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E-4 Risk Analysis for Concrete Arch Dams

E-4.1 Key Concepts

A concrete arch dam provides redundant load carrying capacity. That is, an arch is a very forgiving structure. If one part of the structure is overstressed, for example due to cantilever cracking (e.g. horizontal cracking due to vertical tensile stress) at the upstream heel, the load can be transferred to other parts of the structure and transmitted by arch action to the abutments. Therefore, it is not surprising that historically, the leading cause of concrete arch dam failures has been related to sliding on planes of weakness within the foundation.

There are no known failures of a concrete arch dam due to concrete structural distress or seismic loading. Even Plum Dam, a 70-foot-high arch dam in Fjian Province, China, whose failure in 1981 under static loading is often attributed to structural deficiencies, actually failed by sliding on an artificial joint, constructed near the base of the dam to relieve cantilever tensile stresses (Zuo 1987). No arch dams are known to have failed statically or seismically after five years of successful operation having reached normal operating reservoir level; failures typically occurred during first-filling.

A line of functioning drainage holes in the foundation adds significantly to the sliding stability of concrete arch dams by reducing water pressures (typically referred to as “uplift”) along potential sliding surfaces. A decrease in water pressure increases the effective normal stress and frictional resistance. However, drainage systems can become plugged over time if they are not maintained, and the drainage curtain can be offset under significant seismic displacements, thus becoming less effective.

The tensile strength of concrete is typically an important consideration, particularly in estimating seismic risks for concrete arch dams. See “Chapter E-3, Risk Analysis for Concrete Gravity Structures” for a discussion of concrete tensile strength.

Estimating static risks for concrete arch dams is a difficult proposition. There is not a relatively simple analysis model that can be used to perform a reliability analysis (except for simple foundation blocks as described later), since everything must be analyzed three-dimensionally. Therefore, traditional structural and foundation analyses and significant judgment are typically used in estimating arch dam risks (see “Chapter A-6, Subjective Probability and Expert Elicitation”). See Scott (1999), Koltuniuk et al. (2013), and Scott and Mills-Bria (2008) for additional information on conducting such analyses.

E-4.2 Risks Under Normal Operations

Concrete arch dams that have performed well under normal operating conditions will likely continue to do so unless something changes. Changes could result from plugging of drains leading to an increase in foundation uplift pressures, possible gradual creep that reduces the shear strength on potential sliding surfaces, or degradation of the concrete from alkali-aggregate reaction, freeze-thaw deterioration, or sulfate attack. Some of these may be difficult to detect.

A review of instrumentation results can be helpful. For example, if piezometers or uplift pressure gauges indicate a rise in pressures, and weirs indicate a reduction in drain flows, the drains may be plugging leading to higher uplift and potentially unstable conditions. If conditions appear to be changing, risk estimates are typically made for projected conditions as well as current conditions.

Arch dams are typically designed for normal reservoir water surface elevations that induce compression of the arch. If an arch dam (especially a thin arch dam) experiences drawdown of the reservoir and is exposed to warm temperatures for long periods of time, the arch will tend to expand upstream, causing tensions on the downstream face. If the tensions are high enough, cracking can begin to form both horizontally and diagonally. This likely will not cause a failure under normal operations, but it can create a reduced cross-section of intact concrete when considering the addition of seismic load.

If there are no well-defined potential failure modes and no particular issues associated with a concrete arch dam, it may be appropriate to consider the risks to be negligible under normal loading conditions. If there is a well-defined potential failure mode with a clear series of events that could lead to failure, an event tree should be developed and the risks estimated as described elsewhere in this manual. In the case of foundation block instability, such as that shown on figure E-4-1, if such a block is simple enough a rigid block analysis can be programmed into a spreadsheet, and Monte Carlo reliability analysis performed as described in “Chapter A-7, Probabilistic Approaches to Limit-State Analyses”). Equations for such an analysis considering a block defined by three planes can be found in Hendron et al (1980).

E-4.3 Risks Under Flood Loading

A concrete arch dam will experience an increase in hydrostatic reservoir loading during floods. If the flood is large enough, the arch and abutments could be subject to overtopping and the associated erosion forces acting on the abutments. Evaluating this condition is discussed in “Chapter F-2, Overtopping of Walls and Stilling Basin Failure.” If the dam does not overtop, or overtops to a limited extent such that the abutments are not compromised, then the increased loading on the arch and foundation needs to be evaluated.

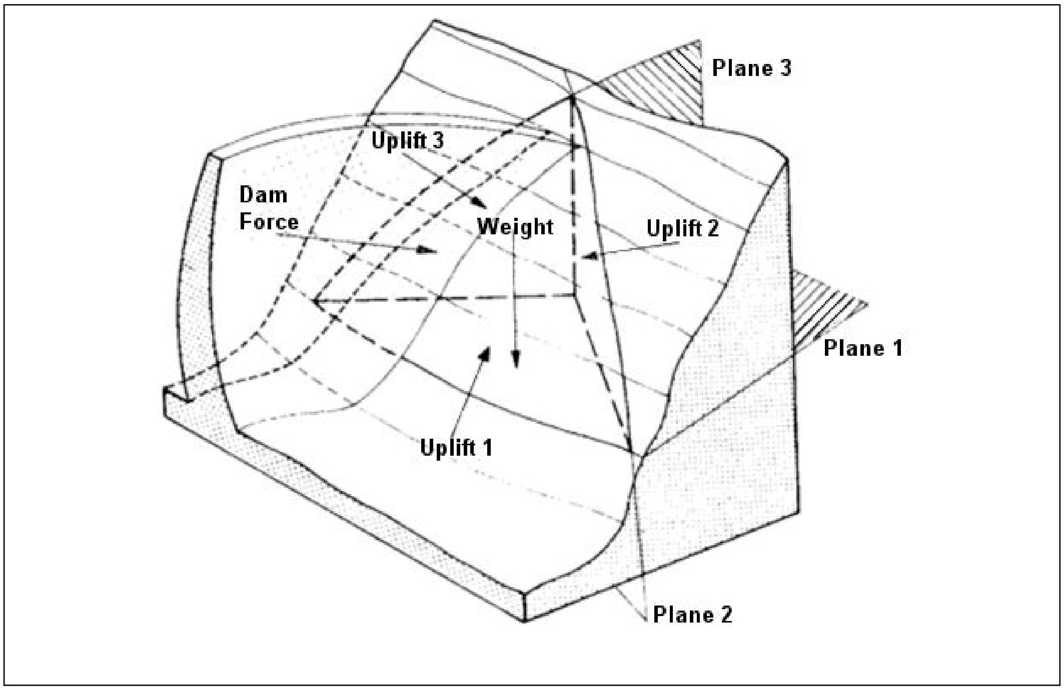

Figure E-4-1.—Typical arch dam foundation block (adapted from Londe 1973).

