 2004

Camara Dam is a straight roller compacted concrete gravity dam started in 2001 and completed in February 2002, located upstream from the towns of Alagoa Grande and Mulungu. It had a crest length of 817 feet, a base width of 130 feet, and a crest width of 20 feet. The schistosity on the left abutment dips about 30 to 35 degrees.

The reservoir filled rapidly in about 2 weeks due to heavy rains in January 2004 and then gradually increased to 35 feet below the crest by June 17, 2004 at the time of the rupture. Some of the observations from January 2004 to the rupture included: Water noise in the gallery and carrying of material in the drains, a damp spot in the left abutment in the region of the fault, over half the drains in the gallery are clogged and those that were working had high flows leading to flooding in the gallery. A crack also developed in the

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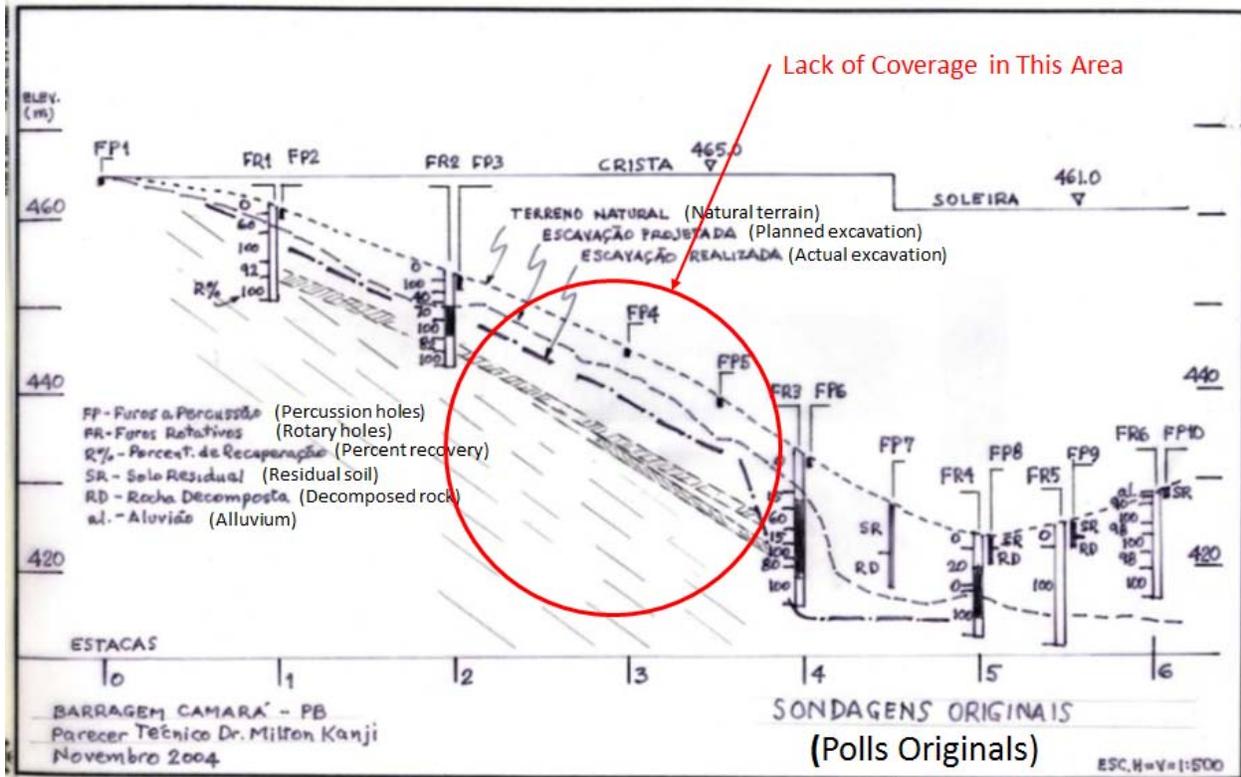
drainage gallery. The left abutment failed below the concrete dam due to erosion from soil-filled discontinuities, causing a flood that killed 5 people and left 800 people homeless.

Post-failure analysis determined that during the design the assessment of the left abutment geology was not precise because some of the site geology was obtained from percussion drilling. A 12-inch thick soil fill was interpreted as being limited in depth to 10 feet into the abutment. The treatment of the left abutment was not sufficient and allowed the preservation of an extensive shear zone. The inspections were sporadic and leakage was ignored. A recommendation to drain the reservoir was ignored. Flowing reservoir water washed out the foundation material to the point that drains were being plugged. As a result, high pressure gradients developed under the dam. As the flow of reservoir water increased the erosion and driving forces on the low-shear strength rock slabs, they began to slide and fail in an upstream progressive manner until a hole was piped beneath the dam. The remaining slab of rock in left abutment is essentially fracture free, therefore percolation occurred along the entire length of the fracture.

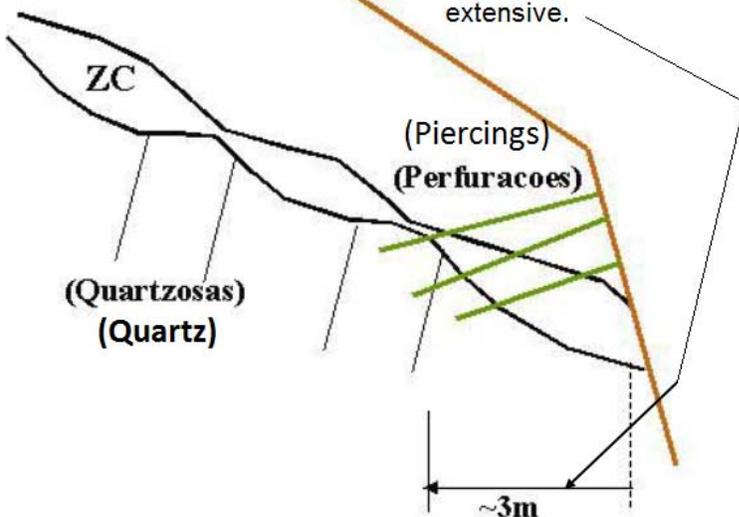


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It was thought that Shear Zone ZC only extended back into the foundation 10 feet because of the thinning of the bed. However, the zone was much more extensive.



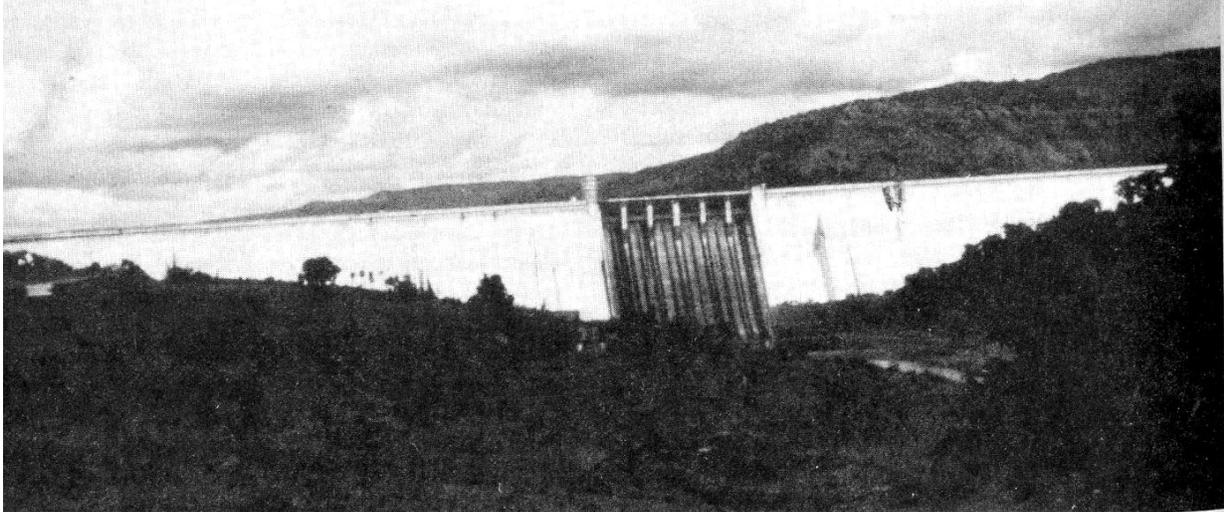
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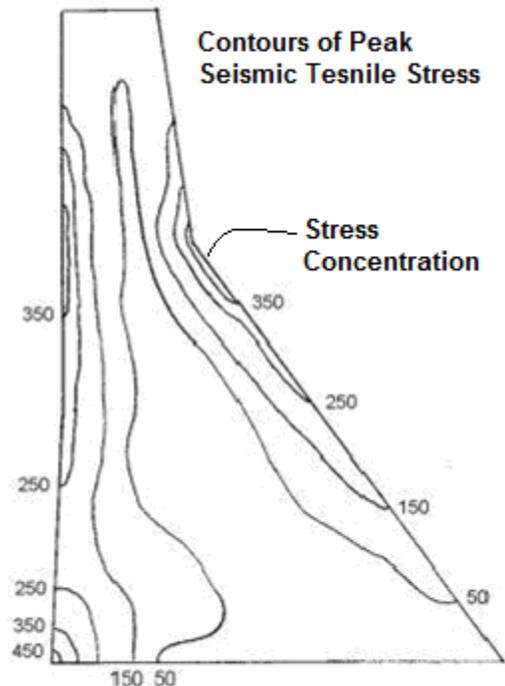
R5.4.1.5 Koyna Dam:

338 foot high dam, India - cracked in 1967

Koyna Dam is a 2,800 foot long concrete gravity dam with 50 foot wide monoliths. It was constructed on the Koyna River in southwestern India between 1954 and 1963. During construction the decision was made to raise the dam and the downstream slope of the non-overflow section was steepened in the upper 120 feet of the structure to accommodate the raise, resulting in a discontinuous change in slope at that location.



The dam was shaken by a M6.5 earthquake on December 11, 1967. A strong motion accelerograph located in a gallery on the upper right abutment recorded a peak ground acceleration of 0.63 g cross-canyon, 0.49 g downstream, and 0.34 g vertical. Although the dam did not fail, deep horizontal cracks formed throughout the upstream and downstream faces near the change in slope where a stress concentration is expected to occur, requiring the installation of tendons and construction of buttresses on the downstream face to stabilize the structure. Finite element analyses indicated stress concentrations near the change in slope that exceed the dynamic tensile strength of the concrete (Anderson et al, 1998). The joints of the dam were not keyed or grouted.



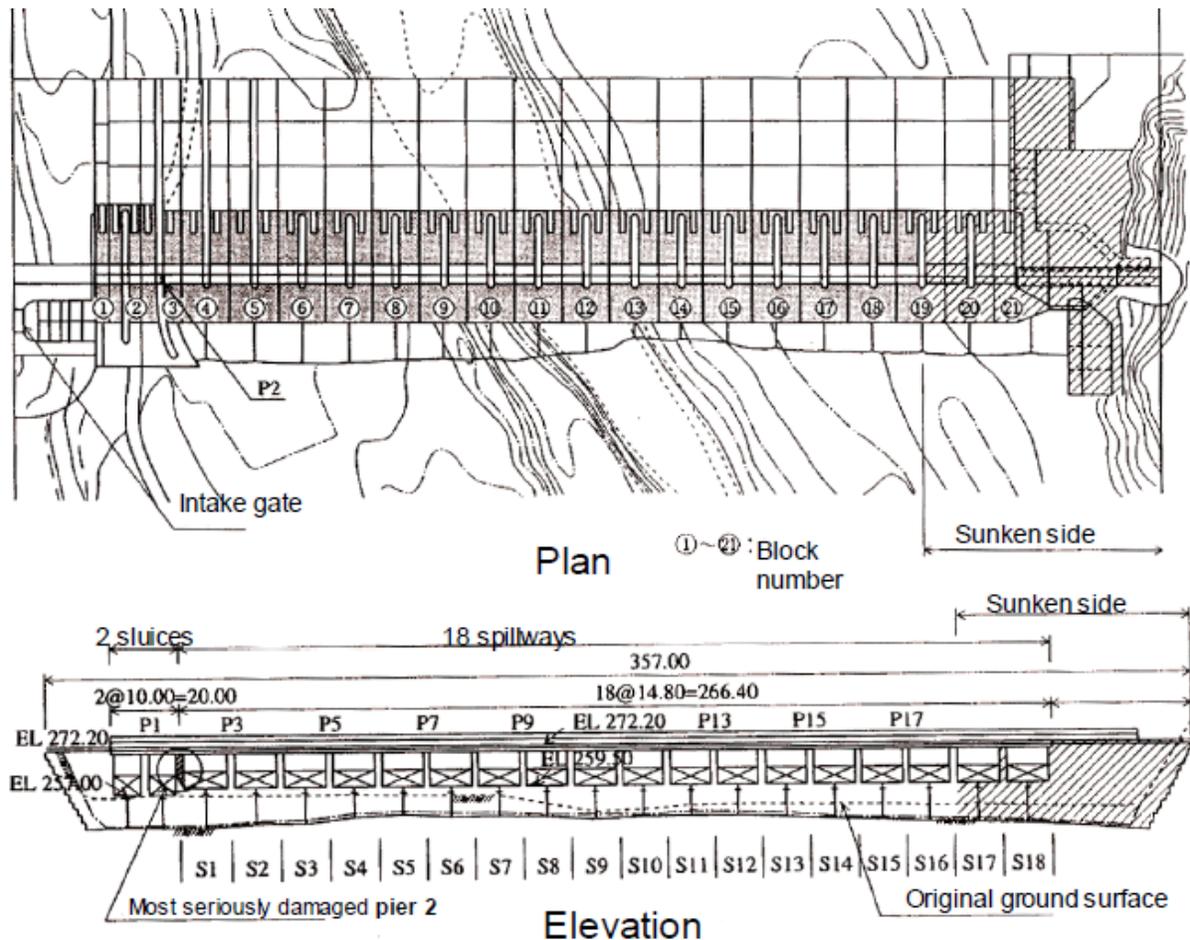
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R5.4.1.6 Shi Kang Dam:

70 ft high Dam, Shi-Kang, Taiwan – Failed in 1999 during the Chi-Chi earthquake.