In the case of structural loading on the arch itself, the increase in loading associated with floods that do not threaten the overtopping stability of the dam is typically small in comparison to the remoteness of the loading. For an arch that carries the static loads well, it is unlikely that flood loading will change the stress distribution enough to be of concern, unless the dam is a flood control dam where most of the upper half of the structure is only loaded during a flood, in which case, the design probably considered this effect. In any case, the structure should be analyzed for the increased loading, and the resulting stress distribution compared to the normal static loading conditions. The appropriate concrete temperature condition should be included with the flood loading, depending on what time of year the flooding is expected.

A possibly more serious condition occurs when there is an abutment foundation block upon which the dam rests, that could become unstable under increased loading due to flood conditions. The increase in reservoir not only affects the dam loads on the block, but also the water forces on the block bounding planes (joints, faults, shears, bedding plane partings, foliation planes, etc.). Overtopping flows could enter discontinuities downstream of the dam, further pressurizing these features. Therefore, it is important to perform abutment stability analyses under the increased loading (see Scott 1999).

Based on the analyses, a judgment is then made as to whether the failure probability is significantly higher under flood loading than normal loading conditions, and if so, an event tree should be set up to evaluate the increase in failure probability in relation to various flood load ranges and their likelihood (see "Chapter A-5, Event Trees"). This may require analyses at various load ranges up to and including the maximum (typically the Probable Maximum Flood).

E-4.4 Risks Under Earthquake Loading

Under earthquake loading, concrete arch dams will respond according to the level and frequency of the shaking, and the reservoir level at the time of shaking. Therefore, sufficient analyses need to be performed to evaluate conditional failure probabilities at various levels of shaking and reservoir elevation. Typically, both structural and foundation failure modes need to be considered.

E-4.4.1 Structural Failure

Since there have been no known arch dam failures as a result of earthquake shaking, there is no direct empirical evidence to indicate how an arch dam would structurally fail under this type of loading. Therefore, shake table model studies have been performed to gain some insight into what might happen (Payne 2002). In the models, and attempt was made to maintain material similitude between the model material and typical dam concrete. The model foundation and abutments were essentially rigid (built of conventional concrete). Some models included contraction joints and/or weak lift joints by inserting plastic sheets in the modeling material as it was placed. However, it was not possible to maintain similitude with the reservoir, and water was used in the models. Therefore, it is difficult to conclude anything about the level and duration of shaking that could lead to failure from the models. Nevertheless, the failure modes were relatively consistent and provide some indication as to how an arch dam might fail under earthquake shaking. In these tests:

- Failure initiated by horizontal (cantilever) cracking across the lower central portion of the dam.
- This was followed by diagonal cracking parallel to the abutments.
- The cracking propagated through the model, forming isolated blocks within the dam.
- Eventually, the isolated blocks rotated and swung downstream releasing the reservoir (figure E-4-2).

With this description forming the sequence of events for potential failure, an example event tree is shown on figure E-4-3. The complete tree is shown for only one branch. In this case, a single reservoir elevation was picked as being critical to the response of the dam and the resulting consequences. However, this was relatively unimportant as the reservoir is above the critical elevation 97 percent of

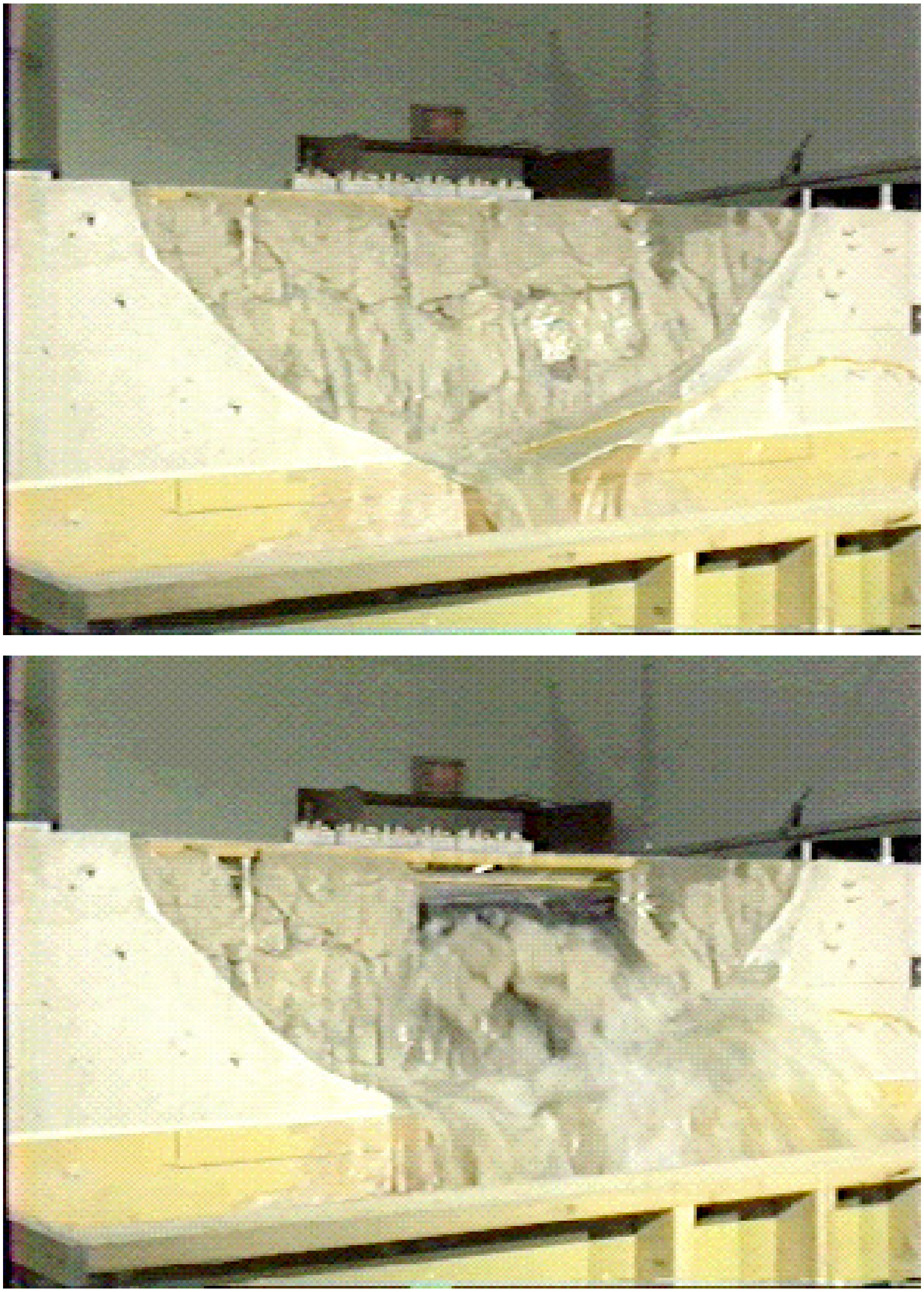

Figure E-4-2.—Seismic failure mode manifest by model shake table tests.

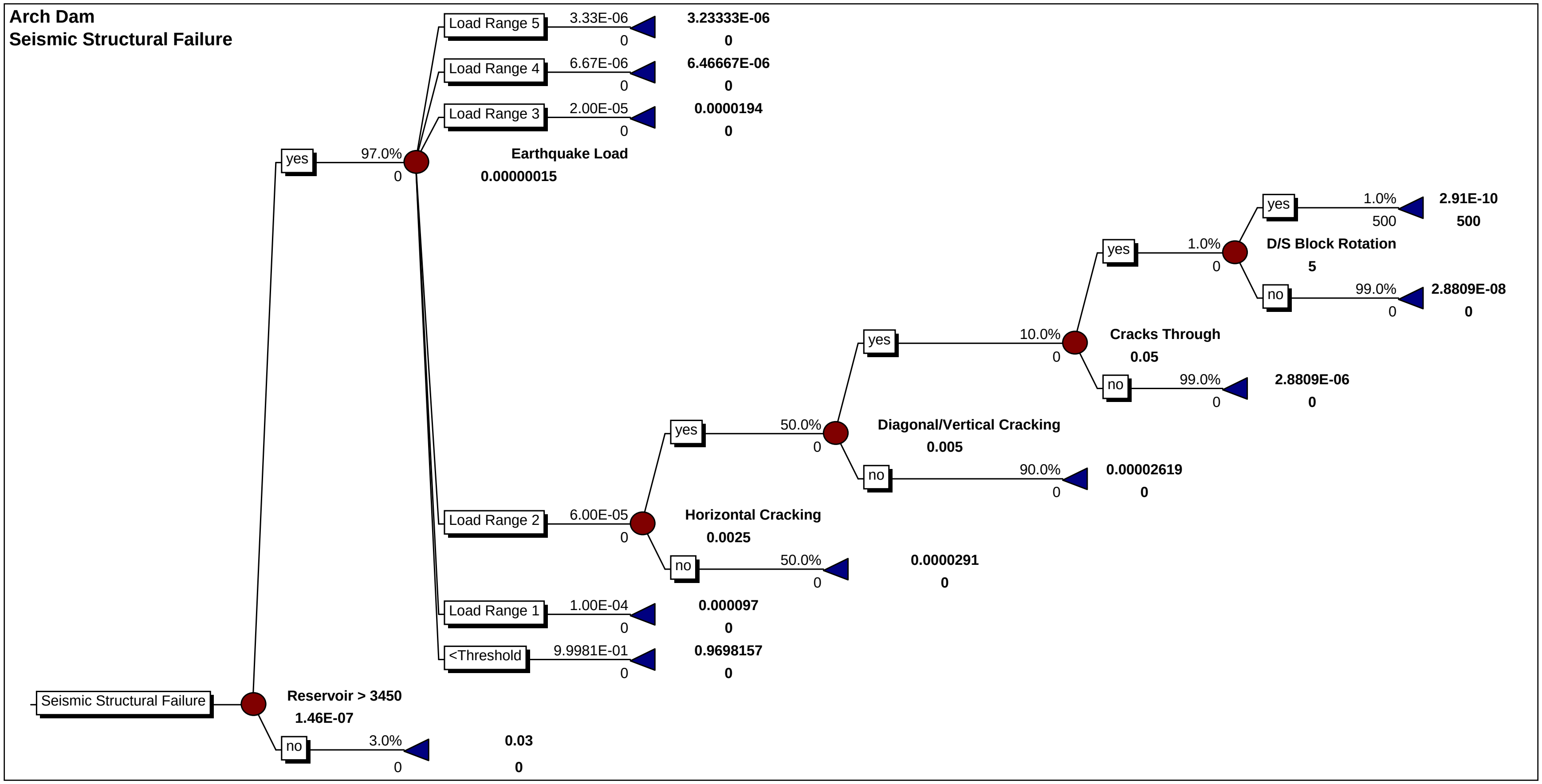

Figure E-4-3.—Example arch dam seismic structural event tree.

the time. It is necessary to perform structural analyses at various earthquake loading levels to evaluate the potential for each subsequent step in the tree. If the reservoir fluctuates significantly over the course of a year, and the response changes dramatically with reservoir elevation, it may also be necessary to include different reservoir load ranges (with the appropriate temperature conditions) in the tree as well. Similarly, if the concrete temperature conditions control the initial static stress conditions, and the results are highly sensitive to these initial stresses, it may be appropriate to set up the tree with the various temperature conditions (and corresponding reservoir elevations).