Shih-Kang dam is a concrete gravity dam completed in 1997 with 18 spillway bays controlled by Tainter gates. The design was based on the traditional design concept of the pseudo static earthquake acceleration. The design horizontal earthquake acceleration coefficient was 0.15 g and the effect of the vertical motion was neglected.



On September 9th 1999 the Chi-Chi earthquake struck at 1:47 am. The peak ground motion at station TCU084 at Sun-Moon Lake, 6 miles away from the epicenter, was 1.0g.

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The horizontal and vertical peak ground motion of station TCU068, located at Shih-Kang Primary School 500 meters away from the dam, were 0.51 g and 0.53 g, respectively. A fault running underneath the dam caused enormous offsets predominantly in the vertical (over 30 feet) and cross-stream (20 feet) direction, leading to the failure of multiple bays and widespread cracking. Although sections of the dam overlaying the fault failed, the rest of the structure performed quite well and stayed intact. The reservoir was also offset and therefore only a small volume of water was released and the consequences of the earthquake were not exacerbated.

The dam had supplied 50 percent of the water for the Taichung area and its failure led to severe shortages.



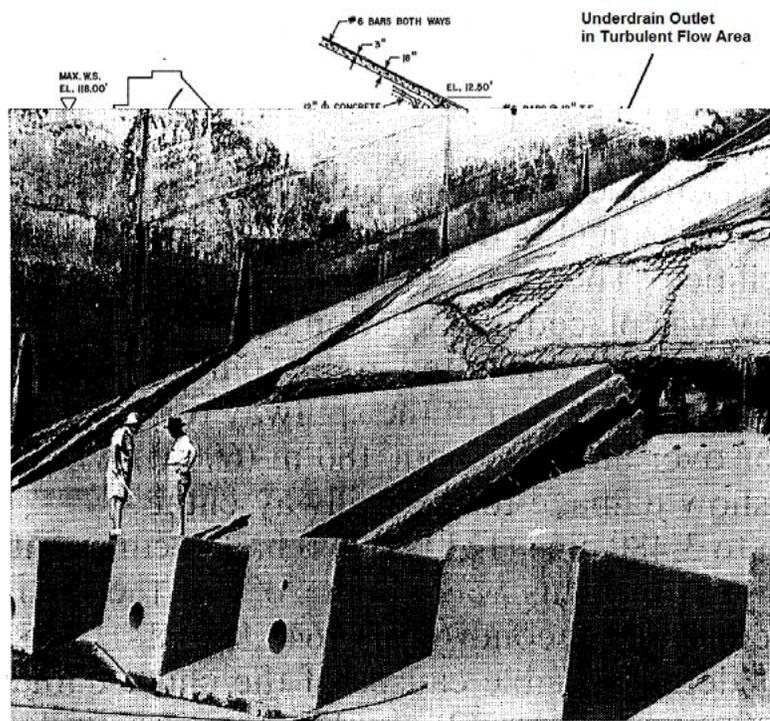
Pier Cracking



Bays 16-18: Right side rose 32 feet while the left side rose 7 feet

R5.4.1.7 Karnafuli

Karnafuli Hydroelectric Project is located in Bangladesh. The project was completed in June 1961. The spillway was opened that year to pass monsoon floods. On August 13, 1961 distress was noted in the spillway flow. Inspection revealed extensive damage to the spillway chute slab. It was



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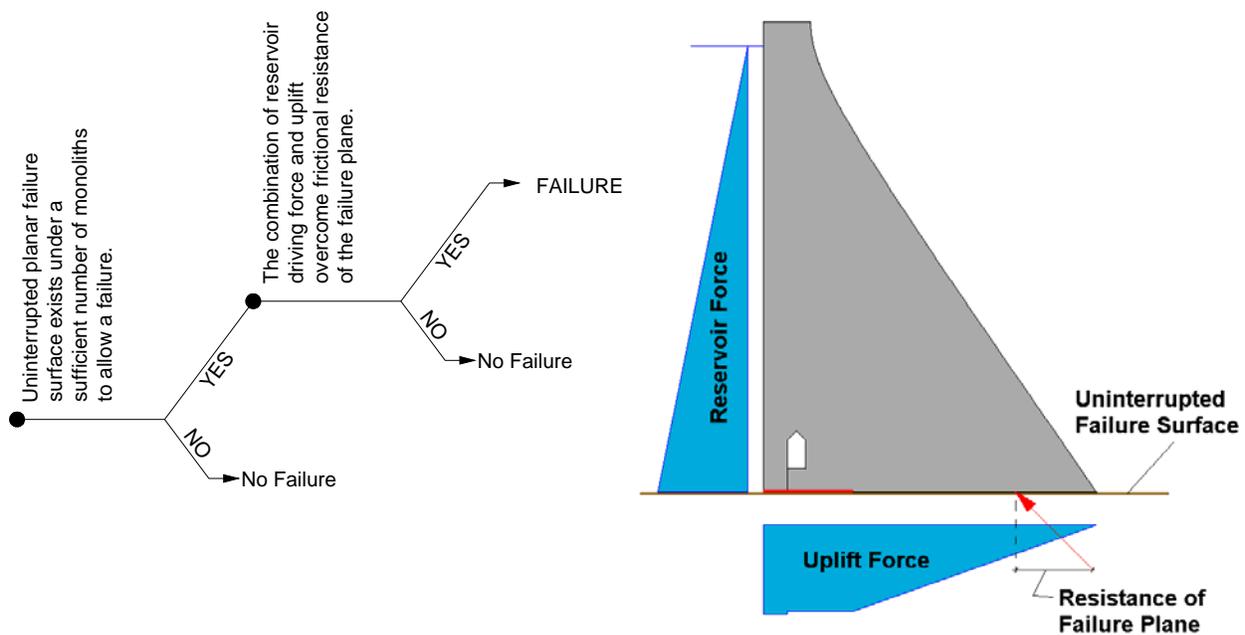
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concluded by C. Edward Bowers of the Saint Anthony Falls Hydraulic Laboratory, that high transient pressures in the hydraulic jump pressurized the drainage system under the chute slab. This is because the drains exited into the turbulent area of the hydraulic jump. These pressures were sufficient to lift the slab off its foundation, crack it, and allow portions of it to be washed away by the spillway flow. A huge amount of erosion of the underlying foundation material resulted.

R5.4.2 Potential Failure Modes.

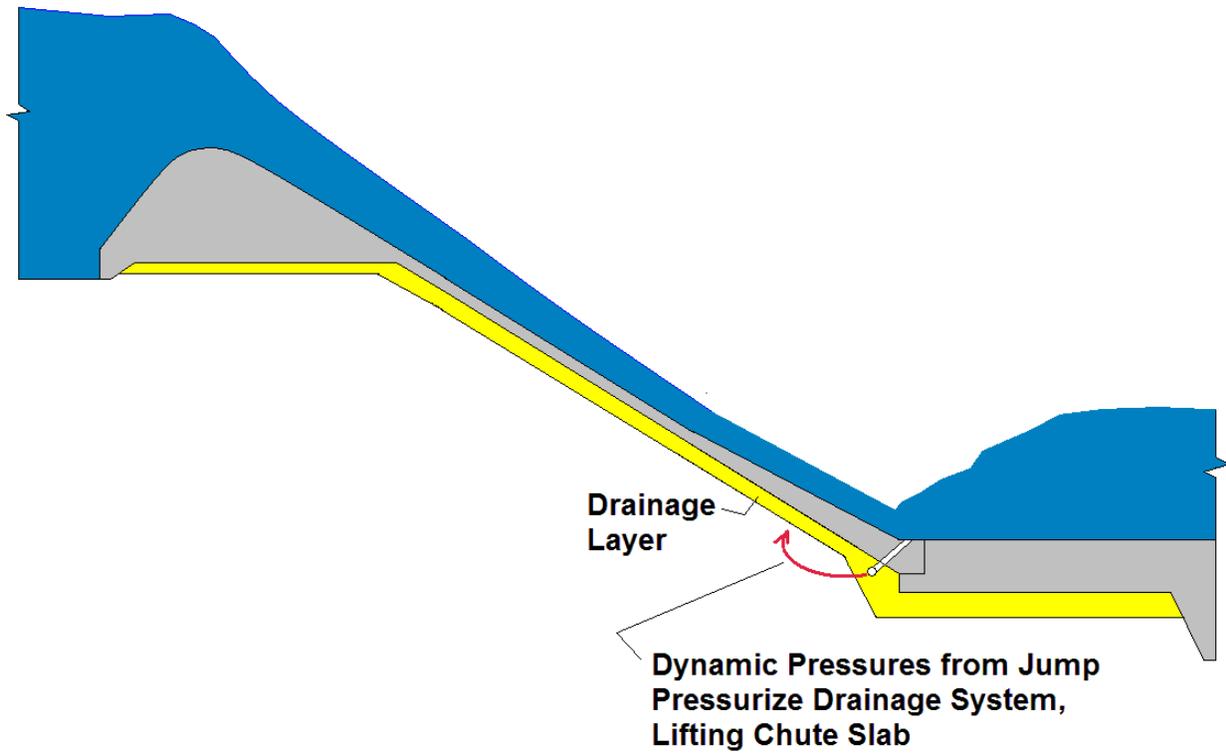
The potential failure modes depicted below are not an exhaustive list. Failure modes must be developed for individual structures on a case by case basis. In a departure from previous guidelines that required calculation of safety factors for a pre-determined set of loading conditions, risk analysis requires that failure modes be identified as part of the engineering effort. Evaluation of only the failure modes below does not constitute due diligence.

R5.4.2.1 Gravity Dam Fails in Sliding/Overturning on the Dam/Foundation Contact.



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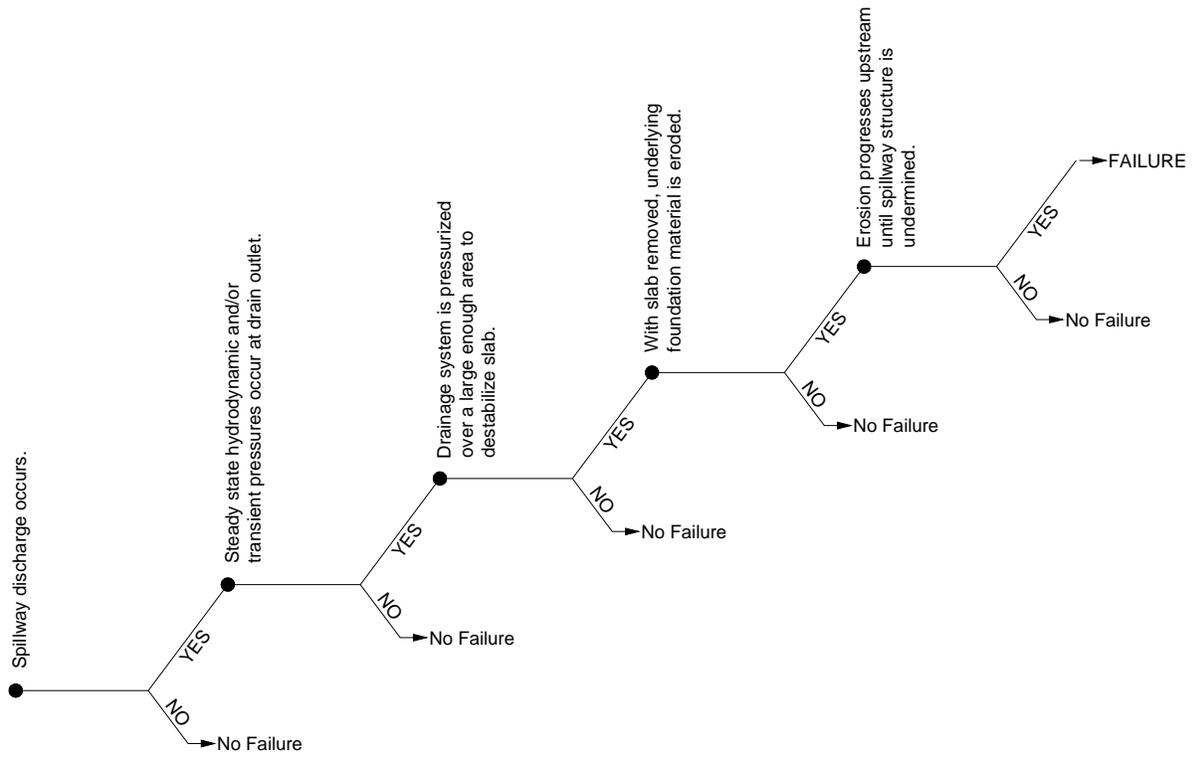
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R5.4.2.5 Drain System Back Pressure Lifts Concrete Slab

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R5.4.3 Analysis Techniques to Develop Risk Information

R5.4.3.1 Gravity Dam Sliding/Overturning due to Flood (See Historic Failure R5.4.1.1 and Potential Failure Mode R5.4.2.1)

Analysis of gravity dams for sliding and overturning due to flood loading can be done using procedures outlined in Chapter 3 of the FERC Engineering Guidelines. These guidelines prescribe procedures for determining factors of safety with respect to a specific reservoir and tailwater loading, specific material properties, and specific assumptions with respect to drain effectiveness. What must be determined in quantitative risk analysis is the annualized probability of failure factoring in all floods, not a factor of safety with respect to one specific flood.

Consider the example below. The failure mode being investigated is sliding/overturning on the interface between the dam and foundation. In this example it will be assumed that there is good confidence in the material parameters, but that there is uncertainty with respect to how well the drains will function in unprecedented flood conditions.

Force	Fx↓	@Y=	Fy↓	@X=	Moment (0,0)
Dam_Weight	0.00	0.00	-540.10	23.27	12568.03
Headwater	281.58	31.67	0.00	0.00	8916.70
Tailwater	-0.00	0.00	-0.00	0.00	-0.00
Uplift	-0.00	0.00	257.86	24.82	-6399.23
Totals->	281.58		-282.24		15085.50

Resultant Location @ X= 53.45 Y= 0 26.7

% of base cracked

Heel Stress= 0 (ksf) Toe Stress= 10.901 (ksf)

Sliding Safety Factor= 1.00

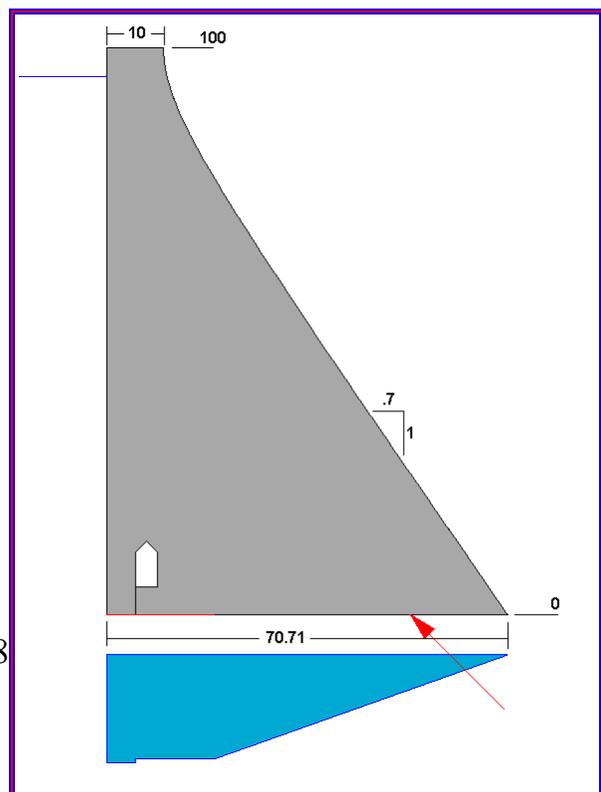
Concrete unit wt. 0.145

Drain effect. 0.035

Foundation phi & C 45 0

Head/tailwater el 95 0

Using the simple gravity analysis shown above, the drain effectiveness can be raised or lowered to yield a factor of safety of



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exactly 1.0. For the case above, if the drain effectiveness is less than .035, a reservoir elevation of 95 will fail the dam. The procedure can be repeated for a whole suite of possible reservoir elevations. Consider for instance the case where the reservoir is at 110, overtopping this dam section by 10 feet, and the tail water is at elevation 20. In this case, 75% drain effectiveness is required.

These repeated analyses yield a relationship between reservoir elevation and drain effectiveness as can be seen adjacent. In a deterministic analysis, one specific drain effectiveness would be assumed, but in quantitative risk analysis, the uncertainty of drain effectiveness can be factored into the probability of failure. This adjacent curve can be used in conjunction with a curve that relates reservoir stage to annual probability of exceedence.

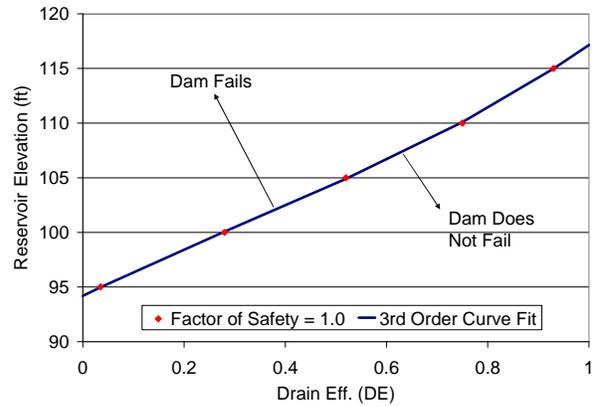
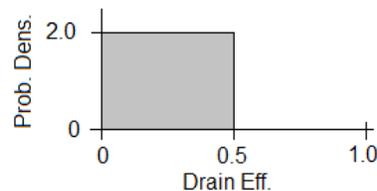


Fig. R5.4.3.1.1

Suppose that peizometric data indicates a drain effectiveness of 50% under normal loading conditions. Suppose there is little or no drain effectiveness data available for large floods. Suppose also that expert elicitation results in a consensus that drain effectiveness under extreme flooding could be anywhere from the current measured 50% to 0. The probability density function should reflect the certainty of the consensus. For example, if there was great confidence that the drain effectiveness would be 25%, a normal distribution centered on 25% with a very small standard deviation may be appropriate. In this case however, let us assume is that all the experts can agree on is that the drain effectiveness will be somewhere between 50% and 0. The appropriate PDF is then a constant (uniform) distribution:

$$\begin{aligned} \text{PDF} &= 2.0 \text{ for } 0 > \text{D.E.} < .5 \\ &= 0 \text{ for } \text{D.E.} \geq .5 \end{aligned}$$



Failure probability is then given by:

$$Pf = \int_0^{.5} AEP_{Elev.} * PDF_{(DE)} * \partial DE \quad \text{Eq. 1}$$

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where P_f is the probability of failure and $AEP_{Elev.}$ is the annual exceedence probability of the reservoir elevation which produces a factor of safety of 1.0 at drain efficiency DE.

The $AEP_{Elev.}$ can be generated from an AEP_{Flow} curve. Consider the flow data and curve fit depicted below. (Fig. R5.4.3.1.2) (This curve is for illustration only. For guidance on flood frequency analysis, consult Chapter D2.) This can be converted from a flow AEP to elevation AEP by the substituting the Q appropriate to the desired stage as shown below:

$$AEP_{Elev.} = AEP_{flow}(Q_{(Y)}) \quad \text{Eq. 2}$$

where Q is total discharge as a function of elevation, and Y is the elevation of interest.

Fig. R5.4.3.1.3 shows discharge (Q) as a function of elevation. What is assumed here is that the dam has a spillway crest at elevation 80 and a dam crest at elevation 100.

Evaluating Eq. 2 with the functions depicted in Fig. R5.4.3.1.2 and Fig. R5.4.3.1.3 yields the $AEP_{Elev.}$ relationship depicted in Fig. R5.4.3.1.4.

With the $AEP_{Elev.}$ now defined, Eq. 1 can be evaluated. The result is .02. In other words the probability of triggering the failure mode considered due to all possible flood events is .02. The concept of safety factor is no longer relevant.

It is interesting to note the effect of drain effectiveness uncertainty. If drain effectiveness of 50% were a certainty, the failure probability under all flood events would have been the AEP of the elevation associated with a drain effectiveness of 50% or elevation 107.9. (See Fig R5.4.3.1.1). The AEP of this elevation is $7e-6$. (See Fig. R5.4.3.1.4)

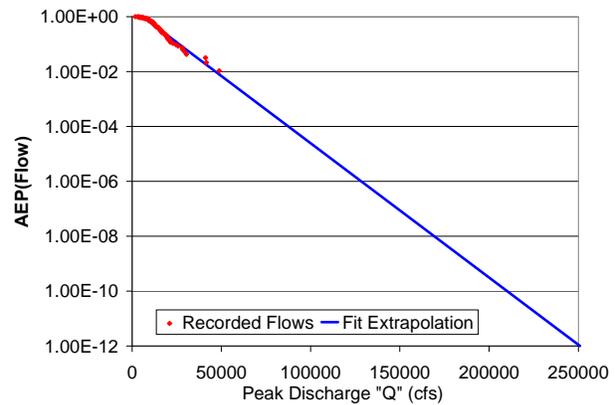


Fig. R5.4.3.1.2

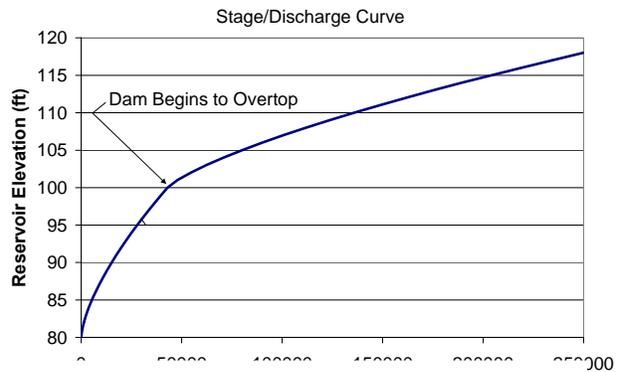
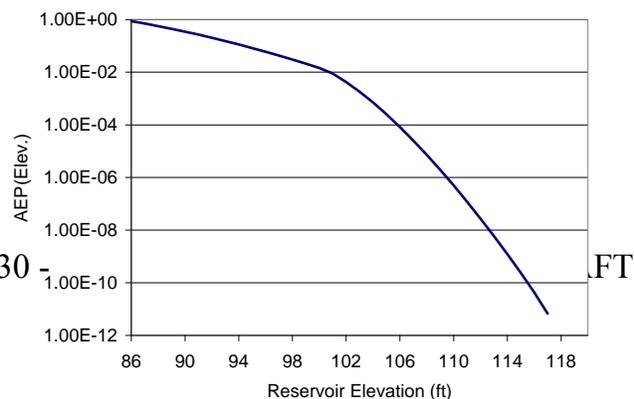


Fig. R5.4.3.1.3



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In this example, there were 2 sources of uncertainty; the flood elevation and the drain effectiveness. Other sources of uncertainty could also be considered. For example, suppose the spillway is controlled by 4 gates, each with an individual gate reliability factor of 95%. That is to say that the probability of a gate failing to operate on demand is .05. This uncertainty can also be factored into the total probability of the failure mode occurring under any flood event.

For a spillway with N gates, the probability of exactly J gates opening is given by the Eq. 3.

Fig. R5.4.3.1.4

$$\Pr(J) = RF^J * (1 - RF)^{(N-J)} * \left(\frac{N!}{(J!) * (N - J)!} \right) \quad \text{Eq. 3}$$

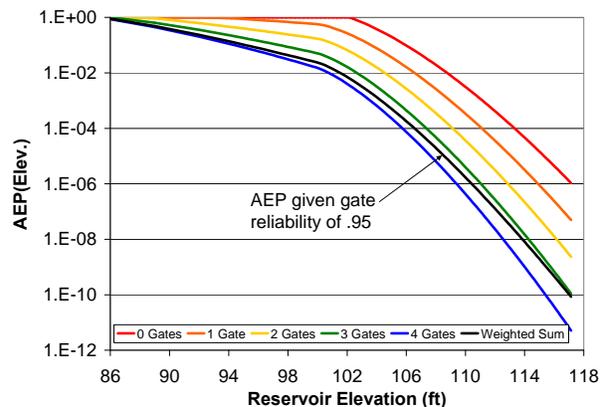
Where RF is the reliability factor of a single gate.

For a 4 gate spillway with RF=.95, the evaluation of Eq. 3 is shown below.

Table 1
Probability that exactly

J	J gates will open
0	6.25E-06
1	0.000475
2	0.013538
3	0.171475
4	0.814506

Since there are 5 different possibilities of gate performance, each with a non zero probability of occurrence, the processes of evaluation Eq. 2 must happen 5 different times, each with a spillway rating curve appropriate to the number of functioning gates. The $AEP_{Elev.s}$ using these 5 different spillway rating curves are then weighted by the values in Table 1 and summed to yield an $AEP_{Elev.}$ factored by



gate reliability. (See the black curve in Fig. R5.4.3.1.5)

Fig. R5.4.3.1.5

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Equation 1 can now be evaluated using this modified $AEP_{Elev.}$ curve. The result is a failure probability of .027 as opposed to the .02 calculated assuming 100% gate reliability.

Note that one specific failure mode was considered here, namely sliding/overturning on the dam/foundation interface plane. Other failure modes would require their own analysis, but what has been presented above would proceed in a similar manor.

The example evaluations above were performed in a single EXCEL spreadsheet of the course of about 8 hours.

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R5.4.3.5 Drain Backpressure Causing Instability. (See Case History R5.4.1.7, Karanfuli & Failure mode R5.4.2.5)

Probabilities can be assigned to the event tree in R5.4.2.5. A relationship between spillway discharge and pressure at the drain outlet must be determined. This coupled with the AEP of flow will give an AEP of drain pressure.

The pressure from the drain must exert itself over a large enough area of the slab to overcome the slab weight and the weight of the water on top of the slab. In addition, the pressure must persist long enough to move the slab through some finite distance. If for example, the pressure is due to a dynamic transient lasting 1/100th of a second, the slab won't have time to move. However, the difficulty of accurately computing the potentially time varying underslab pressure distribution can be daunting. If the drain pressure exceeds what is required to lift a unit block of the slab, it is conservative but not unrealistic to assume that the slab will fail. This threshold pressure (P_t) is easy to calculate.