For each reservoir and seismic load range that is established for the estimating process (see "Chapter A-5, Event Trees"), the likelihood of cracking at various orientations (over significant areas – not just localized cracking), and cracking through the dam body must be estimated. The best approach for this is to perform three-dimensional dynamic time-history finite element studies. Examination of output stresses and their duration indicates whether the estimated dynamic tensile strength of the concrete is exceeded at the dam faces during seismic response. The orientation of the principal stresses can be used to gauge the likely crack directions for various times during the earthquake loading. As the amplitude of these stresses and number of excursions above the tensile strength increase, the likelihood of through cracking also increases. One or two excursions above the tensile strength do not necessarily equate to a high likelihood of cracking, particularly if the amplitudes do not exceed the tensile strength by a significant amount. If contraction joints are modeled in a nonlinear analysis, it is possible that diagonal principal stresses will not develop due to "opening" of the contraction joints. In this case, the potential for "stair-stepped" cracking roughly parallel to the abutments must be assessed.

If the shaking is severe enough, the cracking can extend through the structure forming an adverse cracking pattern. This is typically evaluated by examining the magnitude and duration of tensile stresses that exceed the dynamic concrete tensile strength on both the upstream and downstream faces. Again, judgment is required on the likelihood of cracking through the structure at various earthquake (and possibly reservoir/temperature) levels. The contraction joints may facilitate in forming continuous failure planes, but they are typically oriented in a direction that promotes stability (radial). However, if the contractions joints are not oriented radially, they could contribute to seismic instability. The team should consider this when estimating likelihood of cracking through the structure.

If the arch cracks all the way through, the likelihood of isolated blocks rotating and displacing downstream is next estimated. Information typically used to make this assessment includes the duration of strong shaking and stresses. The primary questions to be asked is, "how likely is it that there will be sufficient energy left in the earthquake to move blocks downstream after the dam has cracked through?" and "how likely is it that the cracking pattern will be adverse enough that the portion of the dam isolated by the cracking will be unstable statically following the shaking?" For the latter to occur, the "semicircular" cracking pattern that typically results in the upper part of the dam (figure E-4-4) must be smaller on the upstream face than the downstream face, such that the cracks diverge in the downstream direction. Most modern finite element programs have the capability to generate color contour plots of vertical or principal stresses, such as that shown on figure E-4-5 for a nonlinear analysis where contraction joints are allowed to open. These may not be as useful as stress vectors in the plane of the face. However, it may be important to evaluate the effects of contraction joint opening in relieving tensile arch stresses, by performing such nonlinear analyses.

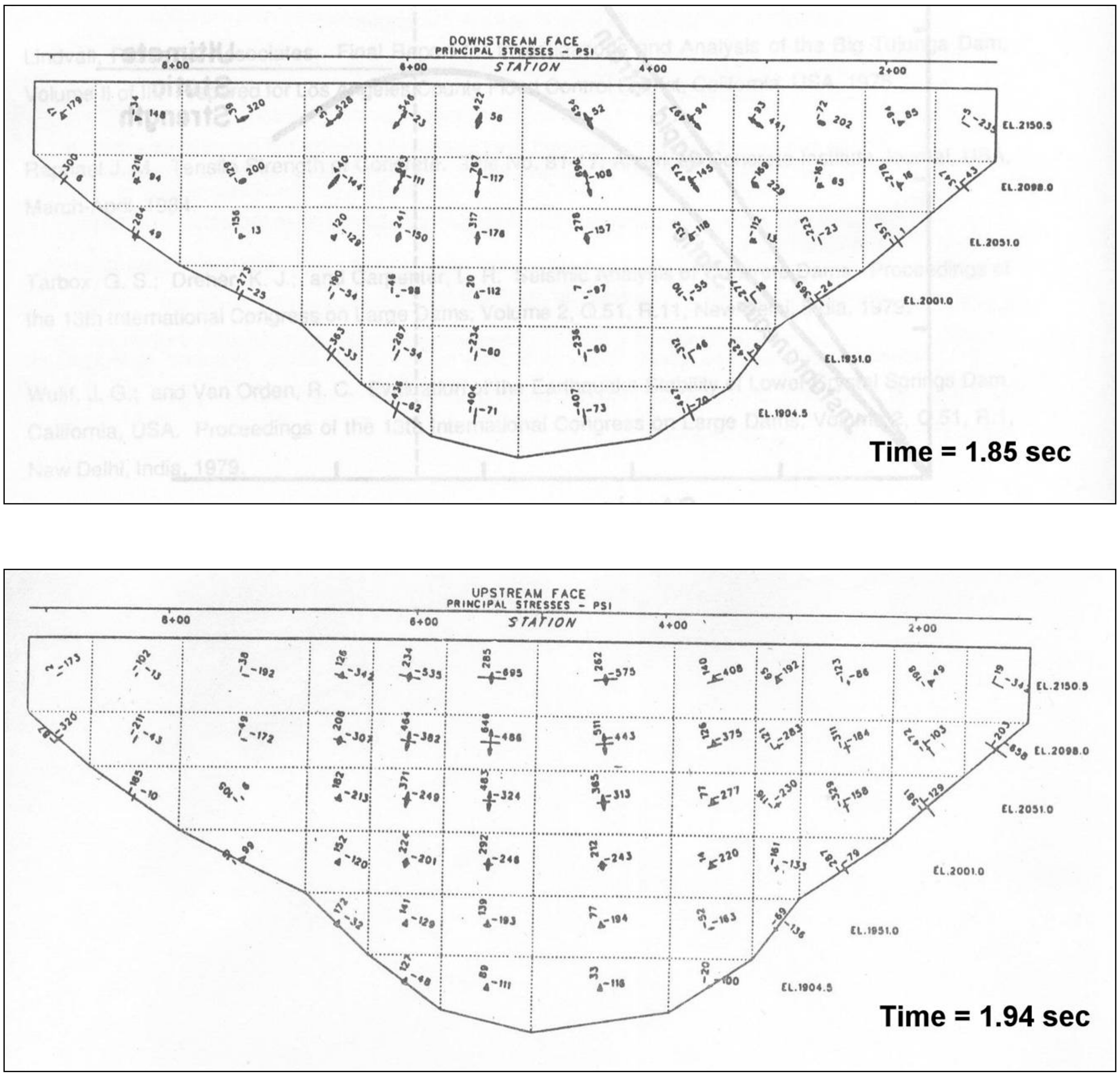

Figure E-4-4.—Semi-circular cracking pattern (normal to tensile principal stress vectors) evident in upper portion of arch dam from linear elastic analysis.