The probability of slab failure is then:

$$P_f = AEP_{\text{Drain Pres.}} |_{P_t}$$

Where:

P_f is the probability of slab failure.

$ARP_{\text{Drain Pres.}}$ is the annual exceedence probability of a drain pressure evaluated at P_t .

P_t is the drain pressure that equals the weight of a unit block of slab + the overflowing water.

The probability of the foundation material being eroded is a function of unit discharge and foundation erodibility. A fragility curve could be developed that relates the probability of erosion to unit discharge, but again it is not unreasonable to assume that at some reasonably conservative threshold, the probability of erosion goes from 0 to 1.0. If this threshold unit discharge is smaller than that which failed the slab to begin with, then the probability of erosion equals the probability of slab failure.

The probability of erosion continuing upstream is a function of time, discharge, and erodibility. In the case of Karnafuli, this is where the failure sequence stalled. However, in lieu of convincing studies to the contrary, if erosion is likely, there is no reason for

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confidence that the dam won't fail. In such a case, the probability of dam failure is equal to P_f from the equation shown above.

In many cases, it may be prudent to do no risk analysis at all, but to simply route all drain outlets to areas of still water, thus precluding the failure mode.

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R5.5 Arch Dams

R5.5.1 Historic Failures and Near Failures

R5.5.1.1 Malpasset Dam: 220 ft high Dam, Frejus, France –Failed in 1959 after a slow initial filling period which took 5 years.

Malpasset Dam was a thin double-curvature concrete arch completed in 1954 in southern France. The dam was 5-feet-thick at the crest and 22-feet-thick at the base. Blanket grouting was performed at the dam-foundation contact, but no grout curtain or drainage was installed, and no instrumentation other than survey monuments was provided. The dam was founded on gneiss.

The reservoir filled for the first time on December 2, 1959. Although earlier there had been some clear seepage noted on the right abutment and a few cracks had been observed in the concrete apron at the toe of the dam, engineers visiting the site on December 2 did not notice anything unusual. About 9:10 p.m. that evening, the dam tender heard a loud cracking sound, about 1 mile downstream of the dam. The sudden failure sent a flood wave down the river causing destruction along a 7 mile course to the Mediterranean Sea. The number of deaths resulting from the failure was reported to be 421.

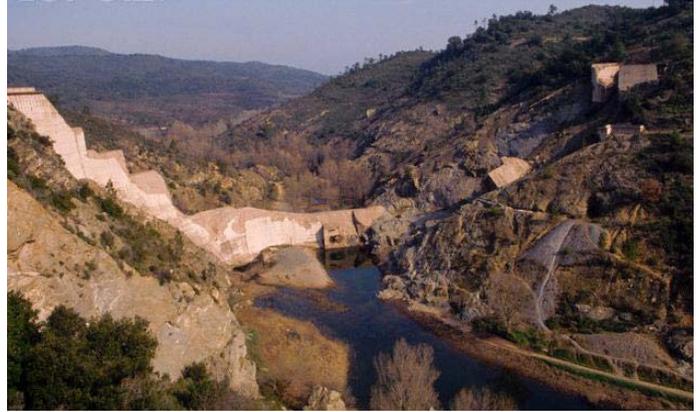
The failure was attributed to sliding of a large block of rock in the left abutment of the dam formed by an upstream dipping fault on the downstream side, and a foliation shear on the upstream side. The “mold” left by removal of the block could be clearly seen following the failure. Large uplift pressures were needed on the upstream shear in order to explain the failure. Experiments suggested that the arch thrust acting parallel to the foliation decreased the permeability perpendicular to the foliation to the point where large uplift pressures could have built up behind a sort of underground dam. The uplift forces in combination with the dam thrust were sufficient to cause the block to slide, taking the dam with it (Anderson et al 1998).

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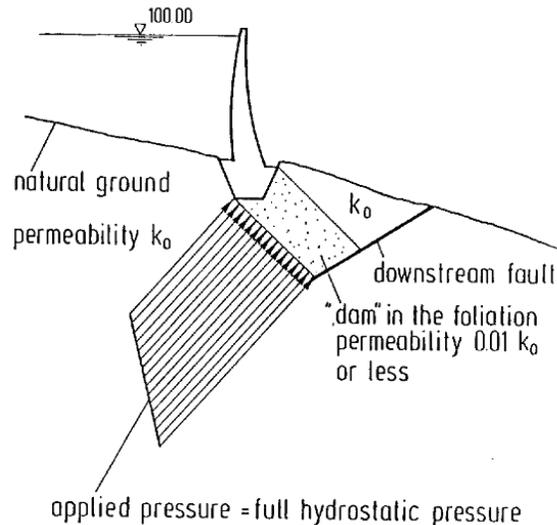
Before Failure



After Failure

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Geology:

The right abutment was massive gneiss and the left abutment was fresh to altered schist. Water tests showed tight abutment

Faults:

East-west (abutment to abutment) striking, upstream dipping North-south (upstream-downstream) striking along foliation, steeply dipping downstream into left abutment. Thrust block moved about 3" into abutment and slightly downstream.

Upstream dipping fault and downstream dipping foliation shear formed left abutment block. Arch thrust in direction of foliation decreased permeability. Tensile stress at upstream face opened foliation shear. Nearly full uplift developed on foliation.

Block slid out on fault ($\phi = 30^\circ$) and dam went with it.

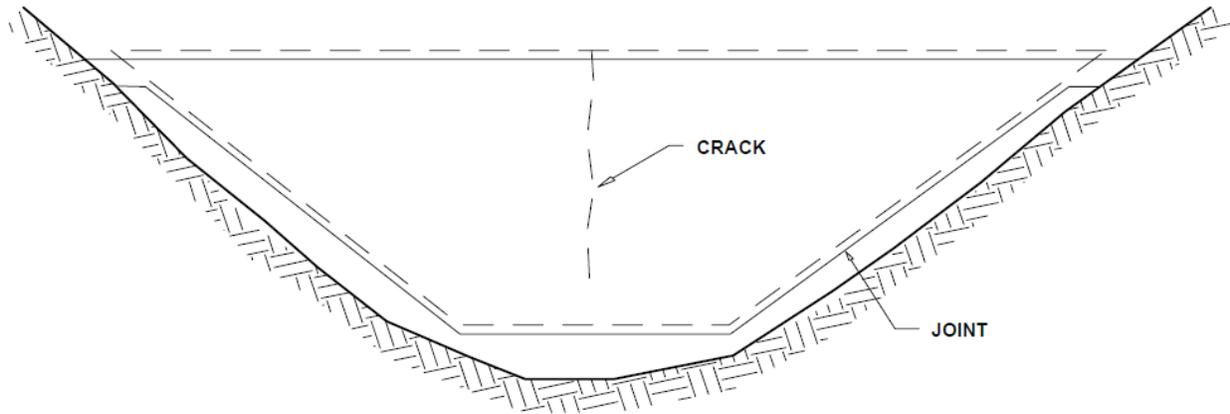
R5.5.1.2 Experimental Plum Dam

Plum Dam, located in Fujian Province in south-east China, was an experimental cylindrical arch dam with a height of 72 ft (22 m) and a crest length of about 238 ft (72.6 m). The dam failed in September 1981 shortly after it was completed in May of the same year. Since the dam was an experimental structure built at a coastline site, its failure caused negligible property damage and no loss of life.

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Field investigations indicated that the failure occurred possibly due to the upward and downstream sliding of the dam along a peripheral joint. The dam was built as a masonry structure composed of granite blocks in the main body of the dam and included a peripheral joint between the dam and its artificial concrete abutment. The joint surfaces were coated with bitumen and polyvinyl chloride was used to seal the joint.



Upward sliding of Plum Arch Dam along its artificial flat abutment.

Completed in May 1981, full storage of the reservoir was reached in June 1981, and the dam was overtopped by 1 foot over the crest on July 20. The dam was overtopped again on September 11 and 12, but no damage or unusual behavior was observed. On the morning of September 18, the dam was inspected and nothing unusual was noticed. At 1:25 pm on the same day a local person had walked across the dam, but 10 minutes later the dam ruptured spectacularly without any warning.

Field investigations of the failure resulted in the following observations:

- The artificial concrete abutment was found to be intact with no sign of cracking or movement.
- The dam body was totally destroyed with smaller debris from the right and central portions of the dam found long distances downstream. Large trapezoid blocks were found near the left abutment.
- The top 5.6 ft (1.7 m) of the dam, which had no peripheral joint, was sheared off at both abutments along the masonry placement joints. At the right abutment, debris showed signs of upward movement of 2 to 3 inches (5 to 8 cm). At the left abutment some debris were found upstream, confirming that the rotation of the dam must have occurred with respect to this abutment.

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- Peripheral joint surface showed two sets of frictional traces, one parallel to the dam axis and another inclined toward the downstream at 30°. The traces parallel to the dam axis were light and those toward the downstream were deep scratches at the upper elevations and shallow traces at lower levels.

Based on the above observations and detailed inspection of the failed dam, the following scenario was offered as the most probable mode of failure

- The body of the dam moved up along the peripheral joint, producing the first set of frictional traces parallel to the axis of the dam.
- The upward movement in turn caused widening of the horizontal arch spans, stressing the crown to the point of rupture.

R5.5.1.3 Bureau of Reclamation Shake Table Tests.

The Bureau of Reclamation (Reclamation), from 1999 to 2001, performed 15 shake table tests on 1/150-scale medium-thick arch dams [Reclamation 2006]. The dams had different geometric nonlinearities: monolithic, one vertical contraction joint along the crown cantilever, one horizontal unbonded mid-height lift surface, 17 vertical contraction joints, and 17 vertical contraction joints with 2 horizontal unbonded lift surfaces.

Scale laboratory models have the difficult task of meeting similitude requirements. As such, materials used in the tests do not necessarily resemble concrete, water, or foundation rock. However, they provide greater understanding of the failure mechanisms of arch dams under seismic loading. Note that in every case, cracking developed in the dam approximately parallel to the valley wall. Vertical cracking also occurred. These cracks then formed free monoliths capable of independent movement.

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Initial cracks and final failure photographs

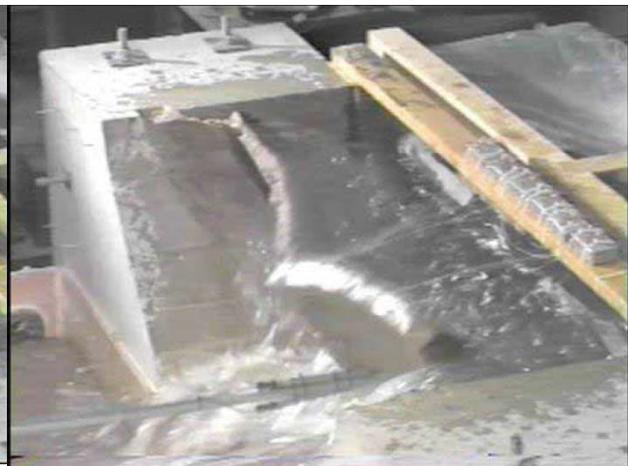
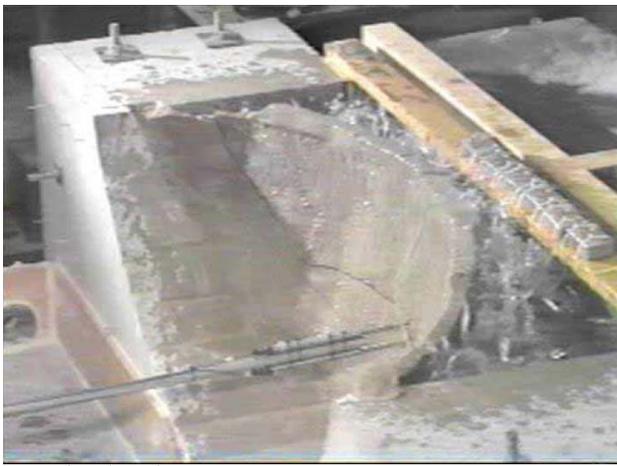
Initial Crack

Final Failure

Monolithic model



Horizontal Joint



Vertical Joint

Vertical Joint



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R5.5.2 Potential Failure Modes

R5.5.2.1 Unstable Abutment Rock Block Fails, Denying Support to the Arch:

This failure mode has a dramatic historical example in the Malpasset dam failure (See R5.5.1.1)

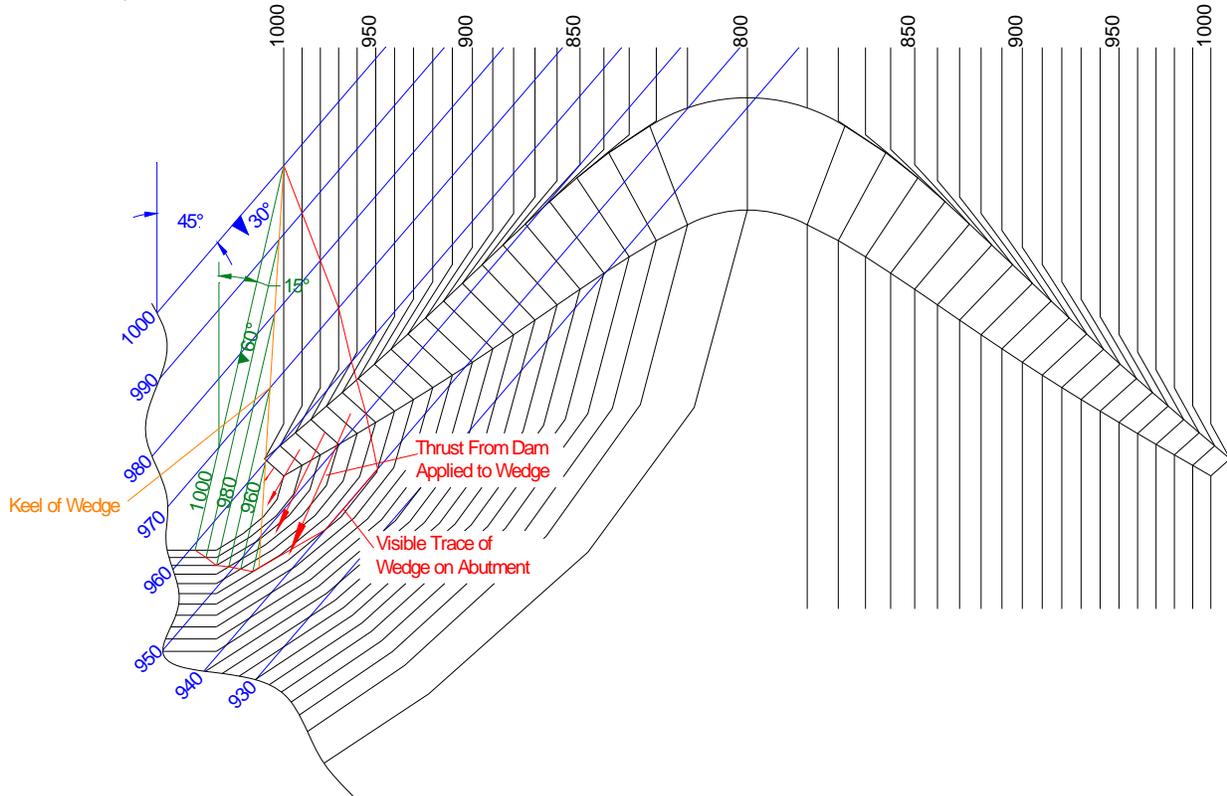
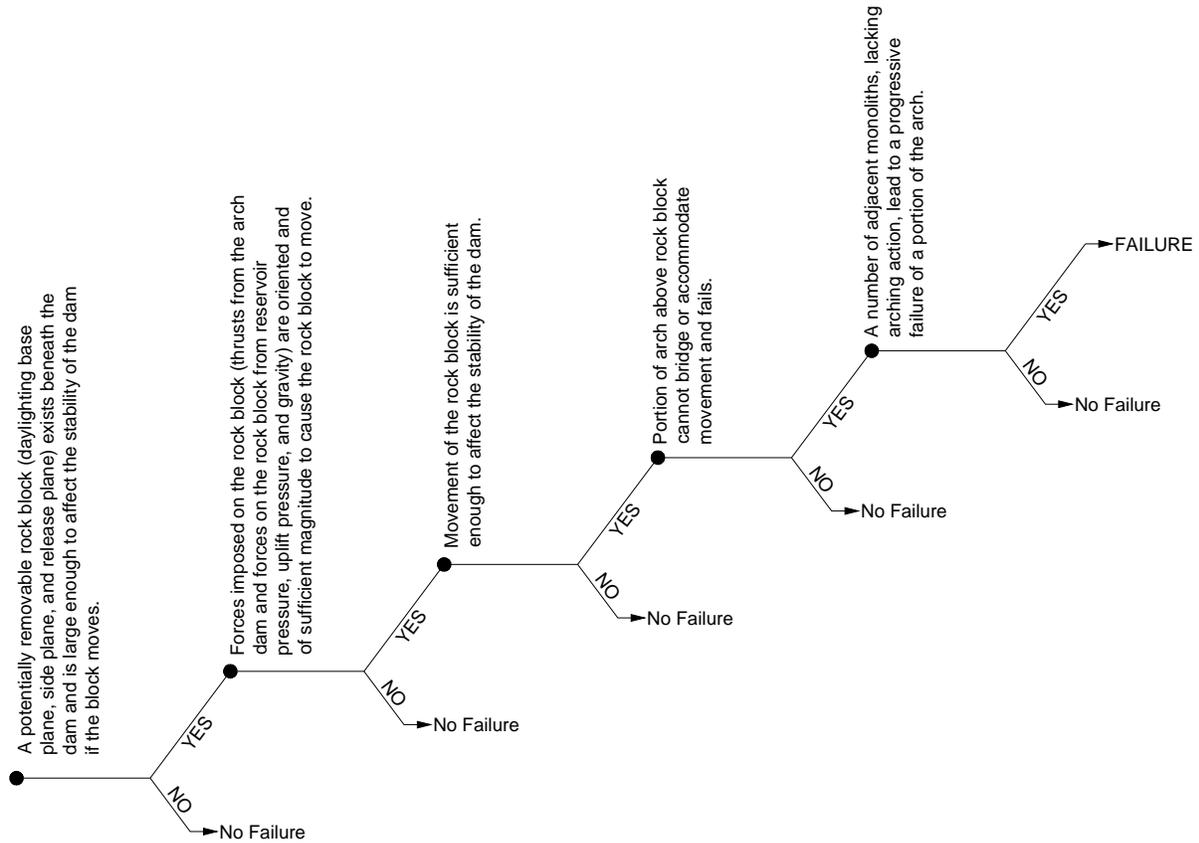


Fig. R5.5.2.1

The failure sequence is as follows:

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R5.5.3 Analysis Techniques to Develop Risk Information

R5.5.3.1 Abutment Instability (See R5.5.1.1 and R5.5.2.1)

1) A potentially removable rock block must exist with a daylighting base plane, side plane, and release plane). All abutments are jointed, but that does not necessarily imply instability. The block must be able to move, i.e. a joint must daylight on the abutment face. Effort must be taken to accurately map joints in the field and determine their continuity. The information gained in the field should be presented in a way that allows one to visualize the block's extent and orientation with respect to the dam and abutment face. The block/dam/abutment kinematic system is three dimensional and care should be taken to present it so the stability implications can be clearly seen. For example in figure R5.5.2.1, a block is defined by the intersection 2 joint sets depicted by blue and green contour lines, and the abutment face.

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2) Forces imposed on the rock block must be oriented and of sufficient magnitude to cause the block to move. Three dimensional rock wedge stability analysis must be performed and the force magnitudes and directions must be determined. Typically the forces imposed by the arch dam are taken out of the output of a finite element study. It is important that the dam forces be accurately determined, and for this reason it is preferable to use modeling software that accounts for the nonlinearities of vertical monolith joints and a dam/foundation contact that is also very likely to be cracked. Reservoir driving forces are easily calculated, however uplift is a function of water flow through the joints in question. For an existing dam, piezometers placed along the discontinuities may be required to develop the 3-dimensional uplift profile around the rock block. As was seen in the Malpasset failure, failure to accurately account for uplift on a potentially unstable block can be disastrous.

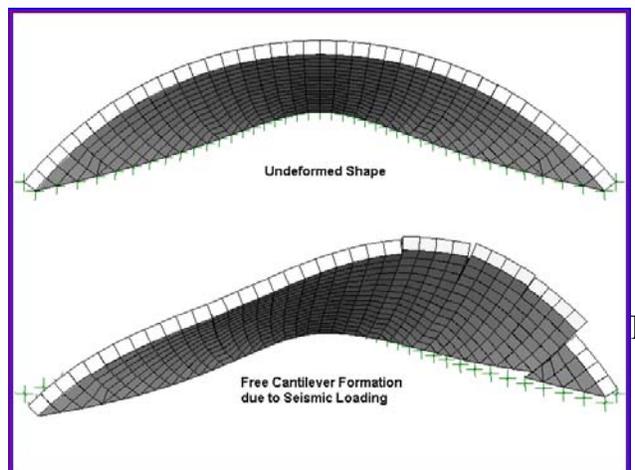
3) Block movement must be of large enough extent that the arch cannot bridge over the failed area. It may be that there is enough structural redundancy in the arch to simply bridge over the failing block. If this is the case, the block may move slightly, but once it does, the forces from the dam that were driving it will be relieved and the block may be stable in its altered location. Therefore, even if it is shown that the block in question is unstable under the applied loads, its movement may not result in dam failure.

To determine whether the dam/block system can fail it is helpful to model the dam and the block together in a coupled analysis using modeling software that can account for joint movements simultaneously in the dam (contraction joints) and foundation (discontinuities). This is typically done with finite element software that includes gap-friction elements that allow joint opening, capture frictional sliding, and can be pressurized with appropriate uplift.

All these analyses are subject to material parameters, assumptions about uplift pressures on the failure plane, and the reservoir elevation. Uncertainties surrounding these items can be evaluated directly using quantitative risk analysis in a manner similar to that laid out in the gravity dam example (R5.4.3.1)

R5.5.3.2 Seismic Induced Failure of an Arch Dam (Shake Table Test R5.5.1.3)

For failure to occur, large blocks of the dam have to move significant distances. As can be seen in the Reclamation shake table tests, cracks must propagate to the



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extent that free blocks are formed. These blocks then have to be able to move in such a way to release the reservoir.

Analysis must focus on the kinematic necessities of the failure mechanism. The failure mechanism must be fully defined and understood before analysis takes place. For example, consider the adjacent figure. At one instant in time, free cantilevers can exist due to upstream movement of a portion of the dam. If free cantilever motion is sufficient to allow the cantilevers to fall back together significantly misaligned, it is possible that the arch action which the dam relies on could be disrupted enough to allow a free block to snap though, and start a progressive failure. The question then becomes “How much relative motion is expected?” For a typical arch dam 10s of feet thick at its thinnest, a few inches of relative displacement would be insignificant.

Rigorous analysis of such a failure mechanism involves dynamic analysis capable of handling both material non-linearities (cracking, sliding, & crushing) and geometric non-linearities (large deflections). Reservoir coupling and wave propagating foundations should also be modeled directly. With current computer hardware and modeling software it often takes days to run one earthquake simulation.

In order to produce a true fragility curve, dozens of earthquake scenarios would have to be analyzed. With the fragility curve, and some information on earthquake magnitude vs. return interval, the probability of failure could be rigorously calculated. This is simply not practical for the majority of Arch Dams under FERC jurisdiction. As stated above, the time and cost of running non linear dynamic analysis for a whole suite of earthquakes for many different failure mechanisms is prohibitive.

This is why it is so important to fully understand the failure mechanism before modeling. Failure mechanisms that are kinematically impossible should be ruled out before rigorous analyses are undertaken.

Linear elastic stress analysis will show where the dam is likely to be overstressed by the earthquake, but overstressing does not necessarily mean failure. It is possible that a linear elastic analysis could show that no cracking or joint opening is expected. In such a case, that would be sufficient to rule out a failure mode. However, if a linear elastic analysis indicates tensile stresses in excess of tensile strength, or any tensile stress across contraction joints or cracks, the only conclusion that can be drawn is that linear elastic analysis was not appropriate. The USACE has a guideline for determining when linear elastic analysis is no longer appropriate based on the magnitude and duration of overstressing predicted. It can be found in [EM 1110-2-6051](#), “Engineering and Design - Time-History Dynamic Analysis of Concrete Hydraulic Structures”. Experience has

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shown that with even moderate ground motions, unrealistic tensile stresses result from linear elastic assumptions, and therefore linear elastic analysis is not likely to be very informative.

Rather than attempt to prove that the dam will not crack, it is more prudent and in most cases more efficient to make reasonably conservative estimates of how much damage will be done by an earthquake, and then do a post earthquake static analysis to see if the dam can resist the static reservoir loading in a damaged condition. Monolith offsets and misalignments can be assumed and input into the post earthquake model. Severe cracking can also be assumed, as well as residual strength parameters on failure planes. The damage done must be estimated based on engineering judgment. The more damage assumed, the more compelling the argument for stability if the dam is stable in the post earthquake condition.

In the example depicted below (Fig R5.5.3.2.1), just this was done. The non linear finite element model was constructed with cantilever offsets as shown in Figure R5.5.3.2.1. Bureau of Reclamation shake table test showed the formation of a through going crack parallel to and slightly above the dam/foundation contact. Therefore the base of the dam was assumed to be fully cracked, with a residual friction angle of 40° . The result of these assumptions is dam failure as depicted in Figure R5.5.3.2.2. The damaged dam fails under post earthquake static loading.

One can always assume enough damage to fail the dam. The question then becomes “Are the damage assumptions reasonable?”. In this example, the assumed cantilever offsets were rather extreme. (5' max). The damage assumption must be appropriate to the magnitude and duration of shaking. Peak ground displacement should be considered. Assumed offsets should be somewhat tied to peak ground displacement. If for example, peak ground displacement was 6", 5 feet of assumed offset would be unreasonable, however if peak ground displacement was 30", 5 feet is more credible.