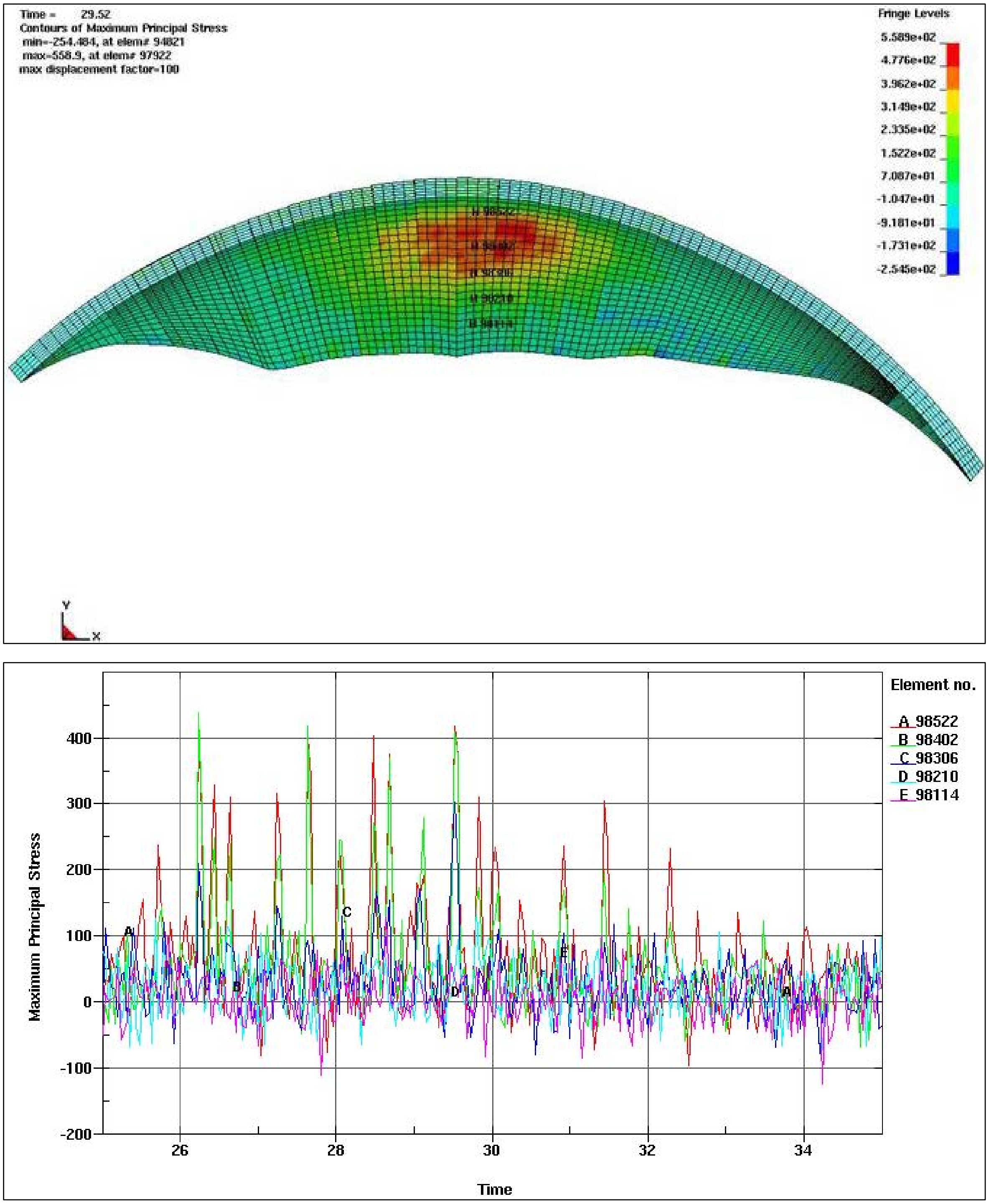

Figure E-4-5.—High maximum principal stresses evident in upper portion of arch dam and associated stress histories from LS-DYNA non-linear analysis.

E-4.4.2 Foundation Failure

Historically, arch dam failures have resulted primarily from foundation deficiencies. Sliding of large blocks (bounded by geologic discontinuities) within the foundation or abutments of an arch dam resulted in some of the biggest civil engineering disasters of the 20th Century (see notable case histories below). Typically, these failures were sudden, brittle, and occurred on first-filling of the reservoir. Although no concrete dam foundations are known to have failed as a result of earthquake shaking, unprecedented seismic loads would in effect be a first-loading condition that could trigger movement and failure of arch dam foundation blocks. Therefore, it is important to analyze and evaluate the risks associated with potential earthquake induced foundation instability.

The first step is identifying and describing the potential failure mode as discussed in "Chapter A-3, "Potential Failure Mode Analysis." Typically, this involves identifying geologic discontinuities that could form blocks within the foundation of an arch dam capable of moving under the applied loads. A typical event tree for this type of failure mode associated with a gravity arch dam is shown on figure E-4-6. Again, the branch for only one load range is shown completely. This tree does not have additional breakdown for reservoir load ranges or temperature conditions. In some cases, especially for thin arch dams, the loading and subsequent foundation stability may be highly dependent on these factors, in which case they should be added near the beginning of the event tree.