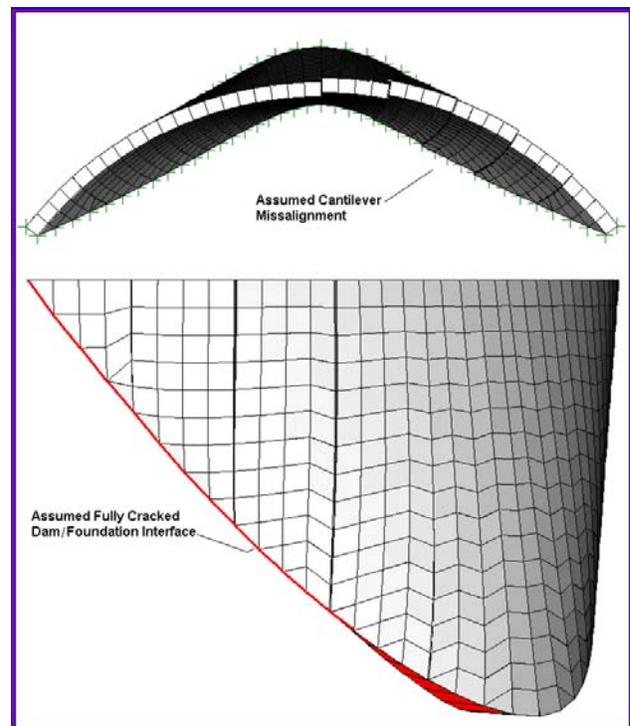


Fig. R5.5.3.2.1

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The generation of a fragility curve is not necessary if it can be shown with reasonable confidence that the dam will not fail under an earthquake loading whose annual probability of exceedence is sufficiently small. It is not necessary to determine the earthquake magnitude that will fail the dam if acceptable performance can be demonstrated under events that are rare enough.

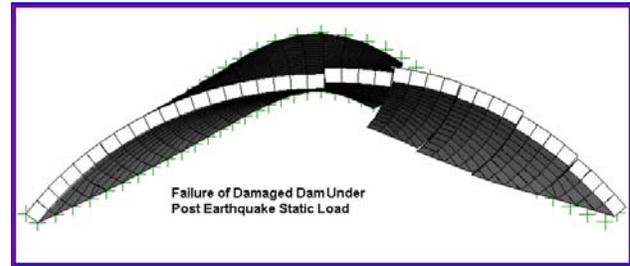


Fig. R5.5.3.2.2

For example, suppose the failure probability of a dam of lower than 10^{-5} were acceptable. If the dam was rigorously modeled with 3 earthquakes with 10^{-5} annual exceedence probabilities or less, and the dam was shown not to fail, the probability of failure would be shown too be sufficiently low, even though a fragility curve was not generated. It is much more valuable to do a good comprehensive non linear dynamic analysis than to generate a fragility curve based on over simplified analyses and conjecture.

What will have to be done in most cases is sufficient analysis to show that under a given level of ground motion the dam will not fail. Beyond that level, failure will simply be assumed. In cases where the cost consequences of assuming a failure are too high, a rigorously determined fragility curve may be warranted.

R5.6 Buttress Dams

These dams include slab and buttress dams, Amberen dams, and buttress arch dams.

R5.6.1 Historic Failures and Near Failures

R5.6.1.1 Morris Sheppard Dam: 188 foot high dam, Fort Worth, Texas – sliding discovered in 1986

Morris Sheppard is a massive slab buttress dam with 40 buttresses. It was completed in 1941 across the Brazos River with a total length of 2740 feet. The dam design did not consider uplift pressures.

During a Federal Energy Regulatory Commission (FERC) mandated inspection,

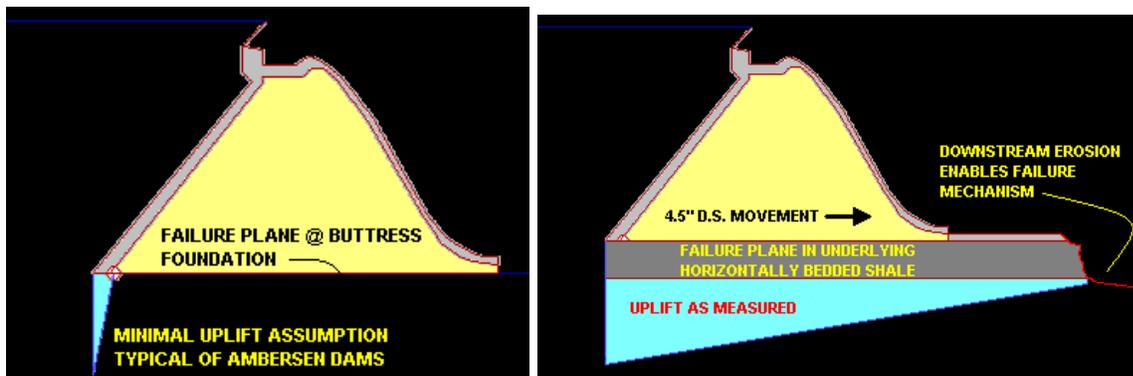


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it was discovered that the hollow spillway section of the dam had moved 4.5 inches downstream. Complete failure did not occur and releases were kept within the channel capacity as the reservoir was quickly lowered 13 feet. Cracks in the footings of the hollow spillway confirmed that the dam had serious problems.

The dam had been constructed over a layer of shale which provided the failure plane. The uplift pressure at the buttress/foundation contact was minimal, however the uplift pressure in the shale was quite high, exacerbating the sliding problem. To remediate the dam a series of 6 inch diameter relief wells, spaced every 13 feet were planned, but some wells encountered pressure flows of up to 450 gallons per minute. A total of 147 wells, some as large as 12 inches in diameter, were installed in an attempt to lower the hydrostatic pressure on the failure plane. The factor of safety against sliding was calculated as 1.5 after the first phase of modifications was completed. Subsequent modifications, such as the addition of ballast to the spillway bays to increase the weight resisting the uplift forces, would bring the factor of safety against sliding to 1.75.

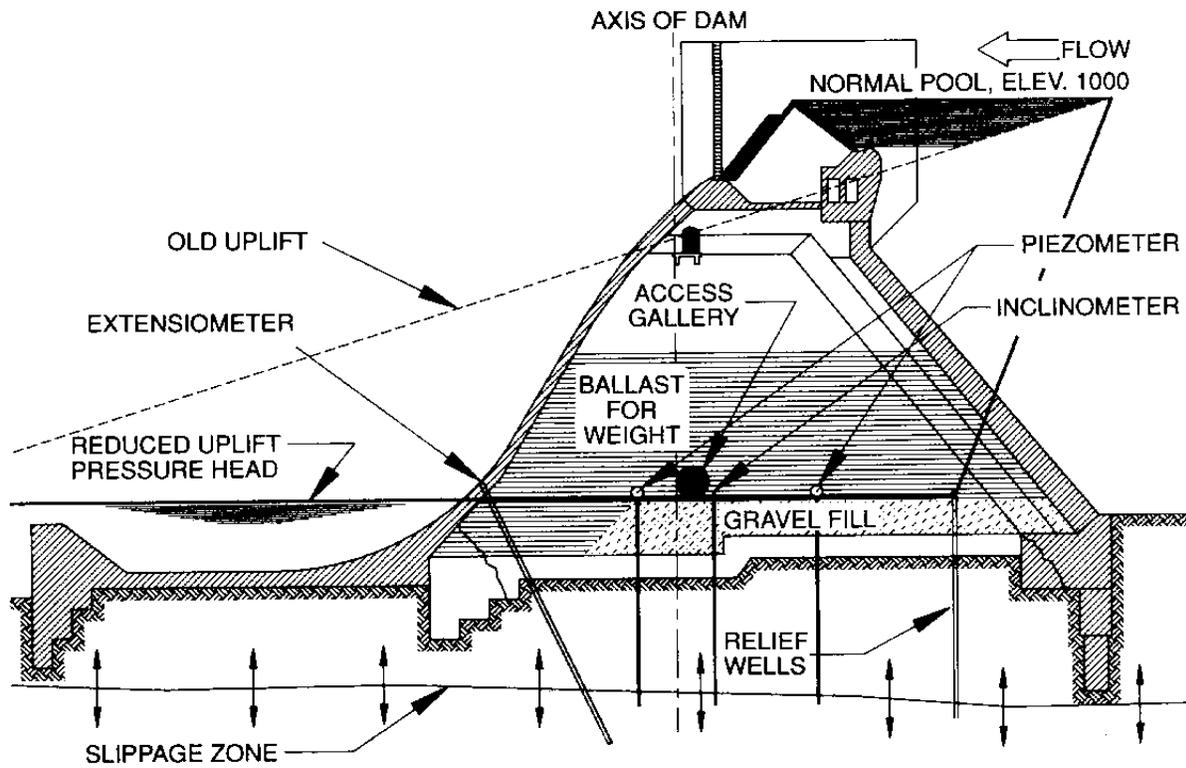


Safety factor of 1.4

Safety factor of 0.38

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Retrofits at Morris Sheppard Dam

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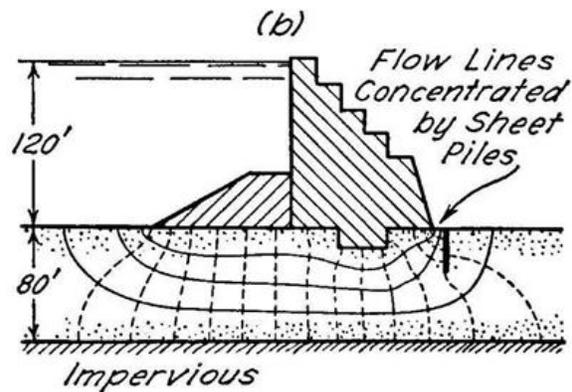
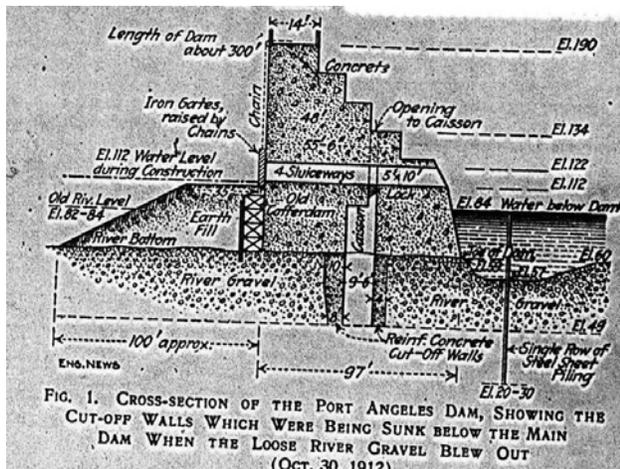
Concrete Dam Analysis - Arch, Gravity and Buttress Dams

R5.5.1.2 Elwa Dam: 108 ft high dam, Washington State, CA - Failed in 1912

The Elwha Dam is located on the Elwha River approximately 4.9 miles upstream from the mouth of the river on the Strait of Juan de Fuca. The dam was built under the direction of Thomas Aldwell. He began construction of the Elwha Dam in 1910.

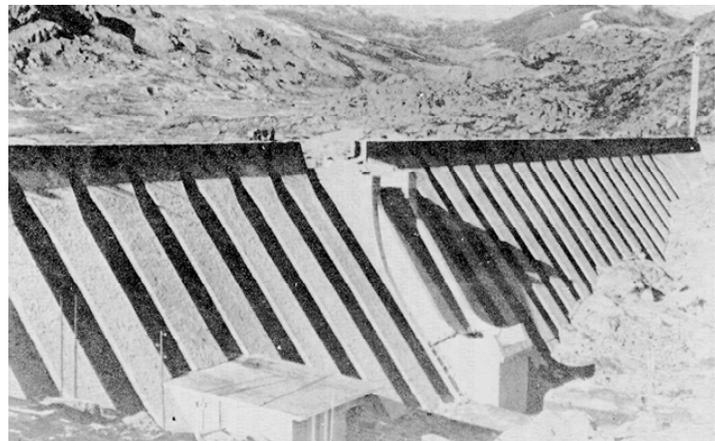


Aldwell and his contractors cut corners on constructing the dam and did not secure it to the bedrock. Thus, in October 1912, as the reservoir filled behind the nearly completed dam, a piping failure of the alluvium under the dam occurred. This caused the lower sections of the dam to give way and a torrent of water took out a bridge. About 10,000 acre feet passed under the dam in about 2 hours, causing a hole that was 75' deep at the upstream face and 90' deep at the downstream face. The main portion of the dam continued to span the hole. This failure caused extensive property damage but no loss of life.



R5.6.1.3 Vega de Tera Dam

Vega de Tera Dam is a 112 ft. high buttress dam made of cement mortared masonry buttresses and



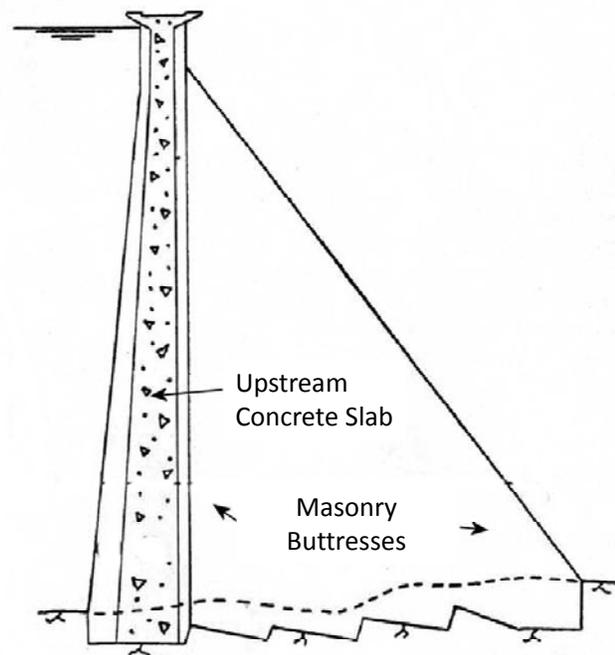
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concrete face slabs for the upstream water barrier. It is located in Northern Spain on a tributary of the Douro River. It was completed in 1957. The upstream face sloped at 1:10 (H:V) and the downstream side of the buttresses sloped at 0.75:1. The upstream face is very steep and did not take advantage of the stabilizing force of the reservoir water on flatter sloping upstream faces typically seen on buttress dams. The upstream face concrete varied in thickness from 4 feet at the crest to 10 feet at the base. The buttresses had a uniform thickness of 14 feet and were spaced 28 feet on centers. Two buttresses and the facing concrete were to function together with joints in the facing spaced at 56 feet apart

The dam failed on January 10, 1959 after heavy rains filled the reservoir to the maximum level for a few days. A micro-seismic shock was recorded at Coimbra in Portugal almost coincidental with the failure. There was a successive collapse of 17 buttresses that released 6,300 acre-feet in a very short period of time. Failure occurred in the depth of winter so no post-failure inspection was done until April 1959.

As a result of the failure, 144 people lost their lives.



As can be seen in the photos above, the



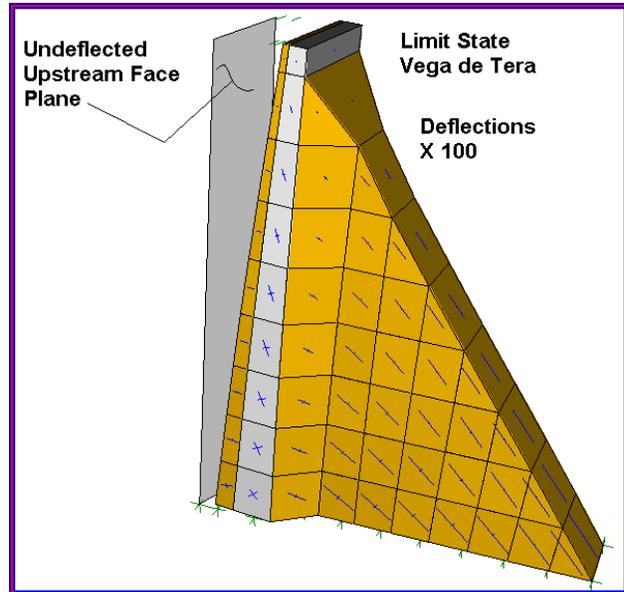
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horizontal failure surfaces on the masonry buttresses suggest sliding along a masonry joint as the failure mechanism.

Back calculation indicates that a failure surface friction angle of 40° should have been sufficient to give a factor of safety of 1.0 at full pool. This does not seem excessive. The highest stress in the buttress was only 180 psi (σ_3), again not excessive. Stresses are compressive and oriented as one would expect as can be seen in the figure adjacent.

All this lends credence to the idea that a weak plane was caused by a construction defect. It was reported that little attention was paid to the lift surfaces after a winter shutdown. This may have been the fatal error.

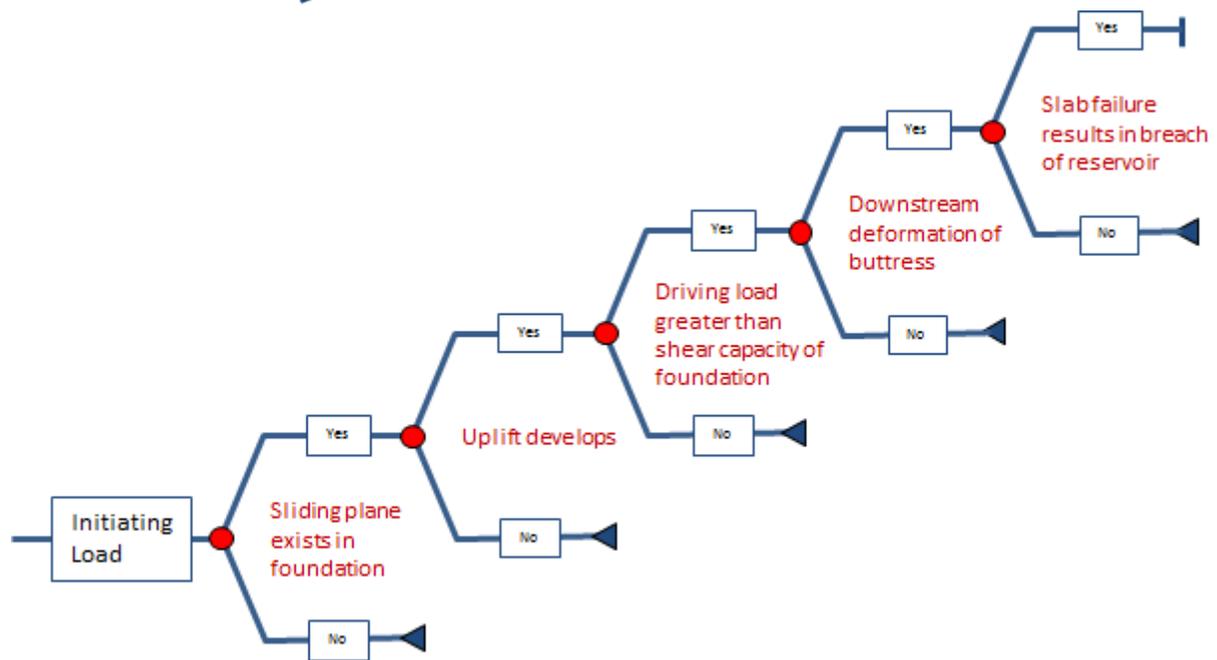
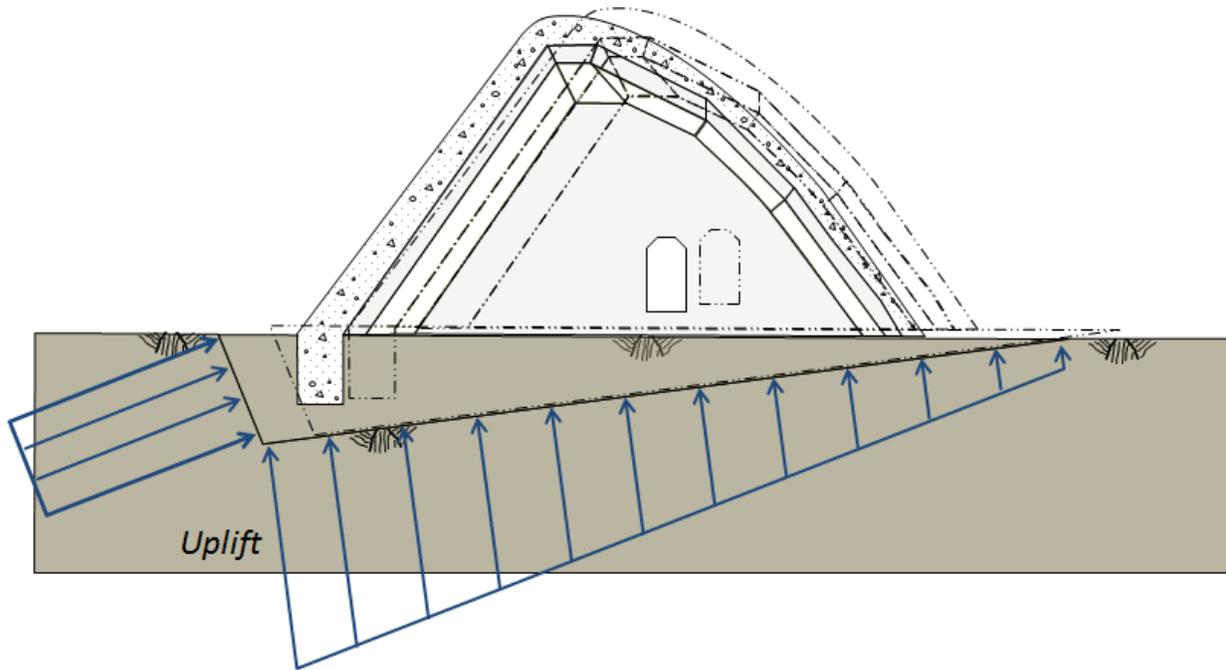


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R5.6.2 Failure Modes

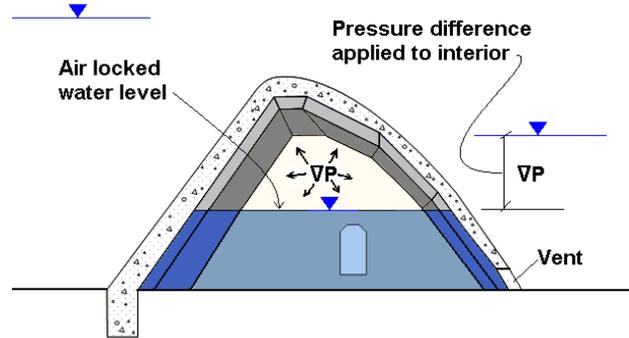
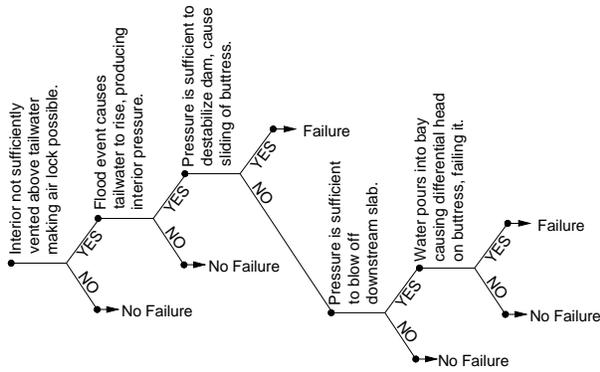
R5.6.2.1 Sliding on a weak layer in the foundation.



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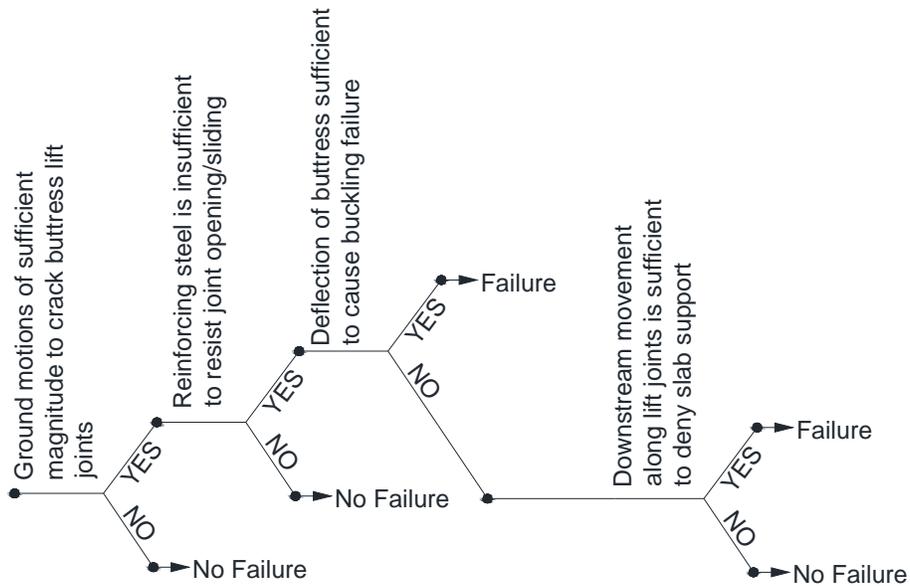
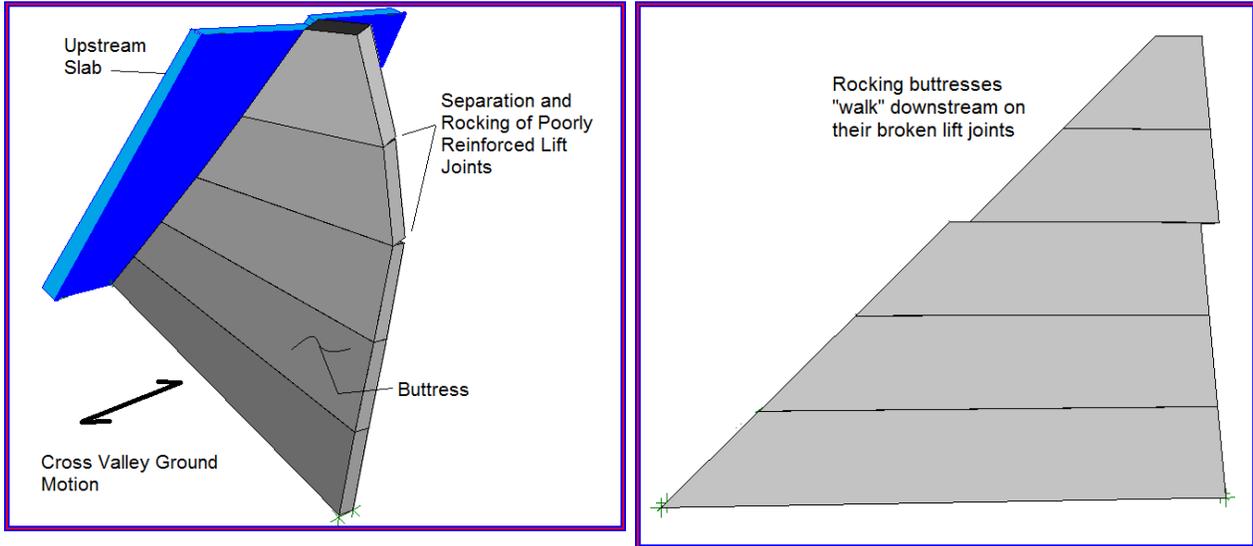
R5.6.2.2 Interior Pressurization of Slab and Buttress Dam.



R5.6.2.3 Cross Channel Seismic Loading.