The first node in the example event tree deals with how likely it is that the planes bounding a foundation block are continuous enough to form a block that will not be solidly held in place by intact rock bridges. Discontinuities forming block back release planes near the upstream side of the dam will typically be placed into tension during movement. Therefore, intact rock on such planes will not impede movement to the same extent as on side or base planes, where intact rock bridges would need to shear in order for block movement to occur. The field evidence is weighed, and judgments made for this node. Engineering geologists should be part of the estimating process for this node.

Following the earthquake load ranges, the next node deals with the likelihood of movement under the applied loads. This typically requires stability analyses to examine the factors of safety. Initially, uncoupled analyses are performed whereby loading from the dam is calculated from finite element analyses and applied in a separate rigid block foundation analysis. The methods for this type of analysis are described in Scott (1999).

Typically, time-history rigid block analyses are performed, and when the factor of safety drops below 1.0 during the earthquake, the permanent displacement can be estimated using the so called "Newmark" method. These would be conservative "worst case" displacements since it is assumed that the loads follow the block as it displaces. In reality, load would be redistributed by the dam.

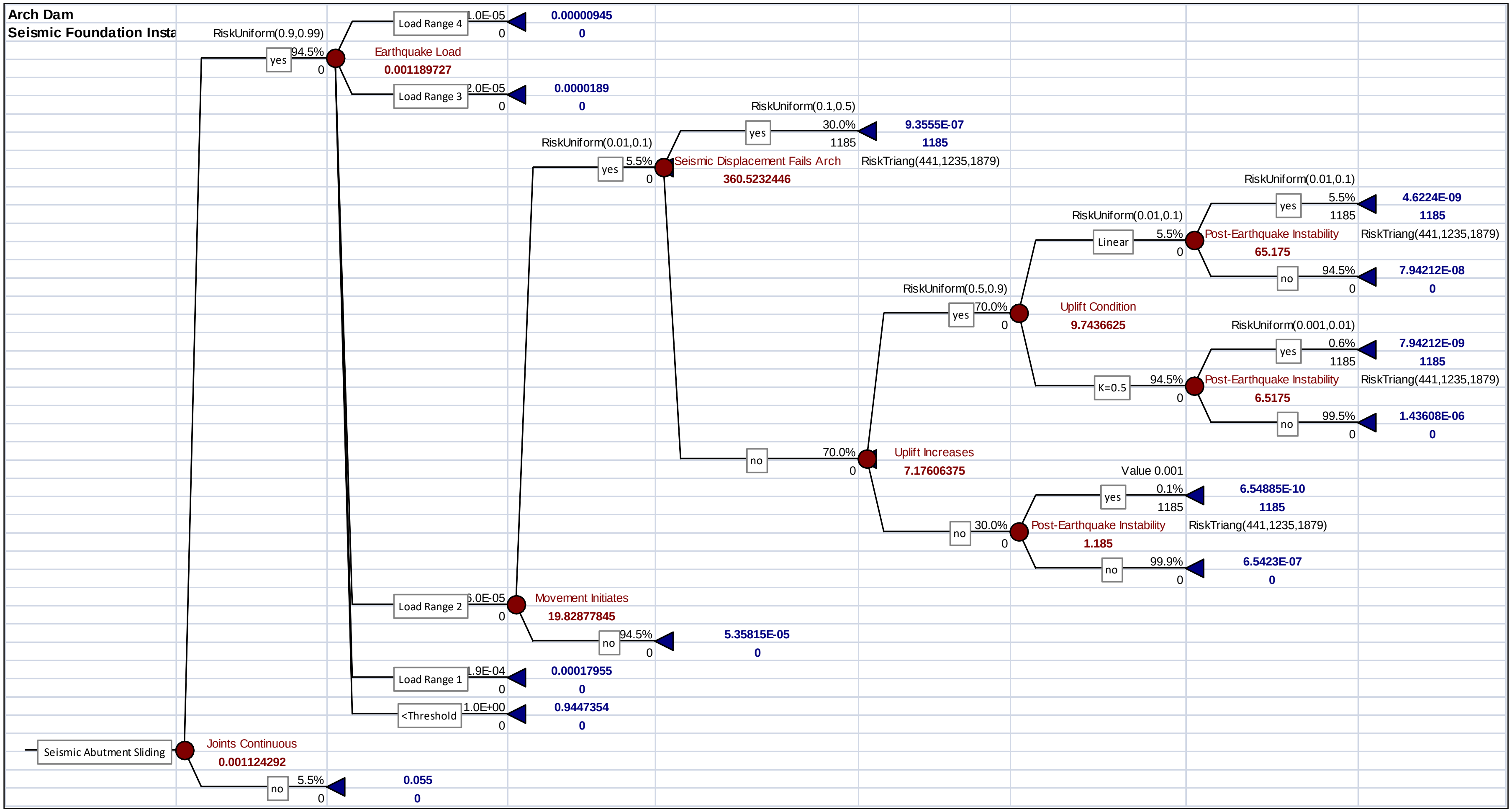

Figure E-4-6.—Concrete thick arch dam foundation instability, seismic loading.

In certain critical cases, a coupled dam-foundation analysis may be warranted. See Powell et al (2008) and Scott and Mills-Bria (2008) for additional information related to performing these types of analyses.

Judgment is required to estimate the likelihood of movement initiating based upon the results of the analyses. Model uncertainty (how well the model represents the actual field conditions) as well as uncertainty in the input parameters should be taken into consideration in making these estimates. Just because a model predicts movement does not necessarily mean there is a 100 percent chance of movement initiating.

For thin arch dams, sufficient movement may be generated during the shaking to cause rupture of the dam. If that is the case, a separate node can be placed next in the event tree to capture that possibility, as shown on figure E-4-6. Given that movement initiates but is not enough to fail the dam, the next node asks the question, how likely is it that water forces acting on the block planes will increase as a result of the movement? Mechanisms that could lead to this include opening of a back-release plane, which allows full hydrostatic reservoir head to penetrate the full depth of the block (which is normally assumed to occur under static loading anyway), dilation of block planes that allow more water to penetrate along the planes, or severing/disruption of foundation drains that could impair their ability to relieve seepage pressures. The primary consideration for estimating the likelihood of increased pressures involves the likely range in estimated displacement magnitude. If maximum dilation is achieved, more water will enter the planes. If the drains are completely severed by the movement, it is likely they will be less effective in reducing foundation water pressures. If the flow paths pinch off downstream, the uplift will increase.

If there is some probability that the foundation water forces will not increase, the example event tree (see figure E-4-6) indicates the likelihood of post-earthquake foundation instability would still be estimated for this case. This may not be necessary (i.e. if uplift doesn’t increase, the branch ends with “no failure”) depending on the stability analysis results and the consequences of dam failure. If the block is extremely stable as long as the water pressures don’t increase, and thus the associated small likelihoods would not contribute significantly to the risk, this node could be eliminated from the tree.

The next node addresses to what extent the foundation water pressures might increase, given that they actually increase. In the example event tree, two cases were included. The first would be that associated with movement that would not render the drains completely ineffective but would reduce their effectiveness.
In this case, additional analyses would be needed to examine post-earthquake foundation stability under the new estimated drainage effectiveness. The second possibility shown in the example event tree involves complete offset of the drains that, in combination with increased flow, renders them essentially ineffective. For this case, additional analyses would be performed with foundation water pressures varying from full reservoir head on the upstream side to tailwater (or the daylight

elevation of the potential sliding planes) on the downstream side. It may be possible to eliminate this node, if the estimated displacements are such that there is only one concept of how the foundation water pressures might be affected.

Stability analyses simulating post-earthquake conditions are used to assess the likelihood of post-earthquake instability (i.e. the last node on the example event tree). Again, if the foundation block is relatively simple and uncoupled analyses are used, a spreadsheet could be programmed, and a reliability or probabilistic stability analysis could be performed as described in “Chapter A-7, Probabilistic Approaches to Limit-State Analyses”). Otherwise, judgment and subjective probability is used, as described in the section on Judgmental Probabilities and Expert Elicitation.

E-4.4.3 Foundation Contact

It is desirable to have the foundation contact of an arch dam excavated to radial lines or at least partially radial lines. In some cases, this was not achieved, and the excavation surfaces dip downstream on sections cut radial to the dam axis. Even if the abutments were excavated radial to the axis, severe earthquake shaking could break the bond between the dam and foundation. Subsequent sliding and rotation at the base of concrete monoliths could disrupt the geometry such that arch action is lost and subsequent instability ensues.

Figure E-4-7 shows an example event tree for this potential failure mode, associated with a thick arch dam. For each seismic load range, the first node represents the likelihood of separation at the contact. In order to assess this node, forces from the finite element analyses must be resolved parallel and perpendicular to the dam-foundation contact. Some nonlinear finite element programs will do this automatically if “contact surfaces” are used to model the interface. In this case, “tie-break” tensile and shear values are input, and the output of nodes that separate are output, as shown on figure E-4-8. It is important to note that some finite element codes assume the contact surface is broken when the following criterion is satisfied:

$$\left(\frac{|\sigma_n|}{NFS}\right)^2 + \left(\frac{|\tau|}{SFS}\right)^2 \geq 1$$

Equation E-4-1

Where:

σ_n = Normal stress
NFS = Normal failure stress
T = Shear stress
SFS = Shear failure stress

If the normal stress is compressive, the first term is ignored. Thus, the frictional component of shear strength is not considered, which could be quite conservative