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R5.6.3 Analysis Techniques to Develop Risk Information

R5.6.3.1 Sliding Along Weak Plane in Foundation (See R5.6.1.1, R5.6.2.1)

For this example it is assumed that at least one buttress is sitting on a layer of shale, creating a sliding plane and allowing for high uplift pressure. For the failure mode to advance it is necessary for the shear capacity of the failure surface to be exceeded.

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Consider that a field investigation shows that cohesion C is 100 psi with a standard deviation of 15 psi and that friction angle ϕ is 10 degrees with a standard deviation of 1 degree. For a normal stress of 200 psi and a shear stress of 100 psi you get the following:

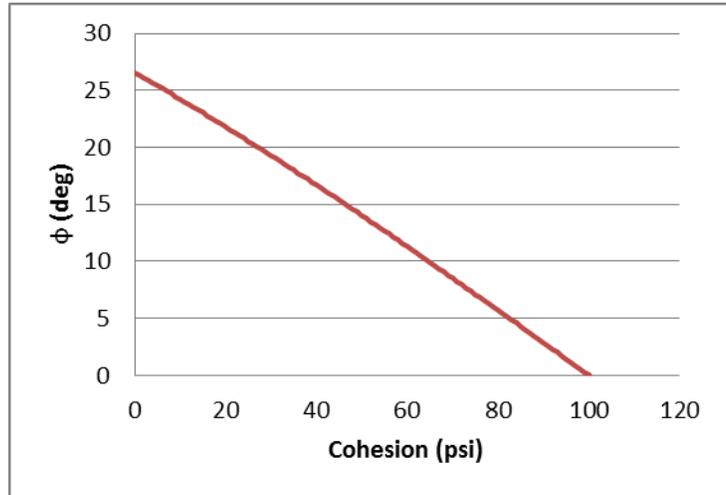
$C + 200 \cdot \tan(\phi) = 100$, which can be solved for ϕ :

$$\phi = \arctan((100 - C) / 200)$$

where ϕ is the critical friction angle at or below which failure occurs for a given C (see graph to the right).

The probability of failure is then:

$$\Pr(\text{Failure}) = \int_0^{\tau} \text{PDF}(C) * (\text{Cum Pr}(\phi)) \delta C$$



This can be integrated numerically using Excel functions such as Norm.Dist. The result is a failure probability of 1.1%.

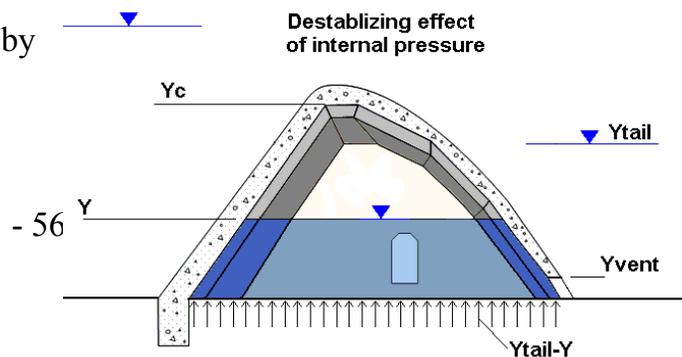
It is important to note that the probability of failure can increase dramatically if the layer of shale affects multiple buttresses. What has been calculated above the failure probability for 1 buttress. If there are 10 buttresses subject to the same failure mode, then the probability of failure would be:

$$1 - (1 - 0.011)^{10} = 0.1047, \text{ or } 10.47\%$$

R5.6.3.2 Interior Pressurization of Slab and Buttress Dam.

Conditions have to be right for air locking to occur. (See failure mode R5.6.2.2) If the top portion of the interior of the dam is vented to free air, and the vent is large enough to accommodate the air flow produced by rising tailwater, then this failure mode is precluded at the first node. In many cases it may be prudent to do no risk analysis at all, but provide enough ventilation to ensure atmospheric pressure in the interior of the dam.

If not, pressure buildup can be computed by solving for Y in the 3rd order equation shown below.



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$$P_{interior} = P_{atm} \left(\frac{Y_c - Y_{vent}}{Y_c - Y} \right)^2 = P_{atm} + (Y_{tail} - Y)$$

Where:

P_{atm} = Atmospheric pressure (feet of head).

Y = Water elevation in the interior (unknown).

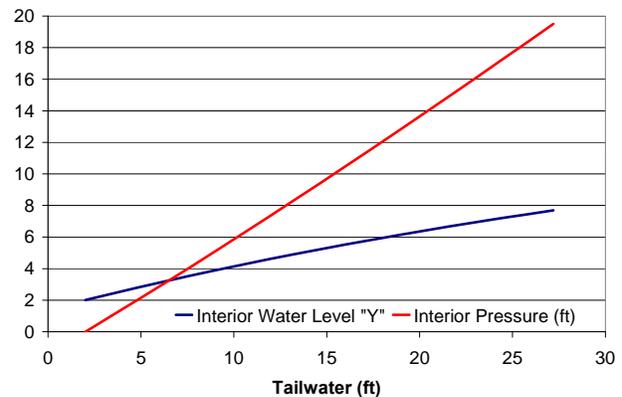
Y_c = The elevation of the crown of the interior.

Y_{vent} = The elevation of the top of the vent

Y_{tail} = Tailwater elevation.

Interior water level and pressure can be computed as a function of tailwater elevation as shown adjacent. Here, Y_c was 30', Y_{vent} was 2'.

Note there are 2 routes to failure in the event tree depicted in R5.6.2.4. The stability of the dam can be computed including the destabilizing effect of interior pressure, which in effect, unweights the dam, reducing frictional resistance to sliding.



The pressure required to blow off a portion of the downstream slab may be quite small. Often downstream slabs were not designed for load reversal. The interior pressure added to potential negative crest pressures due to the overflowing nappe may be substantial.

Both routes to failure are contingent on tailwater elevation, which in turn is a function of flow. Therefore probability of failure can be tied to the AEP of stream flow.

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R5.6.3.3 Cross Channel Seismic Loading

The event tree depicted in R5.6.2.3 has 2 paths to failure. The buttresses can buckle like an axially loaded column or, they can “walk” downstream waddling side to side and slipping a minute amount with each oscillation of the seismic event. Although the buttresses usually have some form of horizontal bracing, it is usually designed ensure that the buttresses remain vertical during normal loading conditions and not designed to withstand the additional load from the earthquake. If horizontal bracing is included in the model, it must be structurally capable of resisting all loads imposed by the earthquake. If it is not, it should be ignored. Ignoring lateral bracing means that these models can be single buttress models with appropriate boundary conditions imposed at slab midspan.

The evaluation of both failure mechanisms requires the inclusion of 2 types of non-linear behavior; 1) cracking/sliding of the failure planes and 2) the geometric non-linearity sometimes called the p - δ effect. The analyses are fret with the same problems as does the evaluation of seismically induced failure of an arch dam covered in R5.5.3.2. Several earthquake records would have to be evaluated to show that at a sufficiently rare ground motion magnitude, the dam would not fail.

The first scenario for the buttress buckling during seismic event occurs because the reservoir load on the slab produces in plane forces in the buttresses, while the inertia loads on the buttress during the seismic event provides the out of plane loading. This out of plane loading cracks the concrete and which allows the increased deflection of the buttress. This increased deflection causes a p - δ effect amplifying the deflection.

Buttresses are often significantly under reinforced. Because the reinforced cracked moment capacity of the buttress is significantly less than the uncracked moment capacity, once the buttress is loaded and the concrete cracks, the movement of the buttress automatically yields the steel and the moment capacity of the buttress is almost completely lost. With the reservoir producing an in plane compression field, the buttress continues to bend toward failure, resulting in the catastrophic release of the reservoir.

The deflections required to fully buckle a buttress are on the order of $\frac{1}{2}$ the buttress width. This means that in most cases, significant ground displacements must occur. In the selection of earthquake records, it is important to look not just at peak ground acceleration, but also ground displacements. Records devoid of long period content should not be used.

The second scenario for failure is that during a seismic event, the out-of-plane bending of the buttress cracks the concrete at cold joints or at the foundation. The seismic event

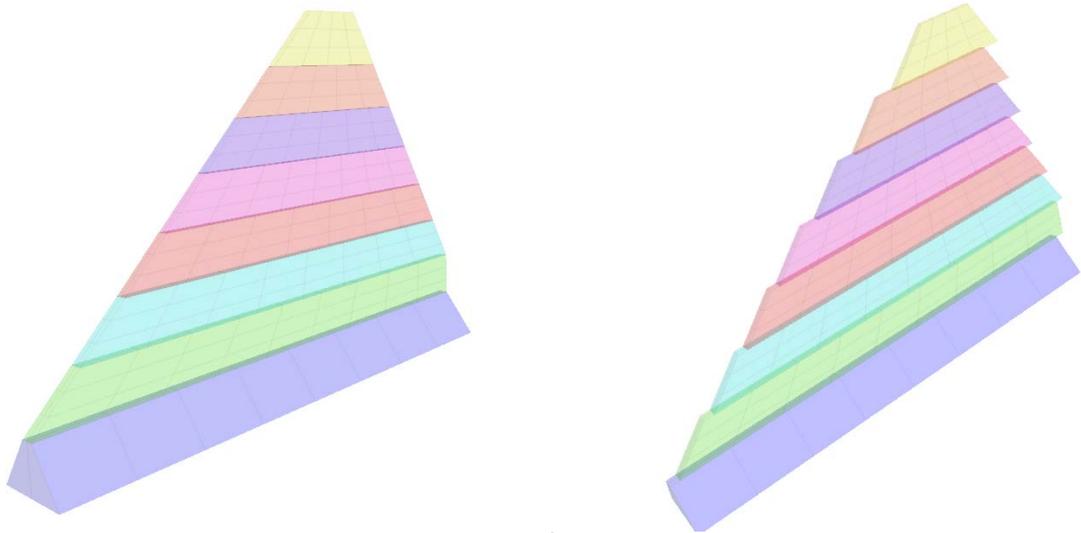
Chapter R5

Concrete Dam Analysis - Arch, Gravity and Buttress Dams

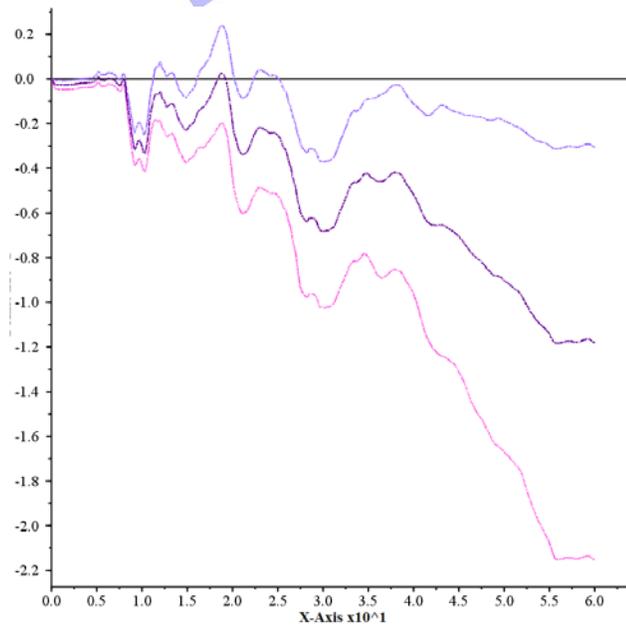
does not impart enough energy to cause the buttresses to topple or to fail in buckling as described above, instead the buttress rotates slightly at these locations and as the buttress rotates, the reservoir load pushes it a very small amount downstream. The movement of the sections is cumulative with each movement increasing the displacement downstream. When sufficient movement has occurred, the slab is no longer supported well and fails, this causes a domino effect causing additional slabs to fail and leads to the loss of the reservoir.

Unlike the buckling scenario described above, the displacement needed for enough rotation of the buttress sections to allow the downstream walking is very small. This means that this failure mechanism can be driven by smaller and more frequent earthquakes

The figure below shows the displacement of the buttress sections downstream before and after the seismic event.



The adjacent figure shows the displacement of different lifts of the buttress during the seismic event. As can be seen in the graph, during the earthquake, the buttress lifts progressively walk downstream. There is no one area of significant



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slippage, but a progressive movement. As can be seen in the graph, the buttress becomes stable again once the shaking stops. For slab and buttress dams in low seismic areas, this downstream progression might not be a concern.

However, for areas of significant seismicity especially in areas those are expected to experience long duration shaking, this movement could be significant and should definitely be evaluated.

R5.7 Monitoring and Surveillance

R5.7.1 Sliding on a Failure Plane (See R5.4.2.1, R5.5.2.1, R5.6.2.1)

1) Movement monitoring is valuable for detection of the onset of dam and rock block movement. Regular high precision surveys can detect movements on the order of a fraction of an inch. Inclinerometers installed across potential failure planes not only detect movement but can also indicate the location of the joint slippage that is causing the movement.

2) Uplift pressure monitoring can be very valuable if the failure mechanism is highly effected by uplift (which it usually is). Rising uplift pressure gives a more advanced warning of a potentially destabilizing situation than does movement monitoring, which only is helpful after a certain amount of movement has occurred.

R5.7.2 Seismic (See R5.5.3.2)

1) Accelerometers/Seismographs at the dam site are valuable tools for determining the magnitude of the seismic load the dam was subjected to. Ideally, it is good to have 1 3 axis accelerometer at a rock location near the dam, but structurally unaffected by it, and 1 near the crest of the dam. This allows determination of not only the site ground motion, but also how the dam responded to the ground motion.

2) Survey markers if present can be used to determine any permanent displacements resulting from the earthquake. Crack monitors are also useful for this purpose.