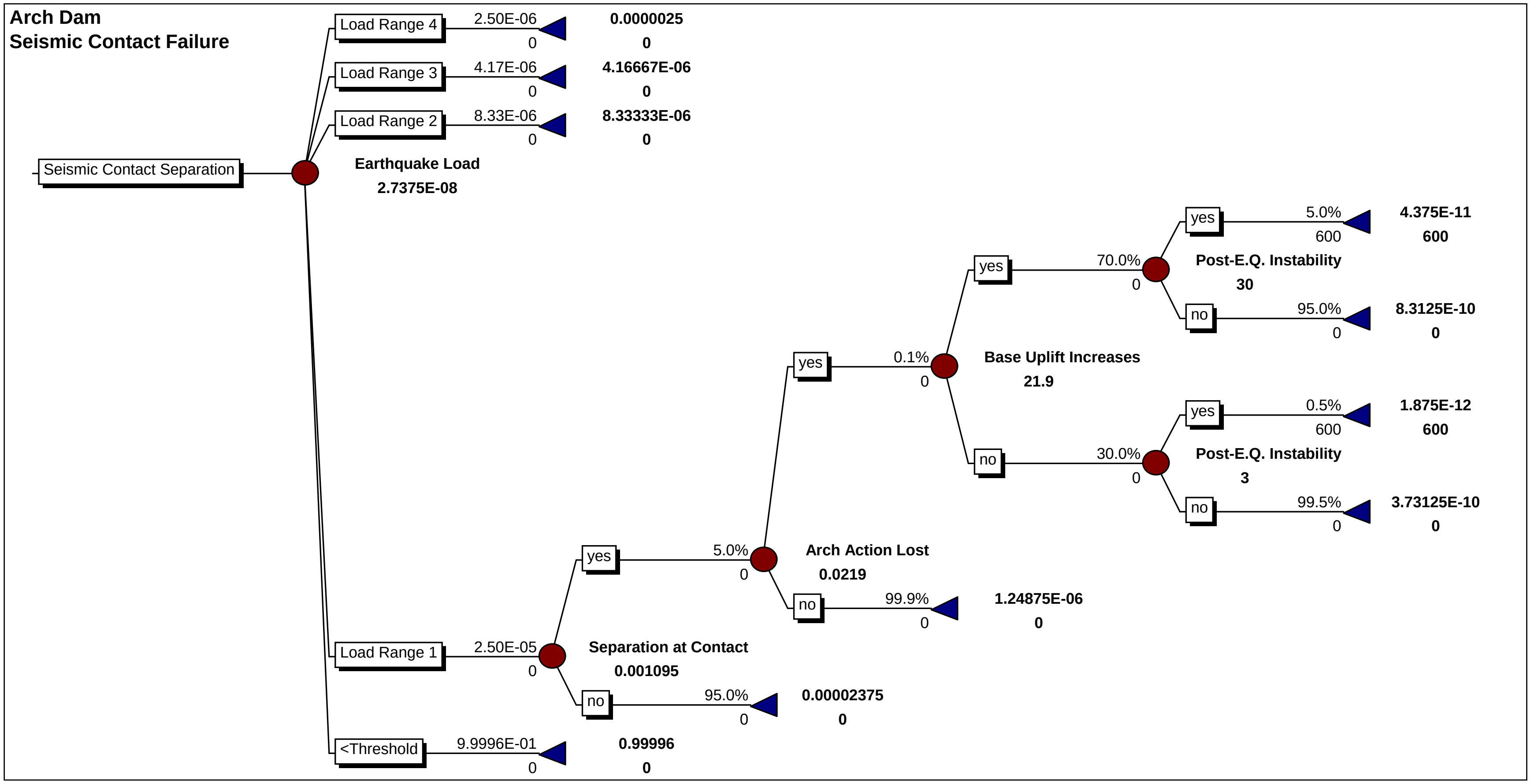

Figure E-4-7.—Example event tree for thick arch dam foundation contact failure.

Figure E-4-8.—Nodes that separated for 50k earthquake ground motions at 5% damping.

when shear failures are predicted. If nonlinear analyses are used, there must be extensive calibration, validation testing, and verification of the model to help ensure the output results are reasonable. See Scott and Mills-Bria (2008) for a description of the types of testing that are needed.

In the case of linear elastic analyses, forces or reactions are typically calculated at nodal points on the dam-foundation interface. These forces are then appropriately summed and resolved normal and parallel to each element face. The appropriate failure criterion can then be applied to the resulting force distribution.

The next node deals with the likelihood of sliding or rotation of the concrete monoliths to the extent that arch action is lost as a result of loading that occurs after bond is broken at the contact. Again, if contraction joints between monoliths and the foundation contact are modeled with contact surfaces in a nonlinear analysis, insights into whether this is likely to occur can be gained directly from the analysis results. For linear elastic analyses, more judgment is needed in making the estimates. For thin arch dams, if arch action is lost, the dam will likely fail.

If the dam is a thick arch, such that a section of the dam may be stable two-dimensionally, the remaining nodes of the example event tree on figure E-4-7 relate to whether stability is achieved once arch action is lost. Similar to the foundation instability potential failure mode, movement can result in disruption of drainage at the contact. The likelihood of an increase in the base uplift, along with how much it is likely to increase must be estimated. Then, the likelihood of post-earthquake instability, with or without an increase in uplift, needs to be estimated. This last step is typically estimated using reliability analysis, as discussed in “Chapter A-7, Probabilistic Approaches to Limit-State Analyses”).

E-4.5 Accounting for Uncertainty

Uncertainty is accounted for by estimating a range or distribution of values for each node on the event tree. A Monte-Carlo analysis is then run for the event tree to display the “cloud” of uncertainty, as described in “Chapter A-8, Combining and Portraying Risks.” Sensitivity analyses are then performed on key parameters to evaluate the changes to the risk estimates.

E-4.6 Relevant Case Histories

E-4.6.1 St. Francis Dam: 1928

St. Francis Dam was a curved concrete gravity dam constructed in San Francisquito Canyon approximately 45 miles north of Los Angeles California. The dam was 205 feet high, 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 composed of two types of rock; the canyon floor and left abutment were composed of relatively uniform mica schist, with the foliation planes dipping toward the canyon at about 35 degrees. The upper portion of the right abutment was composed of a red conglomerate, separated from the schist by a fault dipping about 35 degrees into the right abutment.

During reservoir filling, two sets of cracks appeared on the face of the dam that were dismissed as a natural result of concrete curing. The reservoir stood within 3 inches of the overflow spillway crest for 5 days before the failure. 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. One of the caretakers was seen on the crest of the dam about an hour before failure. 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, as evidenced by the time the Southern California Edison power line downstream was broken. Within 70 minutes, the entire 38,000-acre-foot reservoir was drained. An immense wall of water devastated the river channel for 54 miles to the Pacific Ocean. It has been estimated that 470 lives were lost, but the exact count will never be known (Anderson et al. 1998). Reanalysis of the disaster indicated that failure initiated by sliding along weak foliation planes in the left abutment, perhaps on a remnant of an old paleo-landslide.

E-4.6.2 Malpasset Dam: 1959

Malpasset Dam was a 216-foot-high thin concrete arch structure 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 and the windows and doors of his house, on a hillside about 1 mile downstream of the dam, blew out. The sudden failure sent a flood wave down the river causing total 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).

E-4.6.3 Pacoima Dam: 1971, 1994

Pacoima Dam is a flood control arch dam located in the San Gabriel Mountains north of Los Angeles. It is 370 feet high, 10.4 feet thick at the crest and 99 feet thick at the base. The left abutment is supported by a 60-foot-tall thrust block. The dam was shaken by the 1971 M6.6 San Fernando earthquake, and the 1994 M6.8 Northridge Earthquake. The dam survived both events, but the reservoir was low in both cases. As a result of the 1971 earthquake, a crack formed in the thrust block, a previously grouted contraction joint opened up, and extensive cracks accompanied by displacements up to 8 inches vertically and 10 inches horizontally were found in the gunite which covered the left abutment. Three potentially unstable rock blocks were identified in this abutment, one of which underlies the thrust block. Tendons were designed and installed to prevent movement under future large seismic events. Following the 1994 earthquake, permanent vertical offsets appeared along most of the vertical joints at the crest of the dam, with the elevation of each block dropping from left to right. The joint between the dam and thrust block opened two inches at the crest and a quarter inch at the base of the thrust block. The left abutment gunite was again severely cracked, with evidence that foundation blocks moved 16 to 19 inches horizontally and 12 inches downward at the surface. Elongation and overstressing of the tendons near the thrust block probably occurred. A zone in the tunnel spillway concrete lining, about 20 feet long, was displaced and cracked along a discontinuity in the rock.

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